

**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORTS
ORIOLE GO PARKING OVERPASS
STRUCTURE,
HIGHWAY 401 REHABILITATION FROM
LESLIE STREET TO WARDEN AVENUE
MTO CENTRAL REGION, G.W.P. 2130-01-00
GEOCRES 30M14-333**

Delcan Corporation
Project: TRANETOB01245AA-AC
September 30, 2011

September 30, 2011

Delcan Corporation
625 Cochrane Drive, Suite 500
Markham, Ontario
L3R 9R9

Attention: Ms. Draga Daniel, P.Eng.

Dear Madam:

**RE: Preliminary Foundation Investigation and Design Report
Oriole GO Parking Overpass Structure, Highway 401 Rehabilitation from Leslie Street
to Warden Avenue, MTO Central Region, G.W.P. 2130-01-00, Geocres 30M14-333**

Please find attached our preliminary foundation investigation and design reports relating to the above noted site.

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

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Principal Engineer

**PRELIMINARY FOUNDATION
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OVERPASS STRUCTURE
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**PRELIMINARY FOUNDATION INVESTIGATION REPORT
ORIOLE GO PARKING OVERPASS STRUCTURE,
HIGHWAY 401 REHABILITATION
FROM LESLIE STREET TO WARDEN AVENUE
MTO CENTRAL REGION, G.W.P. 2130-01-00**

1 INTRODUCTION

As part of the proposed rehabilitation of Highway 401 from Leslie Street to Warden Avenue, three highway ramp structures were originally planned to be rehabilitated at the early stage of the project. Coffey Geotechnics Inc. (Coffey) was retained by Delcan Corporation (Delcan) to carry out a preliminary foundation investigation at the site of the proposed rehabilitation of the following existing highway ramp structures.

<u>Structure Name</u>	<u>MTO Structure Number</u>
Highway 401 Overpass at Leslie Street/C.N.R. Ramp W-N/S	37-206/5
Highway 401 Overpass at Leslie Street/C.N.R. Ramp N-E	37-206/6
Highway 401 Overpass at Leslie Street/C.N.R. Ramp N-W	37-206/7

Subsequently, the project scope was changed in late 2010 after the completion of Coffey's foundation investigation program. The new scope is as follows:

- New CN Rail (C.N.R.) overpass structure (single span rigid frame structure)
- New structure(s) over the existing Oriole GO parking
- New Leslie Street overpass structure (two span rigid frame structure)
- Viaduct (northwest quadrant of Highway 401 and Leslie Street interchange)

As Coffey's foundation investigation was carried out for the previous rehabilitation plan for the ramp structures, boreholes drilled by Coffey were generally advanced outside of the newly proposed structures. No additional boreholes were put down for the newly proposed structures and Coffey was asked to prepare this preliminary foundation investigation report based on the available subsurface information only including Coffey's recent boreholes.

This preliminary foundation investigation report is prepared for the proposed new structure(s) over Oriole GO parking. Based on the information provided to us by Delcan, the existing Oriole GO parking lot overpass structure, which retains Highway 401 traffic over the existing Oriole GO Parking Lot, will be replaced with new structures. It is our understanding that details of the new Highway 401 structures over the Oriole GO parking lot, and the applicable foundation details, will be developed during detail design.

2 SITE DESCRIPTION AND GEOLOGY

The site is located in between the existing CN track and Leslie Street, as shown on Drawing 1. In general, the existing grade along Highway 401 falls from the west, from about El. 152 m, to the east to about El. 144 m, above Leslie Street. The elevation of Leslie Street below the interchange is approximately 136 m.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located within the Physiographic Region known as the 'South Slope'.

Within this general area, the overburden consists of Pleistocene or glacial deposits, which were laid down under a vast thickness of ice or as a result of deposition by glacial rivers and lakes associated with the glaciers. Soils which were deposited by the ice are described as glacial till deposits which are mainly unsorted by water action, while those formed by melt waters are typically stratified deposits.

In summary, below the existing fill material at the site, this area is underlain by silty sand to clayey silt (shallow lake deposits - Peel Pond), silty clay (deeper lake deposits - Peel Pond or Lake Iroquois), glacial till and sand deposits.

The depth of the overburden in the general area can be expected to be more than 50 m, with the surface of the bedrock anticipated at about El. 75 to 90 m. The bedrock consists of the grey/dark grey Georgian Bay shale with limestone and siltstone/sandstone interbeds. The formation belongs to the Upper Ordovician Period of the Paleozoic Era and is approximately 440 million years old.

3 SUBSURFACE CONDITIONS

3.1 Past Reports

The existing subsurface information from MTO GEOCRES information system was used to prepare this report. A number of previous geotechnical investigations has been conducted at the site and these investigations are summarized in our previous geotechnical assessment report titled "Draft-Foundation Engineering Assessment Report, Highway 401 and Leslie Street Interchange, Toronto, Ontario, GWP 2130-01-00, Agreement No. 2008-E-0012, MTO Central Region" issued on March 22, 2010. Listed below are the available information/reports at the existing structure(s) over Oriole GO parking with a brief overview and scope of the work. The boreholes used for this report are tabulated in Section 3.2. It should be noted that some of the data are difficult to read because of the original scanned image quality of MTO GEOCRES information.

- ❖ **The Foundation Companies of Canada, Toronto Bypass Highway #401, Soil Conditions – C.N.R. & Leslie Street Overpass, C7142, September 30, 1953.**

The purposes of this study were to assess the embankment failure which took place during the construction of the west approach of the core lanes and to provide remedial measure recommendations for the proposed embankments. Nineteen (19) explorations were advanced for this study (Designated G-series, on Drawing 1) and Boreholes G9, G11, G12, G13, G14 and G15 were advanced close to the existing Oriole GO parking overpass structure.

❖ **Department of Highways Ontario, Foundation Investigation Report for Structures on Leslie Street & Highway 401, W.P. 252-61-3, July 2, 1964.**

The purpose of this study was to determine the depth to the underlying dense till layer in order to establish the lengths of piles to be used to support the proposed structures associated with the widening of the existing overpass. Eighteen (18) sampled boreholes and two dynamic cone penetration tests were performed (Designated B-series on Drawing 1). Boreholes B1 through B7, B11, B12, B13 and B15 were advanced close to the existing Oriole GO parking overpass structure.

❖ **Department of Highways Ontario, Foundation Section, Materials and Testing Division, Caisson Installation, Structure on Leslie Street & Highway 401 Interchange, W.P. 252-61-3, January 26, 1967.**

This memorandum provides a discussion of the subsurface conditions encountered by Dominion Soil Investigation Limited (Soil Investigation, Highway 401 and Leslie Street, 6-12-1, W.P. 266-61, February 9, 1967) in proximity to the caissons to be used to support the core lanes and recommendations for the installation of future caissons. Eighteen (18) borings were advanced (Designated 1A, etc. on Drawing 1). All 18 boreholes were advanced within the limit of the existing Oriole GO parking overpass structure.

Record of Borehole Sheets for all above mentioned boreholes which were advanced close to the existing Oriole GO parking overpass structure are included in Appendix A.

3.2 Compiled Subsurface Conditions Based on Previous Data

3.2.1 Background

In order to gain a better understanding of the subsurface conditions at the existing and newly proposed Highway 401 Oriole GO parking overpass structure, in between CN track(s) and Leslie Street, the data from the previously completed geotechnical studies were compiled and reviewed. To assist in the compilation of the data, the locations of the previously completed boreholes were transferred to a base plan (Drawing 1) and the major strata encountered by others were summarized in a tabular format (See Table in Appendix C). The stratigraphy was based on the borehole logs and descriptions provided in the various reports.

Please note that the geotechnical data used in this study were logged and prepared by a number of consultants and personnel, as such their descriptions and classifications varied somewhat. Therefore, some limited refinement of their interpretations was made when comparing the data as a whole.

Based on the findings of past investigation, estimated approximate subsurface profiles along the Westbound, Core and Eastbound Collector Lanes were prepared and these are presented in Drawing 2.

The following assumptions were made for this project:

- Elevations were assumed to be based on the geodetic datum.
- The surface elevation was based on those indicated on the borehole logs and no correction was made; existing topography was unavailable.
- Depths noted below, for the various units, were based on measurements below the existing ground surface, at the time the explorations were completed, no correction was made.

- Imperial elevations were directly converted to metric and no correction factor was used.
- Locations of boreholes were approximated based on those indicated on the drawings provided in the referenced reports.
- Due to the age of some of the documents and the quality of the original scanning, some of the data were difficult to read. Where borehole locations and/or data could not be accurately interpreted, these data were not plotted and/or not used.

The following provides a compiled overview of the subsurface conditions encountered at the surrounding area of the Highway 401 Oriole GO parking overpass structure, based on a summary of the existing data. The descriptions of the individual strata are provided to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site.

Note that the material boundaries indicated on the attached Drawing 2 are approximate and are based on data by others. 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 will vary across the Highway 401 and the Oriole GO parking area.

It should also be pointed out that the detail subsurface conditions described below may not be perfectly matched with the description in the Record of Borehole Sheets (see Appendix A) since it is considered more reasonable to present the typical subsurface conditions of the overall project site at the time of past investigations.

Table 3.2.1.1 contains a summary of the available boreholes at the Oriole GO parking overpass structure.

Table 3.2.1.1: Borehole Locations and Drilling Depths

Company / Year of Investigation	Borehole No.	Existing Ground Surface Elevation (m)	Bottom Elevation of Boreholes (m)	Piezometer
Dominion Soil Investigation Limited, 1967	1A	141.4	114.8	No
	2A	141.6	114.0	No
	3A	145.0	114.0	No
	4a	141.7	114.0	No
	5A	142.4	114.0	No
	6A	145.4	114.3	No
	7A	142.1	115.9	No
	8A	143.0	117.5	No
	9A	145.9	114.6	No
	10A	142.5	117.5	No
	11A	142.6	116.4	No
	12A	146.5	115.5	No
	13A	143.0	117.6	No
	14A	143.2	116.0	No
	15A	143.0	116.2	No

Company / Year of Investigation	Borehole No.	Existing Ground Surface Elevation (m)	Bottom Elevation of Boreholes (m)	Piezometer
Dominion Soil Investigation Limited, 1967	16A	144.7	111.7	No
	17A	143.8	116.7	No
	18A	147.3	115.8	No
Department of Highways Ontario, 1964	B1	141.4	115.2	No
	B2	140.5	115.8	No
	B3	140.1	114.0	No
	B4	140.5	115.8	No
	B5	140.5	116.0	No
	B6	139.9	115.4	No
	B7	139.6	114.9	No
	B11	142.0	117.3	No
	B12	140.2	115.5	No
	B13	139.9	113.8	No
	B15	143.3	115.5	No
The Foundation Companies of Canada, 1953	G9	140.0	118.5	No
	G11	138.2	120.3	No
	G12	140.0	112.9	No
	G13	140.0	118.8	No
	G14	139.7	119.8	No
	G15	132.2	118.7	No

The Table in Appendix C provides a summary of the primary stratigraphic contacts.

The following paragraphs present an overview of the subsurface conditions at the site based on the available information.

3.2.2 Topsoil/ Fill / Native Clayey Silt

Topsoil, fill and/or native clayey silt were generally encountered from the ground surface (at the time of investigation) at each of the boreholes (except for Boreholes 1A, 5A, B15 and G15) to depths ranging from approximately 0.1 m to 8.5 m below the ground surface, at the time of the explorations, or to elevations of approximately El. 142.4 m to 134.0 m. Note that the surface of the fill (See Drawing 2) was determined based on the surface elevation of a number of boreholes that was advanced between 1953 and 1990 (which were included in our assessment report entitled "Draft-Foundation Engineering Assessment Report, Highway 401 and Leslie Street Interchange, Toronto, Ontario, GWP 2130-01-00, Agreement No. 2008-E-0012, MTO Central Region", issued on March 22, 2010). Since those times, construction has taken place, which, in places, may have either resulted in the removal and/or addition of materials. As such, the accuracy of the surface topography and thicknesses based on recently surveyed elevations are considered very rough. The native clayey silt deposit appears to have been used as a portion of the fill material utilized during the construction of the interchange and as such these materials have been combined for purposes of this report.

The average thickness of the topsoil, fill and/or clayey silt material encountered was approximately 3.6 m, based on past explorations, extending to an average elevation of about El. 138.4 m.

The fill and/or clayey silt material at the site has been described as brown to grey sand, sand and silt, some to traces of gravel and clay, and clayey silt with fine gravel and sand. At Boreholes 6A and 14A, a dark grey silt with organic matter to black organic silty sand (0.9 m to 1.1 m thick) were encountered immediately below the fill.

The fill for the most part was described as a basically granular (i.e. non-cohesive) soil (mostly contacted in A-series boreholes, Dominion Soil Investigation Limited, 1967). The upper clayey silt soils were described as a cohesive soil (mostly contacted in B-series boreholes, Department of Highways Ontario, 1964).

Standard Penetration Tests (SPT) 'N'-values of 2 to in excess of 100 blows/0.3 m were recorded within the fill indicating a loose to very dense relative density in the basically granular fill. These values indicated that the fill, in some places, had not received a systematic compaction during its placement.

3.2.3 Surficial Granular Soils

Underlying the topsoil, fill and/or clayey silt materials and from the ground surface (Boreholes 1A, 5A, B15 and G15), a surficial granular soil, which ranges from sand, silty sand, silt and sand, sand and gravel and silt with fine sand, was encountered at approximate elevations of 143.3 m to 132.2 m. The thickness of the surficial granular soils encountered ranged between 0.9 m and 11.7 m, with an average thickness of approximately 5.9 m.

This stratum was typically described as brown to grey, sand, silty sand, silt and sand, sand and gravel and silt with fine sand. Based on the available data, this stratum can be considered a granular (i.e. non-cohesive) material.

SPT 'N'-values of 0 to in excess of 100 blows/0.3 m were recorded within these surficial granular soils, indicating a very loose to very dense relative density. Typically the stratum was described as compact.

3.2.4 Cohesive Soils

Grey stratified clay with layers of silt and gravel (The Foundation Companies, Canada Report, 1953) to silty clay to clayey silt, some to trace sand and gravel (Department of Highways Ontario, 1964 and Dominion Soil Investigation Limited, 1967) was encountered, typically below the surficial granular soils, at El. 136.9 m to 129.2 m (average El. 132.5 m). In some past investigations, the lower zone of this layer was recognized as clayey silt till (The Foundation Companies, Canada Report, 1953) to silty clay to clayey silt with more sand content (sandy, Dominion Soil Investigation Limited, 1967). This 2.8 m to 7.1 m thick lower cohesive soil (till to till-like material) was encountered at about El. 128.2 to 122.8 m. These clayey silt till and more sandy cohesive soil layer was not recognized as a separate unit in Department of Highways Ontario investigation (1964) and it is our opinion that this clayey silt till layer can be combined with and categorized under one cohesive soil unit with the upper clay with layers of silt and gravel deposit from foundation engineering viewpoint, based on the strength parameters measured by in-situ and laboratory tests. This cohesive deposit (combined upper cohesive soil zone and lower till like cohesive soil zone) was found to be approximately 9.9 m to 16.0 m thick (average 12.7 m).

Atterberg Limits tests performed on samples from the deposit indicated the following index values (See Tables B-1 and B-2 in Appendix B and individual borehole log sheets in Appendix A):

Liquid Limit: 16 - 50% (average 31% for upper clayey silt zone and 33% for lower clayey silt till zone)

Plastic Limit: 10-23% (average 17% for upper clayey silt zone and 16% for lower clayey silt till zone)

Plasticity Index: 6-27% (average 15% for upper clayey silt zone and 16% for lower clayey silt till zone)

The above values are characteristic of a clayey soil of low to medium plasticity. As indicated by the range of values, clayey silt and silt seams were also encountered.

Most of soil samples were obtained by pushing (probably thin walled-Shelby tube samplers) and few Standard Penetration Tests were performed within this deposit in the Foundation Companies, Canada investigation (1953). In the Department of Highways Ontario investigation (1964), sampling within this deposit was generally not carried out (except for Boreholes B5 – two split spoon samples and five thin walled tube samples, B6 – one split spoon sample, and B11 – one split spoon sample). In the Dominion Soil Investigation Limited investigation (1967), most of the soil samples were obtained by Standard Penetration Tests within this deposit. SPT 'N'-values of 2 to 49 blows/0.3 m were recorded in this deposit. It should be noted that average N-values recorded from the upper clay and lower clay (The Foundation Companies, Canada Report, 1953 and Dominion Soil Investigation Limited, 1967) were 6 and 13, respectively. Field vane tests were also carried out within this cohesive soil (Boreholes G14, G15 and B5) deposit resulting in undrained, in-situ shear strengths of 12 kPa to 60 kPa. Based on these test results and a tactile evaluation by others, the silty clay deposit was considered to have a consistency of soft to stiff.

Unconfined compression tests were also carried out in the laboratory on samples from this stratum resulting in unconfined compressive strength (UCS) of 15 kPa to 38 kPa. In general, the measured values were substantially lower than the vane results probably indicating disturbance of the collected samples.

Available in-situ and laboratory test results (Tables B-1, B-2 and Figure B-1) are included in Appendix B.

A 0.3 to 1.2 m thick, very loose to compact granular soil (sand to sand & gravel) was encountered at the interface of the clay with layers of silt with clayey silt till at about El. 128 m and 127 m in Boreholes G13, G14 and G15.

Due to their mode of deposition, the presence of cobbles and boulders can be anticipated in the lower zone of this stratum. It should be also noted that an about 9 inch (0.2 m) size boulder was encountered in Borehole 12A about 2.7 m below the interface of the surficial granular soil and this cohesive layer (thickness of the cohesive soil layer in Borehole 12A is about 11.4 m).

3.2.5 Glacial Till

Underlying the cohesive soil deposit, a glacial till was encountered in the boreholes at depths ranging from about 11.4 m to 28.0 m below the existing ground surface at the time of the explorations or at about elevations of El. 122.4 m to 118.1 m (Average El. 119.8 m). The till deposit was described as a heterogeneous mixture of clayey silt, sand and trace of gravel (Department of Highways Ontario, 1964), sandy till (The Foundation Companies, Canada Report, 1953) and granular to cohesive till with sand, silt seams, clay and embedded gravels (Dominion Soil Investigation Limited, 1967).

In two past investigations (Department of Highways Ontario, 1964 and The Foundation Companies, Canada Report, 1953), the till was generally classified as a basically granular (i.e. non-cohesive) soil. But it also exhibited some apparent cohesion, due to its clay content, especially where the clay content was relatively high. All boreholes were terminated within this till deposit after 0.5 m to 7.9 m penetration into the till deposit (average 3.8 m). No artesian condition or hole-caving was observed upon completion.

In the Dominion Soil Investigation Limited investigation (1967), this till deposit was described as granular, cohesive and mixture of both granular and cohesive till. Slight artesian condition and hole caving were recorded within the till deposit in Borehole 16A. A boulder was also encountered in Borehole 16A at about El. 116 m and borehole was further advanced by rock coring. This till layer was fully penetrated in most boreholes (except for Boreholes 15A and 17A) and thickness of the layer was about 1.4 m to 7.2 m (average 3.1 m). Results of grain size distribution tests done by Dominion Soil Investigation Limited are included in Appendix B (See Figures B-2 to B-6). Due to their mode of deposition, the presence of cobbles and boulders should always be anticipated in the till stratum.

SPT 'N'-values ranging from 13 to in excess of 100 blows/0.3 m were recorded within the till indicating a compact to very dense condition of granular till (typically very dense) and a hard consistency of basically cohesive till.

3.2.6 Lower Sand

Below the till in all boreholes advanced by Dominion Soil Investigation Limited, 1967 except for Boreholes 15A and 17A), a lower sand deposit was encountered at depths ranging from about 24 to 32 m (Average 27 m) below the existing ground surface at the time of the explorations or at elevations of 118.5 m to 112.5 m (Average El. 116.4 m). This lower sand was not fully explored and its lateral and vertical extent are unknown (i.e. deeper boreholes were all terminated within the lower sand after 0.2 m to 2.8 m penetration into the layer).

The sand deposit was described as a fine to coarse sand with boulders, trace of gravel, clay and silt. Results of past grain size distribution tests are included in Appendix B (See Figure B-7).

The sand was classified as a basically granular (i.e. non-cohesive) soil.

Within this deposit, slight artesian conditions were observed at or below El. 118 m to 112 m (average 116 m).

SPT 'N'-values were generally in excess of 100 blows/0.3 m indicating a very dense condition.

3.2.7 Groundwater Conditions

Groundwater levels were reportedly observed in the open boreholes while drilling and upon completion of each borehole. The final recorded values in the open boreholes, as given on the borehole log sheets, are summarized in the table below.

Table 3.2.7.1: Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Water Level Measurement Depth (m)*	Water Level Measurement Elevation (m)*	Remarks
1A	141.4	-	-	No Piezometer
2A	141.6	-	-	No Piezometer

Borehole	Ground Surface Elevation (m)	Water Level Measurement Depth (m)*	Water Level Measurement Elevation (m)*	Remarks
3A	145.0	-	-	No Piezometer
4a	141.7	-	-	No Piezometer
5A	142.4	-	-	No Piezometer
6A	145.4	-	-	No Piezometer
7A	142.1	-	-	No Piezometer
8A	143.0	-	-	No Piezometer
9A	145.9	-	-	No Piezometer
10A	142.5	-	-	No Piezometer
11A	142.6	-	-	No Piezometer
12A	146.5	-	-	No Piezometer
13A	143.0	-	-	No Piezometer
14A	143.2	-	-	No Piezometer
15A	143.0	-	-	No Piezometer
16A	144.7	-	-	No Piezometer
17A	143.8	-	-	No Piezometer
18A	147.3	-	-	No Piezometer
B1	141.4	3.6	137.8	No Piezometer
B2	140.5	4.3	136.2	No Piezometer
B3	140.1	4.7	135.4	No Piezometer
B4	140.5	3.4	137.1	No Piezometer
B5	140.5	2.8	137.7	No Piezometer
B6	139.9	4.3	135.6	No Piezometer
B7	139.6	2.8	136.8	No Piezometer
B11	142.0	1.9	140.1	No Piezometer
B12	140.2	3.8	136.4	No Piezometer
B13	139.9	4.3	135.6	No Piezometer
B15	143.3	12.4	130.9	No Piezometer
G9	140.0	1.9	138.1	No Piezometer
G11	138.2	0.3	137.9	No Piezometer
G12	140.0	1.6	138.4	No Piezometer
G13	140.0	-	-	No Piezometer
G14	139.7	2.4	137.3	No Piezometer
G15	132.2	0.0	132.2	No Piezometer

* may not be stabilized

As can be seen from the table, the observed groundwater levels ranged between elevations of approximately 131 m and 140 m. In addition to the observed groundwater condition, a perched water condition could possibly be encountered at the site due to the accumulation of the surface water in the fill materials and in the underlying surficial granular soils, overlying the less permeable cohesive soil deposit, especially during rainy periods/spring thaw.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The groundwater in the upper units may also be controlled by the water level in the watercourse located about 200 m east of the Leslie Street, known to be a branch of East Don River.

3.3 Recent Investigation Procedures

The fieldwork for the previously proposed rehabilitation of three existing highway ramps at the Leslie Street interchange was performed during the period of November, 2009 through January, 2010. Of these boreholes five are located close to the Oriole GO parking overpass structure, as follows:

Table 3.3.1: Borehole Locations and Drilling Depths

Borehole No.	Location (Coordinates)		Existing Ground Surface Elevation (m)	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
	Northing	Easting			
Highway 401 Overpass at Leslie Street/CNR Ramp N-E (# 37-206/6)					
E1	315808.7	4847320.9	143.2	29.4	Yes
E2	315835.3	4847329.4	140.4	28.8	No
Highway 401 Overpass at Leslie Street/CNR Ramp N-W (# 37-206/7)					
N1	315689.2	4847431.6	146.90	30.6	No
N2	315670.7	4847425.8	142.30	27.6	Yes
N3	315653.1	4847420.9	142.40	27.9	No

Eastern Soil Investigation of Courtice, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer (Mr. Raid Khamis, P.Eng.) from Coffey. The boreholes were put down using a track mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. The boreholes were advanced using continuous flight hollow-stem augers.

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

Where the consistency permitted, in-situ shear vane tests were conducted within the cohesive soils to measure the undrained, in-situ shear strength of the soil. The field vane shear tests were carried out with an MTO 'N' vane.

Dynamic Cone Penetration Tests (DCPT) were performed from the bottom of the boreholes (Boreholes E1 and E2) and from the ground surface (Borehole N3). In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly, into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects, which in some cases affect the SPT results.

Groundwater conditions in the boreholes were observed during drilling and upon their completion in the open boreholes. In addition, a piezometer was installed in each of Boreholes E1 and N2 to enable groundwater level monitoring over a prolonged period of time without interference from surface water. The

remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

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

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content, grain size analyses, Atterberg Limits tests and oedometer test, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in **Appendix B**.

3.4 Subsurface Conditions Encountered During Recent Investigation

As mentioned before, five (5) boreholes (Boreholes E1, E2, N1, N2 and N3) were advanced adjacent to the existing Oriole GO parking overpass structure. Details of the new structure(s) were not available at the time of preparing this report.

The plan locations of these boreholes are presented on Drawing No. 1. Details of subsurface conditions encountered at each borehole location for the investigation, including results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are enclosed in Appendix B.

In general, the sub-surface stratigraphy in the five boreholes comprises fill materials and surficial granular soil deposits overlying typically firm to stiff silty clay, which is, in turn, underlain by cohesive and granular glacial till deposits. All boreholes were terminated within the lower granular soils deposits after 1.0 to 3.8 m penetration into the lower granular soils deposits.

Details of the sub-surface conditions encountered in Boreholes E1, E2, N1, N2 and N3 are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to amplify and complement these data.

3.4.1 Topsoil

Borehole N2 contacted a 1 cm thick topsoil layer at the ground surface. About 0.1 m snow was covering the ground surface at Borehole E1 location at the time of investigation.

3.4.2 Asphalt and Concrete

Borehole N1 contacted a 230 mm thick asphalt layer underlain by 200 mm sand fill which is in turn underlain by a 200 mm thick concrete layer (or concrete pieces). Borehole N2 contacted concrete and asphalt pieces below the topsoil.

3.4.3 Fill

All boreholes contacted fill extending to depths of 2.5 to 10.8 m below the ground surface or to El. 139.8 to 135.3 m.

The fill typically consists of fine grained granular material such as silty sand, silty sand to sandy silt, sand and gravelly sand. In Borehole N3, the top 1.4 m of the fill was found to consist of coarser grained (gravelly sand) material. The fill also contains zones of cohesive soil, especially in Boreholes N1 and N3. The fill also contains traces of some clay, gravel and cobbles. Some rootlets and organics have been encountered at the bottom portion of the fill in Boreholes N1 and N3 and trace of organics was encountered in the middle portion of the fill in Borehole E1.

The grain-size distribution of six samples from the fill is given in Figure B-8, in Appendix B, with the following grain-size distribution: 1-28% gravel, 40-80% sand and 17-49% silt and clay size particles.

The grain-size distribution of a sample from the cohesive zones within the fill is presented in Figure B-9, in Appendix B. The sample displayed the following grain-size distribution: 4% gravel, 32% sand, 39% silt and 25% clay size particles.

An Atterberg limits test performed on a sample from the cohesive zone in Borehole N3 is given in Figure B-10 in Appendix B. The test yielded the following index values:

Liquid Limit:	18%
Plastic Limit:	13%
Plasticity Index:	5

The measured index values are representative of cohesive soils of low plasticity and the measured moisture content is below their measured plastic limit value.

Standard penetration tests performed on the basically granular fill yielded N-values of 4 to 58 blows/0.3 m (in excess of 100 blows/0.3 m in Borehole N1 may be due to a buried concrete piece). These results indicate that the relative density of the granular fill can be described as very loose to very dense but typically compact to dense, while the general consistency of cohesive zone of the fill in Borehole N3 can be described as stiff. It should be noted that some higher N values may have been influenced by the observed gravel and cobble size particles within the fill. These results indicate that, at most locations, the fill has received a reasonable degree of compaction when it was first placed, but there are zones where little or no compaction was applied (e.g. Borehole E2, 1.4 m to 2.9 m depth).

3.4.4 Surficial Granular Soils

Underlying the fill, all boreholes contacted surficial granular (non-cohesive) soils consisting of sand, silty sand to sandy silt and silt.

3.4.4.1 Sand

Below the fill, Boreholes E1, N1, N2 and N3 contacted a 1.1 to 2.6 m thick surficial sand deposit at depths of 2.5 m to 10.8 m below the ground or at El. 139.8 m to 136.1 m. The sand deposit was found to extend to depths of 5.1 m to 11.9 m below the ground surface or to El. 137.3 m to 135.0 m and contains traces of some silt, some gravel. Traces of organics were also encountered in Borehole N1, at the interface with the overlying fill.

The grain size distribution of three samples from the sand deposit was determined in laboratory which showed 0% gravel, 76-87% sand and 13-24% silt and clay size particles (see Figure B-11 in Appendix B). As can be seen from the grain-size distribution curves presented, the material has a uniform grain size distribution, primarily within the fine sand range, and is considered to have a mass coefficient of permeability (k) of the order of 5×10^{-3} cm/s. The sand is a granular (non-cohesive) soil type.

Standard Penetration tests performed in the deposit yielded N-values of 18 to 49 blows/0.3 m, indicating a compact to dense relative density.

3.4.4.2 Silty Sand to Sandy Silt

Underlying the sand in Borehole E1, a relatively finer granular soil (i.e. silty sand to sandy silt) was encountered at a depth of 5.9 m or El. 137.3 m and was found to extend to a depth of 8.3 m or El. 134.9 m. The silty sand to sandy silt deposit contains some clay in the lower portion of this deposit.

The grain size distribution of a sample from the silty sand to sandy silt deposit was determined in laboratory which showed 0% gravel, 41% sand and 48% silt and 11% clay size particles (see Figure B-12 in Appendix B). Based on the grain size distribution curves, the deposit is considered to be somewhat less pervious than the sand materials discussed in Section 3.4.4.1. N-values recorded range from 19 to 22 blows/0.3 m indicating a compact condition.

3.4.4.3 Silt

Below the fill in Borehole E2 and the sand in Boreholes N1, N2 and N3, a 0.9 m to 3.2 m thick silt deposit was contacted at depths of 5.1 m to 11.9 m or El. 137.3 to 135.0 m. The silt deposit contains some sand and traces to some clay size particles and exhibits a dilatant behaviour in the presence of water. It is considered a fine grained granular (i.e. non-cohesive) soil type.

Four grain size analyses were carried out on representative samples of the silt deposit. The results are presented in Figure B-13 in Appendix B, as well as in the Record of Borehole Sheets in Appendix A. The results show the following grain-size distribution:

Gravel:	0%
Sand:	7-19%
Silt:	65-88%
Clay:	5-16%

Based on these results, the deposit is considered to be somewhat finer and less pervious in comparison with the silty sand to sandy silt deposit.

An Atterberg limits test was performed on a sample from the deposit recovered from Borehole N1 and it was found to be non-plastic (i.e. plastic limit could not be determined).

Standard penetration test conducted in this surficial silt deposit gave N-values ranging from 3 to 40 blows/0.3 m which indicate very loose to dense relative density.

3.4.5 Silty Clay

All boreholes contacted a 12.2 m to 13.1 m thick silty clay deposit below the surficial granular soils, at depths ranging from 7.8 m to 12.8 m or at El. 134.9 m to 132.2 m. The deposit was found to extend to depths of 20.5 m to 25.1 m below the ground surface or to El. 122 m to 119.9 m. This cohesive soil deposit frequently has a varved like or stratified structure and contains thin seams of silty sand to sandy silt and sand. The presence of traces to some sand and gravel is also noted within the deposit.

The grain-size distribution of seven samples from the silty clay deposit is given in Figure B-14 in Appendix B. This indicates the following grain-size distribution:

Gravel:	0 – 3%
Sand:	1 – 24%
Silt:	33 – 58%
Clay:	29 – 59%

The results of Atterberg limits tests, performed on eleven samples from the deposit, are presented in Figure B-15 in Appendix B. The test results yielded the following index values:

Liquid Limit:	26 – 48%
Plastic Limit:	14 – 23%
Plasticity Index:	12 – 25
Natural Moisture Content:	16 – 44%

These results are characteristic of clayey soils of low to medium plasticity and the fact that measured natural moisture contents are generally in between the measured liquid limit and plastic limit values indicates the likelihood of a slightly over-consolidated soil deposit.

Two oedometer (one-dimensional consolidation) tests were performed in the laboratory on thin-walled tube (TW) samples (one from Borehole E1 and another from Borehole N3). The results are presented in Figures B-16 and B-17 in Appendix B. These show a possible pre-consolidation pressure similar to the existing overburden pressure (i.e. normally consolidated). However, it is our opinion that the samples may be somewhat disturbed during their retrieval process and as such it is likely that the deposit is slightly over-consolidated. The tests indicate a compression index C_c of about 0.28 and rebound value C_r of 0.06-0.09. It should be pointed out that the presence of silty sand to sandy silt and clayey silt in the deposit may have affected consolidation test results because above mentioned soils are typically less compressible than silty clay itself. A coefficient of consolidation C_v of about $1 \times 10^{-3} \text{ cm}^2/\text{s}$ is indicated from the oedometer test results.

The measured bulk unit weight of the two oedometer samples range from 17.7 kN/m^3 to 18.4 kN/m^3 .

Standard penetration test conducted in the silty clay deposit gave N-values which range from 2 to 10 blows/0.3 m. Undrained in-situ shear strengths, as measured by MTO "N" type field vane, varied from 20 to in excess of 100 kPa indicating a soft to very stiff consistency. The variation of the undrained shear

strength as measured by field vane tests with elevation is given in Figure B-18. It should however be pointed out that some of the higher undrained shear strengths measured may have been obtained from the silty sand to sandy silt and clayey silt interbeds within the silty clay deposit.

3.4.6 Glacial Till

Underlying the silty clay, all boreholes contacted a glacial deposit consisting of a heterogeneous mixture of silty sand to sandy silt and clayey silt. Trace to some gravel was encountered within the glacial till deposit. As well, the presence of boulders can be expected in glacial till deposits, owing to their mode of deposition. Auger grinding was observed during the investigation at various depths within the till deposit. As mentioned, the composition of the deposit was found to range primarily from a basically cohesive deposit consisting of clayey silt till to sandy silt-silty sand till, as discussed in Sections 3.4.6.1 and 3.4.6.2, below.

3.4.6.1 Clayey Silt Till

Boreholes E1, E2, N1, and N2 contacted a glacial deposit consisting of clayey silt till underlying the silty clay at depths of 20.5 and 25.1 m or at El. 122.0 and 119.9 m. Thickness of the clayey silt till was found to be 3.0 to 5.6 m.

The grain-size distribution of five samples from the clayey silt till deposit is presented in Figure B-19 in Appendix B. This indicates the following grain-size distribution:

Gravel:	0 – 3%
Sand:	7 - 36%
Silt:	40 – 80%
Clay:	20 – 36%

The results of Atterberg limits tests, performed on four samples from the clayey silt till, are given in Figure B-20 in Appendix B. The tests yielded the following index;

Liquid Limit:	16 – 26%
Plastic Limit:	11 – 15%
Plasticity Index:	5 – 11
Natural Moisture Content:	9 – 14 %

These results are characteristic of clayey soils of low plasticity. The fact that the measured natural moisture contents are generally near or below the measured plastic limits indicates that the deposit is probably over-consolidated. The clayey silt till is a cohesive material.

Standard penetration test N-values of 9 to in excess of 100 blows/0.3 m were recorded, showing a wide variation. Field vane tests were performed and yielded undrained shear strengths of 64 kPa to in excess of 100 kPa. Based on these field test results, the clayey silt till can be described as having a stiff to hard consistency. The relatively weaker zones (i.e. stiff) were recorded in the upper zone (i.e. immediately below the massive silty clay deposit).

3.4.6.2 Sandy Silt to Silty Sand Till

Underlying the clayey silt till in Borehole N2 and the silty clay in Borehole N3, a relatively coarser (i.e. basically granular) sandy silt to silty sand till was contacted, at depths of 20.9 m and 23.5 m below the ground surface or at or El. 121.5 m and 118.8 m. Thickness of the silty sand to sandy silt till at the borehole locations was found to be 3.1 m to 5.7 m.

The grain-size distribution of three samples from the sandy silt to silty sand till deposit is given in Figure B-21. This indicates the following grain-size distribution;

Gravel:	10 – 23%
Sand:	38 - 59%
Silt & Clay:	31 - 51% (average clay content 16%)

The deposit can be described as a granular (non-cohesive) material.

Standard penetration test performed in this basically granular (i.e. non-cohesive) till yielded N-values typically in excess of 100 blows/0.3 m. An N-value of 20 blows/0.3 m was recorded within the upper 1 m of the deposit in Borehole N3. Based on the test results, the relative density of the glacial till can be generally described as very dense with a compact zone near the top in Borehole N3.

3.4.7 Basal Granular Soils

Underlying the glacial till, the boreholes contacted basal granular soils, varying in composition from sand to gravelly sand as described in Sections 3.4.7.1 and 3.4.7.2, below.

3.4.7.1 Sand

Below the clayey silt till, Boreholes E1, E2 and N1 contacted a basal sand deposit at depths of 25.0 m to 28.1 m or El. 118.8 m to 115.4 m. Boreholes E1, E2 and N1 were terminated within the basal sand at depths of 28.8 m to 30.6 m or El. 116.3 m to 111.6 m. This deposit contains traces to some gravel and silt. While drilling in the this deposit, an about 0.3 m soil back up was observed in Borehole E1 at El. 115.5 m and 114.0 m, while about 3.7 m and 2.5 m back up were observed at El. 114.5 m and 111.6 m in Borehole E2. Dynamic cone penetration tests (DCPT) were also performed from the bottom of Boreholes E1 and E2 where soil back ups were observed, until refusal to the dynamic cone penetration test was encountered.

The grain-size distribution of three samples from the basal sand was determined in the laboratory which showed 1-14% gravel, 68-90% sand and 7-18% silt and clay size particles (see Figure B-22).

This is a granular (non-cohesive) soil deposit.

Standard penetration tests yielded N-values of 67 to in excess of 100 blows/0.3 m, except in the upper zones of the deposit in Borehole E2, where an N-value of 12 blows/0.3 m was recorded. Based on these test results, the relative density of the basal sand can be typically described as very dense, with a compact upper zone in Borehole E2.

3.4.7.2 Gravelly Sand

Boreholes N2 and N3 encountered a gravelly sand deposit underlying the silty sand till at a depth of 26.6 m or at El. 115.7 m and 115.8 m, respectively.

Boreholes N2 and N3 were terminated after 1.0 m to 1.3 m penetration into the gravelly sand deposit. A grain-size analysis performed on a sample from the gravelly sand and the analysis showed 29% gravel, 65% sand and 6% silt and clay particles (see Figure B-23). The material is considered to be a relatively coarse grained granular soil.

Standard penetration test N-values of in excess of 100 blows/0.3 m were recorded and based on these test results, the relative density of the gravelly sand can be described as very dense.

3.4.8 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. The observations made in the boreholes are shown on the individual Record of Borehole Sheets in Appendix A, and are summarized in the following table.

Table 3.4.8.1: Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Remark
E1	143.2	29.0/114.2	7.1/136.1	31 days after completion	Water level in the piezometer measured at 20.7 m one day after completion and gradually rose to 7.1 m/El. 136.1 m after 31 days. Soil back-up noted in the basal sand while drilling.
E2	140.4	-	19.8/120.6*	Upon completion	Caved-in @ 8.5 m ** Excess soil back-up noted in the basal sand at El. 115 m while drilling
N1	146.9	-	17.7/129.2*	Upon completion	Caved-in** @ 15.2 m
N2	142.3	26.8/115.5	6.4/135.9	80 days after completion	Water level in the peizometer measured at 24.4 m two days after completion and gradually rose to 6.4 m / El. 135.9 m after 80 days of completion.
N3	142.4	-	16.8/125.6*	Upon completion	Caved-in** @ 4.3 m

* not stabilized (groundwater table measured before hollow stem auger pull out)

** Caved-in depth measured after the hollow stem augers pull out

Based on these observations made while drilling, an excess upward hydrostatic gradient prevails below the practically impervious silty clay and clayey silt till deposits. From the recorded groundwater levels in the piezometers and the observations made while drilling the excess hydrostatic pressure attains a more aggressive nature in the basal granular soils underlying the glacial tills.

In addition to the observed groundwater conditions, a perched water condition could possibly be encountered at the site due to the accumulation of the surface water in the fill materials and in the underlying surficial granular soils, overlying the practically impervious silty clay deposit, especially during rainy periods/spring thaw.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The groundwater in the upper units may also be controlled by the water level in the watercourse located some 0.2 km east of the Leslie Street, known to be a branch of the East Don River.

For and on behalf of Coffey Geotechnics Inc.


Gwangha Roh, Ph.D.

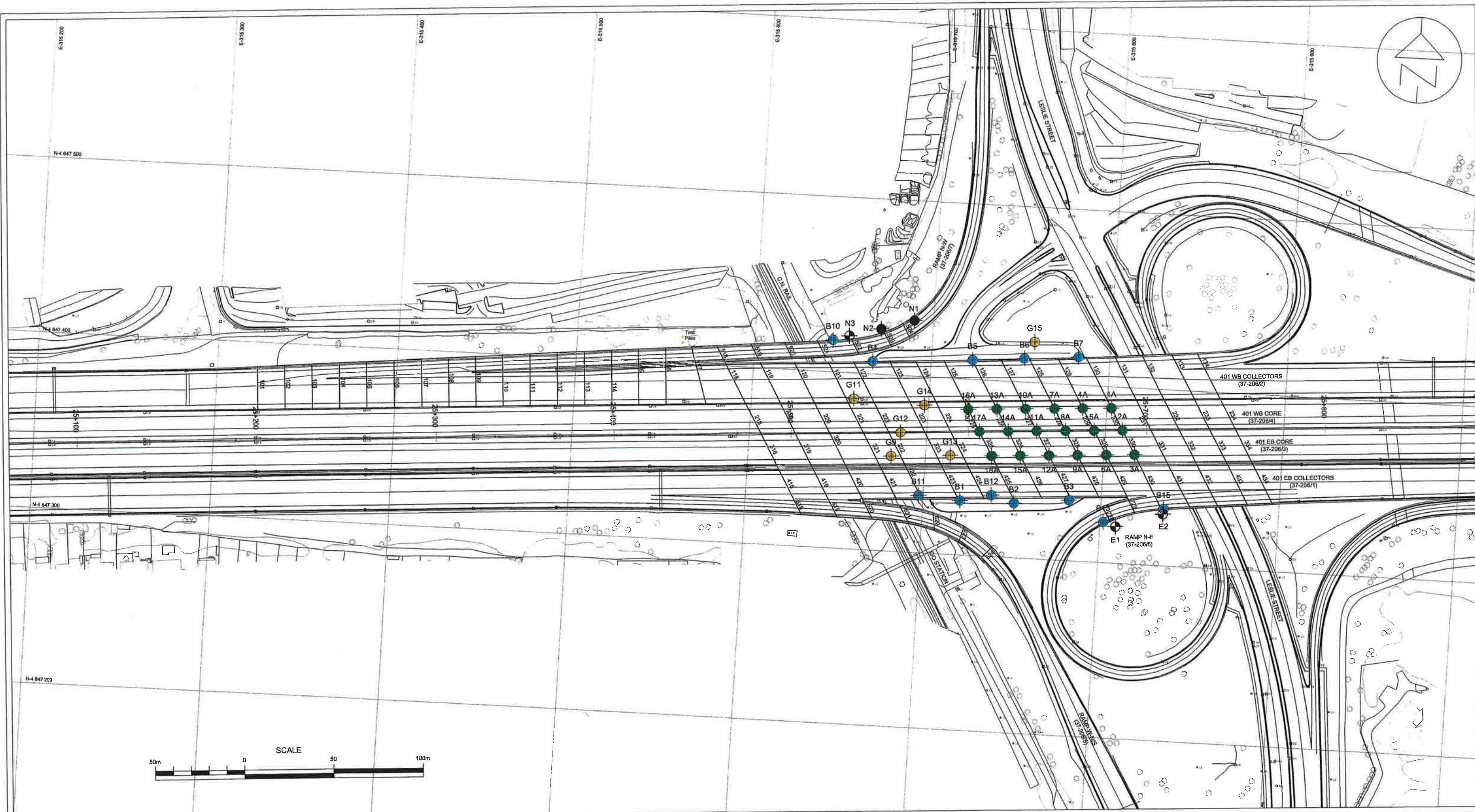

Ramon Miranda, P.Eng.




Zuhtu Ozden, P.Eng.



Drawings



LEGEND

- Borehole (Coffey, 2009)
- Borehole + DCPT (Coffey, 2009)
- Borehole (Department of Highways Ontario, 1964)
- Borehole (The Foundation Company of Canada/Geocon, 1953)
- Borehole (Dominion Soil Investigation Limited/Department of Highways Ontario, 1967)
- Bent Number (TYP)

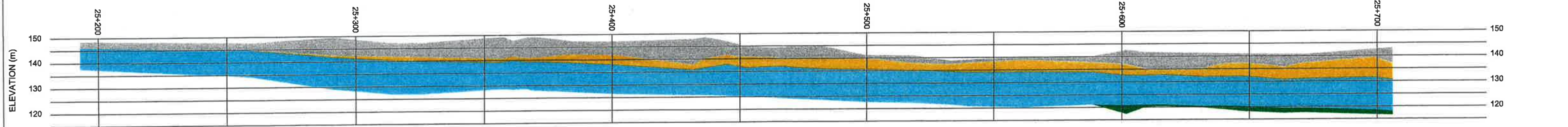
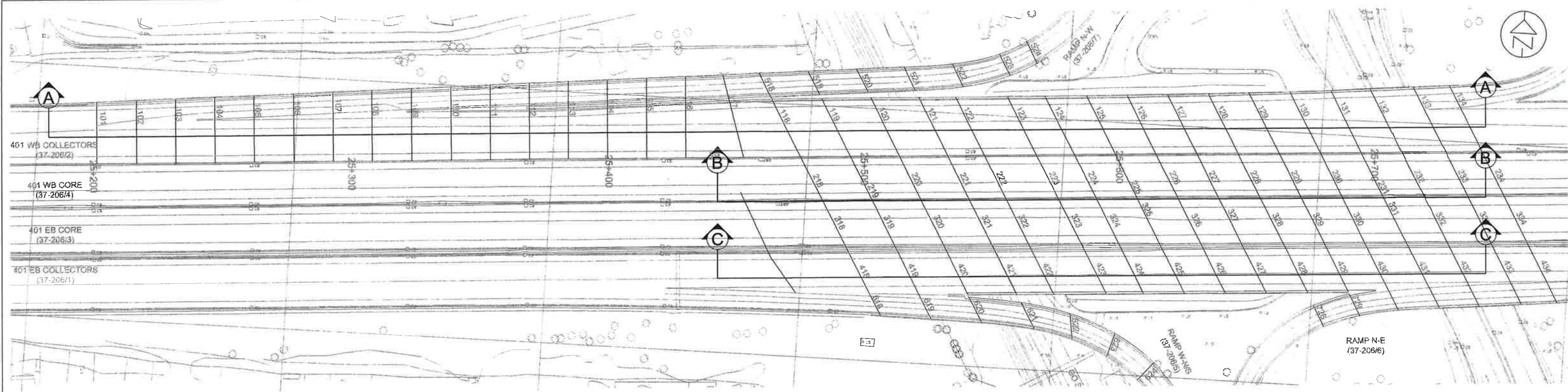
NOTES

1. This drawing forms part of the report (project number as referenced) and should only be used in conjunction with this report.
2. Base plan provided by Delcan.

drawn	SH
approved	RDP
date	June 16, 2011
scale	As Shown
original size	Tabloid

coffey  **geotechnics**
SPECIALISTS MANAGING THE EARTH

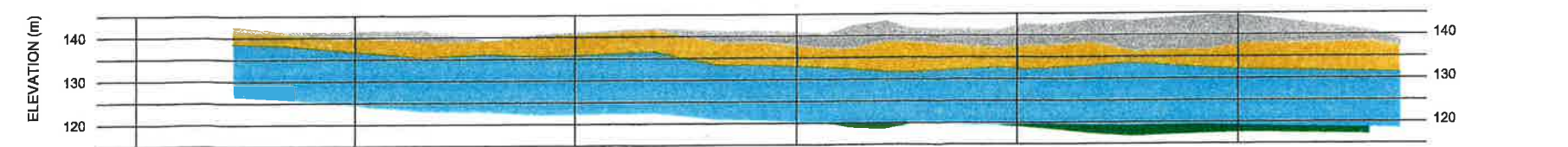
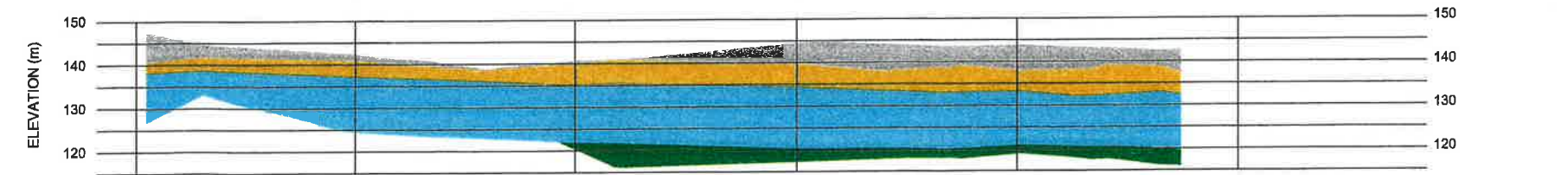
client:	DELCAN CORPORATION	
project:	FOUNDATION ENGINEERING ASSESSMENT HIGHWAY 401 AND LESLIE STREET INTERCHANGE ORIOLE GO PARKING OVERPASS STRUCTURE TORONTO, ONTARIO	
title:	SITE AND BOREHOLE LOCATION PLAN	
project no:	TRANETOB01245AA-AC	drawing no: 1



PROFILE A-A
WEST BOUND COLLECTORS
HORIZONTAL SCALE
25m 0 25 50m

PROFILE B-B
CENTRELINE OF CORE
HORIZONTAL SCALE
25m 0 25 50m

PROFILE C-C
EAST BOUND COLLECTORS
HORIZONTAL SCALE
25m 0 25 50m



LEGEND	
	Fill / Clayey Silt
	Silty Sand
	Silty Clay
	Glacial Till

NOTES	
1.	The topography has been interpreted from historic borehole data completed at the site by others. Between boreholes the topography was assumed from geological evidence. The topography between boreholes may vary from that shown.
2.	For strata details see borehole logs appended to this report.
3.	This drawing forms part of the report (project number as referenced) and should only be used in conjunction with this report.
4.	Base plan provided by Delcan.
5.	Dimensions are in metres unless otherwise noted.

drawn	SH
approved	RDP
date	June 16, 2011
scale	As Shown
original size	Tabloid



client:	DELCAN CORPORATION	
project:	FOUNDATION ENGINEERING ASSESSMENT HIGHWAY 401 AND LESLIE STREET INTERCHANGE ORIOLE GO PARKING OVERPASS STRUCTURE TORONTO, ONTARIO	
title:	ESTIMATED SUBSURFACE PROFILES	
project no:	TRANETO01245AA-AC	drawing no: 2

Appendix A

Record of Borehole Sheets

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . (1A)

OUR REFERENCE NO. 6-12-1
 Your Ref No. W.P. 266-61
 CLIENT D.H.O.
 PROJECT HWY NO 401 & LESLIE ST
 LOCATION BETWEEN CAISSONS 230-1 & 230-2
 DATUM ELEVATION G. S. C.

METHOD OF BORING: WASHBORING
 DIAMETER OF BOREHOLE: 2 3/8"
 DATE: JAN. 3-5, 1967

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				SHEAR STRENGTH lb/sq ft	CONSISTENCY water content % FL W LI	REMARKS
				NUMBER	TYPE	Advance ment of Sampler	2.0	4.0	6.0	8.0			
453.9	0	GROUND SURFACE											
450	10	Compact to Dense Brown SILTY FINE SAND		1	S.S.	32							
440	20												
433.9	28.0	Soft to Firm Grey CLAYEY SILT to SILTY CLAY with some sand		2	S.S.	4							
430	30												
420	40			3	S.S.	4							
410	50												
400	60	Sandy below el. 410 ± ft		4	S.S.	11							
				5	W.S.	—							
392.9	71.0	Very Dense, Grey SANDY SILT with a trace of clay and embedded gravel. (slightly cemented) to SILT with a trace to some clay. (GLACIAL TILL.)		6	S.S.	13							
390				7	S.S.	6.9							
				8	S.S.	175/11							
385.4	78.5			9	S.S.	78							
	80			10	S.S.	110							
380	83.9	Very Dense, Grey FINE to MED SAND with a trace of fine grav.		11	S.S.	164/8							
				12	S.S.	100/6							
87.4	90	END OF BOREHOLE		13	W.S.	—							
				14	S.S.	300/6							
370													

SLIGHT ARTESIAN PRESSURE BELOW EL. 379 ft.

VERTICAL SCALE: 1 IN. TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: D.A.M. CH'D

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 2. (2A)

OUR REFERENCE NO 6-12-1
 Your Ref. No W.P. 266-61
 CLIENT D.H.O.
 PROJECT HWY NB 401 & LESLIE ST.
 LOCATION BETWEEN CAISSONS 230-3 & 330-1
 DATUM ELEVATION G.S.C.

METHOD OF BORING WASH BORING
 DIAMETER OF BOREHOLE 2 3/8
 DATE JAN 5-6, 1967

ENCLOSURE NO

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		SHEAR STRENGTH lbs/sq ft	CONSISTENCY water content % FL W LI	REMARKS
				NUMBER	TYPE	NO. & ADVANCEMENT OF SAMPLE	2.0	4.0			
454.7	0	GROUND SURFACE									
460		Brown SAND and SILT with some gravel, trace of clay and organic matter. (FILL)									
450	10	FINE SAND		1	S.S.	89					
440	20	brwn. grey									
432.2	32.5			2	S.S.	14					
430		FIRM to STIFF Gray SILTY CLAY									
420	40			3	S.S.	5					
410	50	Sandy below EL. 410± ft.									
400	60			4	S.S.	10					
390	74.5	VERY DENSE, Grey SANDY SILT with embedded gravel.		5	S.S.	83					
80		SILT with a trace of clay and gravel. (TILL)		6	S.S.	143					
380.7	84.0	HARD, Grey CLAYEY SILT with embedded gravel (TILL)		7	S.S.	100.3					
380				8	S.S.	100.4					
376.7	88.0	VERY DENSE Medium SAND		9A	W.S.	—					
90				10	S.S.	100.4					
90.8		END OF BOREHOLE		11	W.S.	—					
370	100			12	S.S.	148.5					

SLIGHT ARTESIAN PRESSURE BELOW EL. 376 ft

VERTICAL SCALE 1 IN 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CHD

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 3. (3A)

OUR REFERENCE NO 6-12-1
 Your Ref No WP 266-61
 CLIENT C. H. O.
 PROJECT I.W.Y. No 401 & LESLIE ST
 LOCATION BETWEEN CAISSONS 330-2 & 330-3
 DATUM ELEVATION G.S.C.

METHOD OF BORING AUGERING & WASHBORING
 DIAMETER OF BOREHOLE 3" & 2 3/8"
 DATE DEC. 19-23, 1966 ENCLOSURE NO

ELEVATION h	DEPTH h	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	NO. of Advances of Sampler	2.0	4.0	6.0	8.0	10.0	PL	W	LI	
475.6	0	GROUND SURFACE													
470		Brown SAND and SILT with some clay FILL													
460	14.0	DENSE, Greenish Brown SILTY FINE SAND with organic matter		1	S.S.	42									
450	18.5	VERY DENSE to COMPACT SILTY FINE SAND		2	S.S.	106									
440	30	Brown Grey (wet below El. 443 ft.)		3	S.S.	12									
430	43.5	FIRM to STIFF Grey SILTY CLAY with a trace of fine gravel and occasional fine sand seams.		4	S.S.	6									
420	50			5	S.S.	5									
410	60			5A	W.S.	—									
400	70			6	S.S.	7									
390	80			7	S.S.	7									
380	85.6	DENSE Fine to Coarse SAND		8	S.S.	13									
370	90	VERY DENSE, Grey SILT, trace of clay (slight to no cohesion and plasticity)		8A	W.S.	—									
360	94.0	HARD CLAYEY SILT (GLACIAL TILL)		9	S.S.	29									
350	98.5	VERY DENSE Medium to Coarse SAND		10	S.S.	72									
340	101.5	END OF BOREHOLE		11	S.S.	77/6"									
				12	S.S.	10/9"									
				13	S.S.	84									
				14	S.S.	142									
				15	S.S.	137/10"									
				16	S.S.	130									
				17	S.S.										
				18	S.S.	80/4"									

SLIGHT ARTESIAN
PRESSURE OBSERVED
BELOW EL. 379.1 ft.

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: O. A. M. CND.

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 4 . . (4A)

OUR REFERENCE NO. 6 - 12 - 1
 YOUR REF. NO. W.P. 256 - 81
 CLIENT D. H. O.
 PROJECT HWY. NO. 401 & LESLIE ST.
 LOCATION BETWEEN CAISSONS 229-1 & 229-2
 DATUM ELEVATION G. S. C.

METHOD OF BORING WASHBORING
 DIAMETER OF BOREHOLE 2 3/8"
 DATE JAN. 3 - 4, 1967

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	WATER CONTENT of Sample	20	40	60	80	100	PI	W	LI	
464.8	0	GROUND SURFACE													
460		COMPACT to DENSE Brown SANDY SILT with a trace of clay (Possibly Fill)													
452.8	12.0	to													
450		Brown													
	20	FINE SAND with some silt		2	S.S.	35									
440				3	S.S.	24									
433.8	31.0														
430		FIRM to STIFF Grey													
	40	SILTY CLAY		4	S.S.	6									
420		occasional sand seams													
	50														
410				5A	S.S.	14									
	56.0	Sandy below at 408 ft.													
400				6A	S.S.	41									
	70														
392.3	72.5	VERY DENSE, Grey SANDY SILT with a trace of clay and embedded gravel. (slightly cemented)		7	S.S.	143									
390				8	S.S.	106/6									
	80	to		8A	W.S.										
				9	S.S.	74									
393.8	81.0	SILT with some fine sand and trace of clay		10	S.S.	55									
				11	S.S.	85									
380		VERY DENSE, Grey		12	S.S.	154									
379.3	85.5	FINE to MEDIUM SAND		12A	W.S.										
	90			13	S.S.	50/11									
	90.6	END OF BOREHOLE													
370															
	100														

SLIGHT ARTESIAN
PRESSURE BELOW
EL. 378 ft.

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CHD.

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 5. (5A)

OUR REFERENCE NO. 6-12-1
 Your Ref. No. W.P. 266-51
 CLIENT D. M. O.
 PROJECT HWY. NO. 401 & LESLIE ST.
 LOCATION BETWEEN CAISSONS 229-3 & 329-1
 DATUM ELEVATION G S C

METHOD OF BORING: WASHBORING
 DIAMETER OF BOREHOLE: 2 3/8"
 DATE: JAN 5-6, 1967

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES		PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	2.0	4.0	6.0	8.0	10.0	PL	WL	LI	
467.3	0	GROUND SURFACE												
460	10	COMPACT to DENSE Brown SAND with some silt and gravel		1	S.S.	60								
450	20	SILTY FINE SAND		2	S.S.	42								
440	30													
430	38.0													
429.3	40	FIRM, Grey CLAYEY SILT to SILTY CLAY with traces of sand and gravel.		3	S.S.	6								
420	50													
410	60			4	S.S.	6								
400	70													
391.8	75.0			5	S.S.	86								
390	80	VERY DENSE, Grey SANDY SILT with some clay and embedded gravel to SILT with some clay (GLACIAL TILL)		6	S.S.	93.5								
				7	S.S.	73								
				8	S.S.	51								
385.3	84.0			9	S.S.	137								
380	90	VERY DENSE Grey Fine to Medium SAND		10	S.S.	175								
				11	W.S.	—								
				12	S.S.	1002A								
83.3		END OF BOREHOLE												
370	100													

SLIGHT ARTESIAN
PRESSURE BELOW
EL. 380 ± ft.

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D. A. M. CH'D

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 6. (6A)

OUR REFERENCE NO. 6-12-1
 Your Ref. No. W.P. 266-61
 CLIENT D.H.O.
 PROJECT HWY NR 401 & LESLIE ST
 LOCATION BETWEEN CAISSONS 329-2 & 329-3
 DATUM ELEVATION G.S.C

METHOD OF BORING: WASH BORING
 DIAMETER OF BOREHOLE: 2 3/8"
 DATE: DEC 19-23, 1966

ENCLOSURE NO.

PROJECT HWY N-1										LOCATION BETWEEN CAISSONS 329-2 & 329-3										DATUM ELEVATION G.S C										PENETRATION RESISTANCE blows per foot 210 410 610 810 100										CONSISTENCY water content % 1 W 11										REMARKS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
ELEVATION ft		DEPTH ft		STRATIFICATION DESCRIPTION		STRATIFICATION SYMBOL		SAMPLES		NUMBER		TYPE		-1- or Advances at Sampler		SHEAR STRENGTH lbs sq ft																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											

SLIGHT ARTESIAN
 PRESSURE AT
 EL. 378 ft

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CHD

HWY NR 401 & LESLIE ST

PLANS 100 120 150 180 210 240

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 7 (7A)

OUR REFERENCE NO. 6-12-1
 Your Ref. No. W.J. 266-81
 CLIENT D.M.O.
 PROJECT HWY NO 401 @ LESLIE ST.
 LOCATION BETWEEN CAISSONS 228-1 & 228-2
 DATUM ELEVATION G.S.C.

METHOD OF BORING WASHBORING
 DIAMETER OF BOREHOLE 2 3/8"
 DATE DEC 30, 1966 - JAN 4, 1967
 ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	N ₆₀ or Admission at Sampler	2.0	4.0	6.0	8.0	10.0	PL	
466.3	0	GROUND SURFACE											
460	10	COMPACT Brown CLAYEY - SILTY FINE SAND (FILL)		1	SS	25							
451.3 450	15.0	COMPACT Brown SILTY FINE SAND		2	SS	17							
440	20												
433.3 430	33.0	SOFT to FIRM Grey SILTY CLAY with occasional brown fine sand seams		3	SS	4							
420	40												
410	50			4	SS	5							
				5	SS	4							
400	60	Sandy below El. 403 ± ft		6	SS	9							
393.8 72.5	70	VERY DENSE, Grey SANDY SILT with a trace of clay and gravel.		7	SS	100/25							
390	76.0	HARD, Grey CLAYEY SILT with some embedded gravel (GLACIAL TILL)		8	SS	86							
	80			9	SS	78							
				10	SS	197							
384.3 82.0		VERY DENSE, Grey FINE SAND with pockets of clayey silt.		11	SS	150/8							
380	86.0	END OF BOREHOLE		12	SS								
	90			13	SS	150/6							

BOULDER AT
EL. 380 ft.

VERTICAL SCALE 1 IN 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CHD

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 8. (8A)

OUR REFERENCE NO. 6-12-1
 YOUR REF. NO. W.P. 266-61
 CLIENT D.H.O.
 PROJECT HWY. NO. 401 & LESLIE ST
 LOCATION BETWEEN CAISSONS 228-3 & 328-1
 DATUM ELEVATION 6.5 C

METHOD OF BORING WASHBORING
 DIAMETER OF BOREHOLE 2 3/8"
 DATE JAN. 6 - 7, 1967

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	1/2" or Advance of Sampler	20	40	60	80	100	PL	W	LI	
459.1	0	GROUND SURFACE													
		Brown CLAYEY SILT and SAND													
450	10	(FILL)													
456.1	13.0	DENSE, Brown Medium to Fine SAND trace of gravel		1	S.S.	44									
450				2											
449.0	20	COMPACT, Brown Fine SAND with some silt (wet)		2	S.S.	24									
440	30														
434.1	35.0	FIRM to STIFF													
430	40	Grey		3	S.S.	8									
420	50	CLAYEY SILT with some sand													
410	60			4	S.S.	10									
400	70														
393.1	76.0	VERY DENSE, Grey SANDY SILT to		5	S.S.	36									
390	80	SILT with a trace of clay (GLACIAL TILL)		6	S.S.	138									
387.1	82.0	VERY DENSE - SAND		7	S.S.	84									
83.5		END OF BOREHOLE		8	S.S.	100/5									
380	90														
	100														

SLIGHT ARTESIAN
 PRESSURE OBSERVED
 BELOW EL. 387 ft.

VERTICAL SCALE 1 IN 10

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CHD

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 9. (9A)

OUR REFERENCE NO 6-12-1
 Your Ref No W.P. 266-61
 CLIENT D. H. O.
 PROJECT HWY. NO 401 & LESLIE ST.
 LOCATION BETWEEN CAISSONS 32B-2 & 32B-3
 DATUM ELEVATION G S C.

METHOD OF BORING AUGERING & WASHBORING
 DIAMETER OF BOREHOLE 3" B 2 3/8" ENCLOSURE NO
 DATE DEC. 19-29, 1966

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content % PL W LI	REMARKS
				NUMBER	TYPE	N or Adjacent to Sample	20	40	60	80		
478.6	0	GROUND SURFACE										
470	10	Brown SILT and SAND with some gravel and clay. FILL		1	S.S.	24						
460	20	COMPACT SILTY FINE SAND		2	S.S.	27						
450	30			5	S.S.							
440	40	Brown Grey		4	S.S.	29						
430	50	FIRM to STIFF Grey CLAYEY SILT		5	S.S.	6						
420	60	with some sand and fine gravel		6	S.S.	7						
410	70			7	S.S.	10						
400	80	occasional thin sand seams below elev. 404 ft.		8	S.S.							
				9	S.S.	16						
				10	S.S.	10						
392.3	86.3	HARD, Grey CLAYEY SILT		11	S.S.	70/8"						
39.7	90	SILT SEAM (SLIGHTLY COHESIVE) with some embedded gravel. (GLACIAL TILL)		12	S.S.	153						
				13	S.S.	130						
				14	S.S.							
181.3	97.3	VERY DENSE MEDIUM to FINE SAND some silt, gravel		15	S.S.	150/2"						
180	100			16	S.S.	149/2"						
102.5	102.5	END OF BOREHOLE										

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CH'D

GEOTECHNICAL DATA SHEET FOR BOREHOLE .10. (10A)

OUR REFERENCE NO 6 - 12 - 1
 Your Ref No 266 - 61
 CLIENT D. H. O.
 PROJECT HWY No 401 & LESLIE ST
 LOCATION BETWEEN CAISSONS 227-1 & 227-2
 DATUM ELEVATION G. S. C

METHOD OF BORING WASH BORING
 DIAMETER OF BOREHOLE 2 3/8"
 DATE JAN 9, 1967

ENCLOSURE NO

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content % PL W LI	REMARKS
				NUMBER	TYPE	Advancement of Sampler	2.0	4.0	6.0	8.0	10.0		
467.5	0	GROUND SURFACE											
460	10	COMPACT Brown SANDY SILT with some clay (FILL)		1	SS	10							
440	20												
439.5	28.0	COMPACT, Brown SILTY FINE SAND wet		2	SS	21							
432.5	35.0												
430	40	FIRM, Grey CLAYEY SILT with some sand		3	S.S.	4							
420	50												
410	60			4	S.S.	8							
400	70												
393.5	74.0	VERY DENSE, Grey SANDY SILT with some clay and gravel (GLACIAL TILL)		5	S.S.	9.9							
390	76.0			6	S.S.	9.4							
388.5	80	VERY DENSE, Brown FINE to MEDIUM SAND		7	S.S.	10.3							
381.9	84.0	END OF BOREHOLE		8	S.S.	10.4							
380	90												

SLIGHT ARTESIAN
 PRESSURE AT
 EL. 387.5'

VERTICAL SCALE: 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE C. A. M. CRD

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . (11A)

OUR REFERENCE NO. 8-12-1
Your Ref. No. W. P. 266-61

CLIENT D. H. O.
PROJECT HWY. NO 401 & LESLIE ST.
LOCATION BETWEEN CAISSONS 227-3 & 327-1
DATUM ELEVATION G. S. C.

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE 2 3/8"
DATE JAN. 13 - 14, 1967

ENCLOSURE NO

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE		CONSISTENCY		REMARKS
				NUMBER	TYPE	Advancement of Sampler	blows per foot	lb/sq ft	Pl	W	
168.0	0	GROUND SURFACE									
		Brown SAND, SILT and some clay FILL									
160	10										
154	14	Compact to Dense Brown FINE SAND with some silt		1	SS	18					
150	20										
140	30			2	SS	38					
133.5	34.5										
130	40	Firm to Stiff Grey CLAYEY SILT		3	SS	4					
120	50										
110	60	SANDY below EL. 410 ft.		4	SS	6					
100	70			5	SS	8					
				6	SS	49					
90.5	77.5	HARD, GREY CLAYEY SILT with a trace of sand and embedded gravel (GLACIAL TILL)		7	SS	44					
85	85	VERY DENSE SAND		8	SS	81					
182	86	END OF BOREHOLE		9	SS	107/6					
180	90										
	100										

SLIGHT ARTESIAN PRESSURE OBSERVED AT EL. 383 ft.

VERTICAL SCALE: 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE V. G. H. CHD

GEOTECHNICAL DATA SHEET FOR BOREHOLE... 12. (12A)

OUR REFERENCE NO. 6-12-1
Your Ref. No. WP. 266-61
CLIENT D. H. O.
PROJECT HWY # 401 & LESLIE ST.
LOCATION BETWEEN CAISSONS 327-2 & 327-3
DATUM ELEVATION G.S.C

METHOD OF BORING, AUGERING & WASHBORING ENCLOSURE NO
DIAMETER OF BORING 3" 8 2 3/8
DATE DEC 20-29, 1966

ELEVATION ft		DEPTH ft		STRATIFICATION DESCRIPTION		STRATIFICATION SYMBOL		SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %		REMARKS		
								NUMBER	TYPE	Advancement of sampler	20	40	60	80	100				
											SHEAR STRENGTH lbs sq ft								
180.6	0	GROUND SURFACE																	
		Brown SAND and SILT with some gravel and clay.																	
		FILL																	
	10							1	S.S.	26									
	20	COMPACT FINE SAND and SILT						2	S.S.	18									
	30							3	S.S.	16									
	40							4	S.S.	10									
	50	Brown Grey						5	S.S.	5									
	60	FIRM to STIFF 9" BOULDER						6	P.L.										
	70	Grey CLAYEY SILT with some fine sand and gravel						7	S.S.	7									
	80							8	S.S.	11									
	90							9	S.S.	12									
	100							10	S.S.	7									
	110	HARD, Grey SILT with some clay to CLAYEY SILT (GLACIAL TILL)						11	S.S.	100									
	120							12	S.S.	75/8									
	130	VERY DENSE, Grey FINE to MEDIUM SAND with a trace of gravel						13	S.S.	54/8									
	140							14	S.S.	25/8									
	150							15	S.S.	25/8									
	160	BOULDER						16	S.S.	25/8									
	170	END OF BOREHOLE						17	S.S.	50/8									

So. 16-W.S. - 95'5" to 96'
So. 18-W.S. - 96'5" to 97'

So. 22-RC - 101' to 101'5"

SLIGHT ARTESIAN PRESSURE BELOW EL 380 ft.

MADE: D. A. M. CND:

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CH'D:

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 13. (13A)

OUR REFERENCE NO. 6-12-1
Your Ref. No. W.P. 266-61

CLIENT: D. H. O.

PROJECT: HWY. NO. 401 & LESLIE ST.

LOCATION: BETWEEN CAISSONS 226-1 & 226-2

DATUM ELEVATION: G.S.C.

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE 2 3/8
DATE JAN. 9-10, 1967

ENCLOSURE NO.

LOCATION: BETWEEN CAISSONS													
DATUM ELEVATION: G.S.C.													
ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE			CONSISTENCY			REMARKS
				NUMBER	TYPE	Advancement of Sampler	blows per foot	2.0	4.0	6.0	8.0	10.0	
SHEAR STRENGTH lbs./sq. ft.													
469.2	0	GROUND SURFACE											
		COMPACT Brown											
460	10	SANDY SILT with some clay (FILL)		1	S.S.	13							
450	20												
441.2 440	28.0 30	LOOSE Brown FINE SAND trace of silt.		2	S.S.	8							
431.2 430	38.0 40												
		STIFF Grey CLAYEY SILT		3	S.S.	8							
420	50												
410	60	Sandy below el. 413.2'		4	S.S.	17							
400	70			5	S.S.	8							
				6	S.S.	30							
393.4	75.8	VERY DENSE, Grey SANDY SILT with some clay and gravel. (TILL) (cohesive)		7	S.S.	103							
390	80			8	S.S.	90							
386.7	80.5	VERY DENSE, FINE SAND with layers of HARD CLAYEY SILT		9	S.S.	166/10							
				10	S.S.	120/4							
83.8		END OF BOREHOLE											
380	90												
SLIGHT ARTESIAN PRESSURE OBSERVED BELOW EL. 387.2													
MADE. D.A.M. CHD.													

SLIGHT ARTESIAN
PRESSURE OBSERVED
BELOW EL. 387.2'

VERTICAL SCALE 1 in TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CHD.

GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 14. (14A)

OUR REFERENCE NO 5-12-1
YOUR Ref. No W. P. 266-61

CLIENT: D. H. O.

PROJECT: HWY. No 401 @ LESLIE ST.

LOCATION: BETWEEN CAISSONS 226-3 & 226-1
DATUM ELEVATION: 6.5 C.

METHOD OF BORING WASHBORING
DIAMETER OF BOREHOLE 2 3/8"
DATE JAN. 11 - 12, 1967

ENCLOSURE NO

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	Advancement of Sampler	210	40	610	810	PL	W	
469.8	0	GROUND SURFACE											
460	10	Brown SAND, SILT some clay FILL											
456.2	13.6	Black, Organic SILTY SAND		1	S.S.	23							
450.8 450	19.0 20	Loose, Grey SILTY FINE SAND		2	S.S.	7							
435.8	34.0	FIRM Grey CLAYEY SILT		3	S.S.	4							
430	40			4	S.S.	6							
420	50			5	S.S.	11							
410	60	SANDY below EL. 410 ft.		6	S.S.	8							
400	70			7	S.S.	92							
391.8 390	78.0 80	HARD, Grey, CLAYEY SILT with embedded GRAVEL		8	S.S.	105/6"							
386.3	83.5	VERY DENSE, Grey SILTY FINE SAND some gravel slightly cemented (GLACIAL TILL)		9	S.S.	118/3"							
383.3	86.5			10	S.S.	125/3"							
380	90	VERY DENSE, Medium To Coarse SAND		11	S.S.	150/3"							
		END OF BOREHOLE		12	S.S.	200/4"							
370	100			13	S.S.	200/4"							

SLIGHT ARTESIAN
PRESSURE OBSERVED
BELOW EL. 391 ft.

VERTICAL SCALE: 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D. A. M. CWD

GEOTECHNICAL DATA SHEET FOR BOREHOLE 15 (15A)

OUR REFERENCE NO. 6-12-1
Your Ref. No. W.P. 286-61

CLIENT D. H. O.
PROJECT HWY. NO 401 & LESLIE ST.
LOCATION BETWEEN CAISSONS 326-2 & 326-3
DATUM ELEVATION G. S. C.

METHOD OF BORING, WASHBORING
DIAMETER OF BOREHOLE 2 1/8"
DATE JAN. 14-17, 1967.

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE					CONSISTENCY water content %	REMARKS
				NUMBER	TYPE	N ₆₀ blows per foot Adjusted to Standard	20	40	60	80	100		
169.5	0	GROUND SURFACE											
460	10	Dense, Brown SAND, SILT and GRAVEL FILL											
455.5	13				SS	13							
450	20	Compact to Dense SILTY FINE SAND											
440	29	Brown											
	30	Grey		2	SS	37/8							
434.8	34.5												
430	40	Firm, Grey SILTY CLAY to CLAYEY SILT		3	SS	3							
420	50												
410	59	SANDY below EL. 410 ft.		4	SS	9							
400	70			5	SS	6							
				6	SS	5							
390	79.2	Very Dense Grey SAND, GRAVEL and BOULDERS with CLAYEY SILT binder between coarse particles		7	SS	120							
	80			8	SS	145							
				9	SS	145							
				10	SS	142							
				11-14	SS	145/50							
				15	SS	100/2							
380	88.2	END OF BOREHOLE											
370	100												

CASING AT EL. 390 ft.
BOREHOLE CAVING
BETWEEN EL. 389 and
385 ft.

CASING AT EL. 385 ft.
BOREHOLE CAVING
BELOW EL. 382 ft.

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE V. G. M. CHD

GEOTECHNICAL DATA SHEET FOR BOREHOLE 16 (16A)

OUR REFERENCE NO. 6-12-1
 Your Ref. No. W.P. 266-61
 CLIENT D.H.O.
 PROJECT HWY. NO. 401 B LESLIE ST
 LOCATION BETWEEN CAISSONS 225-1 & 225-2
 DATUM ELEVATION G.S.C.

METHOD OF BORING WASHBORING
 DIAMETER OF BOREHOLE 2 3/4"
 DATE JAN 9-10, 1967

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	Advancement of Sampler	20	40	60	80	100	Pl	W	
474.6	0	GROUND SURFACE												
470	10	COMPACT Brown SILT with some clay, sand. (FILL)		1	S.S.	16								
453.6	21.0	DENSE Brown FINE SAND with some silt		2	S.S.	44								
431.6	43.0	FIRM Grey CLAYEY SILT with some sand		3	S.S.	6								
410	60			4	S.S.	6								
400	80			5	S.S.	7								
392.6	82.0	VERY DENSE, Grey SANDY SILT with some clay (cohesive)		6	S.S.	128								
390	84.0	DENSE		7	S.S.	100/4								
389.6	85.0	SILTY SAND		7A	W.S.									
386.6	88.0	HARD, Grey CLAYEY SILT with embedded gravel and shale fragments		8	S.S.	125/4								
	90			9	S.S.	100/2								
				10	S.S.	100/2								
				11	S.S.	130/2								
				12	S.S.	135/2								
380		BOULDER (GLACIAL TILL)		13	RC.									
				14	S.S.	135/6								
	100	SAND SEAM		15	S.S.	125/6								
				16	S.S.	100/7								
370		SAND		17	S.S.	102/8								
108		END OF BOREHOLE												

BOREHOLE CAVES AT
 EL. 388 ft. AND
 SLIGHT ARTESIAN
 PRESSURE OBSERVED.
 DROVE CASING TO
 EL. 385 ft.

ARTESIAN PRESSURE
 OBSERVED BELOW
 EL. 368

MADE D. A. M. CHD

DOMINION SOIL INVESTIGATION LIMITED

VERTICAL SCALE 1 IN TO 10 FT

GEOTECHNICAL DATA SHEET FOR BOREHOLE 17 (17A)

OUR REFERENCE NO. W-12-1
Your Ref. No. W.P. 266-61

CLIENT D.H.O.
PROJECT HWY. NO. 401 & LESLIE ST.
LOCATION BETWEEN CAISSONS 225-3 & 325-1
DATUM ELEVATION G.S.C.

METHOD OF BORING WASH BORING
DIAMETER OF BOREHOLE 2 3/8"
DATE JAN. 11-12, 1967.

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE		CONSISTENCY		REMARKS
				NUMBER	TYPE	Advancement of Sampler	blows per foot	SHEAR STRENGTH	water content %		
471.9	0	GROUND SURFACE									
470		COMPACT Brown SILTY SAND with trace of clay and gravel FILL		1	SS	26					
460	10										
454.9	17	Loose to Compact Brown SILTY FINE SAND		2	SS	24					
450	20										
440	30										
432.9	34	Firm to Stiff Gray CLAYEY SILT		3	SS	6					
430	40										
420	50										
410	60										
		SANDY below EL. 412 ft.		4	SS	11					
400	70			5	SS	9					
391.7	80.2	SANDY SILT TILL SAND		6	SS	87/6					
391.0	80.9			7	SS	75/6					
390.5	81.4	HARD, GREY CLAYEY SILT with some sand and embedded gravel and shale fragments (GLACIAL TILL)		8	SS	100/2					
				9	SS	95/2					
				10	SS	104/2					
382.9	89	END OF BOREHOLE		11	SS	100/6					
380	90										
	100										

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE V.G.H. CND

GEOTECHNICAL DATA SHEET FOR BOREHOLE ...18 (18A)

OUR REFERENCE NO 6-12-1
 Your Ref. No W.P. 266-61
 CLIENT D. H. O.
 PROJECT HWY. NR 401 @ LESLIE ST
 LOCATION BETWEEN CAISSONS 325-2 & 325-3
 DATUM ELEVATION G. S. C.

METHOD OF BORING AUGERING & WASHBORING
 DIAMETER OF BOREHOLE 3" @ 2 3/8"
 DATE DEC 21-31, 1966
 ENCLOSURE NO

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE Blows per foot					CONSISTENCY water content % PL W LI	REMARKS
				NUMBER	TYPE	2" or 4" or 6" or Adjustment of Sampler	2.0	4.0	6.0	8.0	10.0		
483.4	0	GROUND SURFACE											
480	10	Brown SILTY SAND with traces of clay, gravel and organic matter		1	SS	24							
470	20	FILL											
460	26.5			2	SS	79							
450	30	COMPACT to DENSE Brown to Grey FINE SAND with some silt		3	SS	40							
440	40												
430	48.0			4	SS	5							
420	60	FIRM, Grey CLAYEY SILT with a trace of sand and fine gravel.		5	SS	4							
410	70												
400	80	Sandy below E' 406 ft.		6	SS	9							
390	90			7	SS	10							
				8	SS	10							
				9	SS	9							
				10	SS	17							
380	92.0	VERY DENSE, Grey SAND, GRAVEL and SILT with some clay. (SLIGHTLY COHESIVE)		11	SS	98							
				12	SS	100							
				13	SS	100							
	99.0	HARD, Grey CLAYEY SILT (GLACIAL TILL)		14	SS	145							
	103.0	FINE TO MEDIUM SAND END OF BOREHOLE		15	SS	139							

SLIGHT ARTESIAN
PRESSURE OBSERVED
AT EL. 380 ft.

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE O.A.M. CND

DEPARTMENT OF HIGHWAYS
MATERIALS & RESEARCH

JOB 64-F-41
W.P. 252-61-3
DATUM G.S.C.

RECORD OF BOREHOLE NO. 1 (B1)

Stn. 128+73 and 130' Rt. of E. Hwy. 401
May 28, 1964.
Washboring using BX casing.

B.H.G.
B.H.G.
M.D.

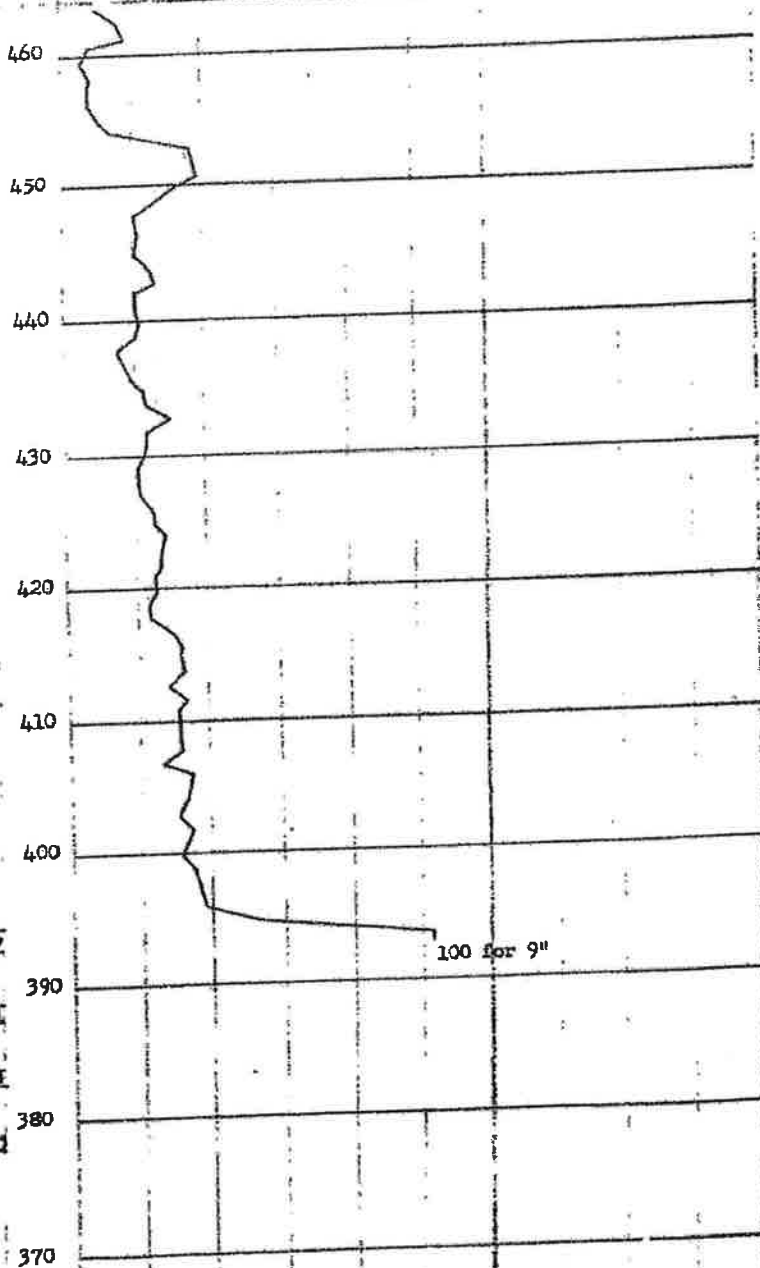
W. PENETRATION RESISTANCE
AS FOOT
20 40 60 80 100

ELEV
DEPTH

454	Groundlevel
0.6	Topsoil
	Clayey silt with traces of fine gravel and organics.
452	Brown.
12.0	Silt and fine sand.
	Brown changing to grey at 24' depth.
435	
29.0	Silty clay with some fine sand.
	Grey.
395	
69.0	Heterogeneous mixt. of clayey silt, sand and trace of gravel. (Glacial Till)
	Dense to v. dense.
	Grey.
378.1	
85.11	End of borehole.

1	SS	37
2	SS	61
3	SS	103
4	SS	100

for 5"



N.L.
452

2 (B2)

B.M.G.
B.M.G.
M.D.

Stn. 129773 and 1351 Rt. of S. Hwy. 491
May 29, 1964.
Washboring using BX casing.

64-F-41
252-61-3
G.S.C.

461	Groundlevel
0.6	Clayey silt with some sand and fine gravel. Traces of organics.
452	
9.0	Silt and fine sand.
	Brown changing to grey at 23' depth.
429	
32.0	Silty clay with some fine sand.
	Grey
393	
68.0	Heterogeneous mixt. of clayey silt, sand and trace of gravel. (Glacial Till) V. dense. Grey.
380	
81.0	End of borehole.

460	
450	
440	
430	
420	
410	
400	
390	
380	

W.I.
El. 447

1	89	52	390
2	89	100	
		for 10"	
3	89	100	380

64-F-41
252-61-3
G.S.G.

Stn. 130775 and 1301 Rt. of E. Hwy. 401
June 2, 1964.
Washboring using BX casing.

3 (33)

B.M.G.
B.M.C.
M.D.

459.5 Groundlevel

0.6 Clayey silt with
trace of sand.

Brown.

445.5

14.0 Silt and fine sand.
Brown changing to
grey at 19'-6" depth.

434.5

25.0 Silty clay with
some fine sand.

Grey.

391.0

68.6 Heterogeneous mixt.
of clayey silt, sand
and trace of gravel.

(Glacial Till)

Dense to v. dense
Grey

374.0

85.6 End of borehole.

1 SS A1

2 SS 37

3 SS 100
for 11"

4 SS 100

for 6"

460

450

440

430

420

410

400

390

380

370

W.L.
El. 444

OFFICIAL
MAINTENANCE

LOG 64-F-41
W P 252-61-3
DATE G.S.C.

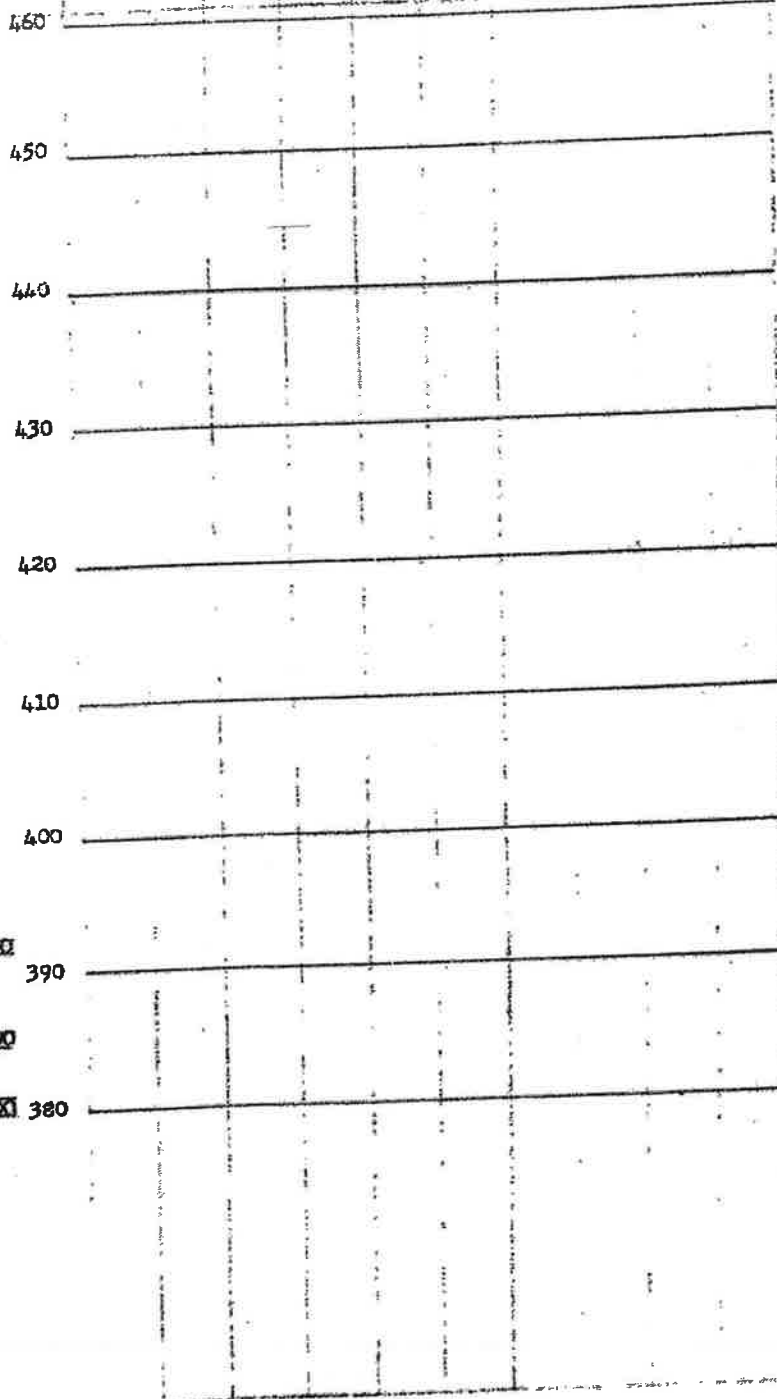
STATION 127+75 and 133' Lt. of G. Bay 401
June 3, 1964.
Washboring using BX casing.

4 (B4)

B.M.O.
B.M.O.
H.D.

461	Groundlevel	
0.0	Clay silt, sand and traces of fine gravel.	
452	Brown	
9.0	Silt and sand.	
	Brown	
440		
21.0	Silty clay with some fine sand.	
	Grey.	
393		
68.0	Heterogeneous mixt. of clayey silt, sand and gravel. V. dense Grey	
380		
81.0	End of borehole.	

1	SS 100	
	for 7"	390
2	SS 100	
	for 6"	
3	SS 100	
	for 6"	380



N.L.
El. 445

RECORD OF BOREHOLE NO. 5 (B.5)

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

JOB 64-F-41

LOCATION Stn. 129+00 and 135' Lt. of E. Hwy. 401

ORIGINATED BY B.H.G.

W.P. 252-61-3

BORING DATE June 4, 1964

COMPILED BY B.H.G.

DATUM G.S.C.

BOREHOLE TYPE Washboring using BX casing

CHECKED BY M.D.

SOIL PROFILE		STRAT. PLCT	SAMPLES		BLOWE / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARK
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE			20	40	60	80	100	PD	W	WL		
461	Groundlevel					460										
0.6	Silty sand with traces of fine gravel, clayey silt and organics to El. 445. Loose to compact. Brown changing to grey at El. 440.		1	SS	9											
			2	SS	13	450										
			3	SS	8											
			4	SS	8	440										
438			5	TW	22											
23.0	Silty clay with some sand and trace of fine gravel. Firm to stiff. Grey		6	TW	P	430										
			7	TW	P											
			8	TW	P	420										
			9	SS	5	410										
			10	TW	P											
			11	SS	4	400										
399.5			12	SS	101											
61.6	Heterogeneous mixt. of sand, gravel up to 1" and clayey silt. (Glacial Till) V. dense Grey.		13	SS	100	390										
			14	SS	100											
			15	SS	100	380										
380.7																
80.4	End of borehole.															
						370										

W.L.
El. 451

127 Sens-6

Sens-2

Sens-2

Sens-2

Sens-2

115

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 6 (B6)

FOUNDATION SECTION

JOB 64-F-41 LOCATION Stn. 130+00 & 136' Lt. of E. Hwy. 401 ORIGINATED BY B.M.C.
W.P. 252-61-3 BORING DATE June 4, 1964. COMPILED BY B.M.C.
DATUM G.S.C. BOREHOLE TYPE Washboring using BX casing. CHECKED BY M.D.

SOIL PROFILE		STRAT. PLOT	SAMPLES		BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARK
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE			BLOWS / FOOT				WATER CONTENT %				
459	Groundlevel					460									
0.0	Clayey silt with some sand. Brown.					450									
448															
11.0	Silt and fine sand. Brown changing to grey at 21' depth.					440									
433															
26.0	Silty clay with some sand and fine gravel. Grey.					430									
						420									
						410									
						400									
396			1	SS	4										
63.0	Heterogeneous mixt. of clayey silt, sand and some fine gravel. (Glacial Till) Dense to v. dense. Grey.		2	SS	33										
			3	SS	66										
			4	SS	100										
					for 6"										
378.5			5	SS	100										
80.6	End of borehole.				for 6"										
						370									

W.L.
El. 445

RECORD OF BOREHOLE NO. 7 (B7)

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

JOB 64-F-41

LOCATION Stn. 131+00 and 141' Lt. of E. Hwy. 401

ORIGINATED BY B.H.G.

W P 252-61-3

BORING DATE June 8, 1964

COMPILED BY B.H.G.

DATUM G.S.C.

BOREHOLE TYPE Washboring using BX casing.

CHECKED BY M.D.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE		WATER CONTENT		BULK DENSITY P.C.F.	REMARK
			NUMBER	TYPE		BLOWS / FOOT	BLANKS / FOOT	W	WL		
458	Ground level				460						
0.6	Clayey silt with sand and gravel. Brown.				450						
					440						
437					430						
21.0	Silt and fine sand with organic wood material around El. 433.				420						
424					410						
34.0	Silty clay with some sand. Grey.				400						
					380						
391					370						
67.0	Heterogeneous mixt. of clayey silt, sand and gravel up to 1"Ø. (Glacial Till) V. dense Grey		1	SS - 100% for 6"							
			2	SS - 100% for 9"							
			3	SS - 100% for 9"							
			4	SS - 100% for 6"							
81.0	End of borehole.										

W.L.
El. 448

RECORD OF BOREHOLE NO. 11 (B11)

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

JOB 64-F-41

LOCATION Stn. 127+95 and 118' Rt. of E. Hwy. 401

ORIGINATED BY D.M.G.

W P 252-61-3

BORING DATE June 12, 1964.

COMPILED BY B.M.G.

DATUM G.S.C.

BOREHOLE TYPE Washboring using BX casing.

CHECKED BY M.D.

SOIL PROFILE		BORE HOLE		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — WL		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.			
466	Ground level								
0.6	Silty fine sand. Brown.				460				W.L. El. 459.
449					450				
17.0	Silty clay with some sand and gravel. Gray.				440				
					430				
					420				
			1 SS 18		410				
401.5					400				
64.6	Heterogeneous mixt. of clayey silt, silt, sand & trace of fine gravel. (Glacial till) V. dense Gray.		2 SS 100 for 6"						
			3 SS 12						
			4 SS 103						
			5 SS 100 for 10"		390				
			6 SS 100 for 6"						
385					380				
81.0	End of borehole.								

RECORD OF BOREHOLE NO. 12 (B12)

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

JOB 64-F-41

LOCATION Stn. 129+35 and 120' Ht. of E. Hwy. 401

ORIGINATED BY B.M.G.

W P 252-61-3

BORING DATE June 15, 1964.

COMPILED BY B.M.G.

DATUM U.S.C.

BORING TYPE Washboring using BX casing.

CHECKED BY H.D.

DATUM U.S.C.		BOREHOLE TYPE		WATER CONTENT		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT		PLASTIC LIMIT		WATER CONTENT		BULK DENSITY		REMARKS	
SOIL PROFILE		SAMPLE		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		REMARKS	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	BLOWS / FOOT	REMARKS
460	Groundlevel				460												
0.6	Clayey silt, some sand and fine gravel. Brown.				450												
446.5					440												
13.6	Silt and fine sand. Brown changing to grey at El. 436.				430												
27.0	Silty clay with some sand. Grey.				420												
392					410												
68.0	Heterogeneous mixt. of clayey silt, sand, & trace of gravel up to 1/4". (Glacial till) V. dense Grey.		1	SS	36												
379.0			2	SS	82												
81.0	End of borehole.		3	SS	100												

W.L.
El. 447.

DEPARTMENT OF HIGHWAYS - CALIFORNIA
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 13 (B13)

FOUNDATION SECTION

JOB 64-F-41
W P 252-61-3
DATUM U.S.C.

LOCATION Stn. 131/32 and 173' Rt. of E. Hwy. 401
BORING DATE June 16, 1964
BOREHOLE TYPE Washboring using BX casing.

ORIGINATED BY B.N.G.
COMPILED BY B.N.G.
CHECKED BY H.D.

SOIL PROFILE			SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — WL		BULK DENSITY	REMARK
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P S F.	PLASTIC LIMIT — WP	WATER CONTENT — W		
459	Groundlevel				460						
0.0	Clayey silt, with some sand and fine gravel. Brown.				450						
16.0	Silt and fine sand Gray				440						
19.0	Silty clay with some sand. Gray.				430						
					420						
					410						
					400						
					390						
71.5	Heterogeneous mixt. of clayey silt, sand & trace of gravel up to 1" (Glacial till) V. dense Gray.		1	SS	106						
			2	SS	100 3/8"						
			3	SS	100 5/8"						
			4	SS	100 6/8"						
85.6	End of borehole.				370						

RECORD OF BOREHOLE NO. 15 (B15)

FOUNDATION SECTION

DEPARTMENT OF HIGHWAYS & BRIDGES
MATERIALS & RESEARCH DIVISION

JOB 64-P-41

LOCATION Sta. 132+50 & 140' W. of E. Hwy. 401

ORIGINATED BY B.M.G.

W.P. 252-61-3

BORING DATE June 18, 1964.

COMPILED BY B.M.G.

DATUM G.S.C.

BOREHOLE TYPE Washboring using BX casing.

CHECKED BY H.D.

DATE		G.S.C.		BOREHOLE TYPE		REMARKS	
SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT		BLOWS / FOOT	
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OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. 31821012 JOB 31821012 BORING 31821012
CASING 1" 22 (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM 22 22 22 DATE REPORT SEP 1968
SAMPLER HAMMER WT. 250 DROP 11 1/2 INCHES COMPILED BY 31821012 CHECKED BY 31821012 BORING DATE AD 1968

SAMPLE CONDITION

**DISTURBED
FAIR
GOOD
LOST**

SAMPLE TYPES

C.S. - CHUNK	F.S. - FOIL SAMPLE
D.O. - DRIVE-OPEN	B.A. - BARREL AUGER
D.F. - DRIVE-FOOT VALVE	S.A. - SPIRAL AUGER
D.P. - DRIVE PISTON	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE
T.P. - THIN WALLED PISTON	

ABBREVIATIONS

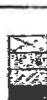
ABBREVIATIONS

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

SOIL PROFILE					T.P. - THIN WALLED PISTON			SAMPLES					
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W %			OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	D.P. RECH.
					NAT.	CLW	PW.						
					PENETRATION TEST RESISTANCE BLOWS PER FOOT								
45.0		100% SAND									1	10	45.0
44.5		100% SAND									2	10	44.5
44.0		100% SAND									3	10	44.0
43.5		100% SAND									4	10	43.5
43.0		100% SAND									5	10	43.0
42.5		100% SAND									6	10	42.5
42.0		100% SAND									7	10	42.0
41.5		100% SAND									8	10	41.5
41.0		100% SAND									9	10	41.0
40.5		100% SAND									10	10	40.5
40.0		100% SAND									11	10	40.0
39.5		100% SAND									12	10	39.5
39.0		100% SAND									13	10	39.0
38.5		100% SAND									14	10	38.5
38.0		100% SAND									15	10	38.0
37.5		100% SAND									16	10	37.5
37.0		100% SAND									17	10	37.0
36.5		100% SAND									18	10	36.5
36.0		100% SAND									19	10	36.0
END OF PENETRATION TEST - ELEV. 35.5													

OFFICE REPORT ON SOIL EXPLORATION

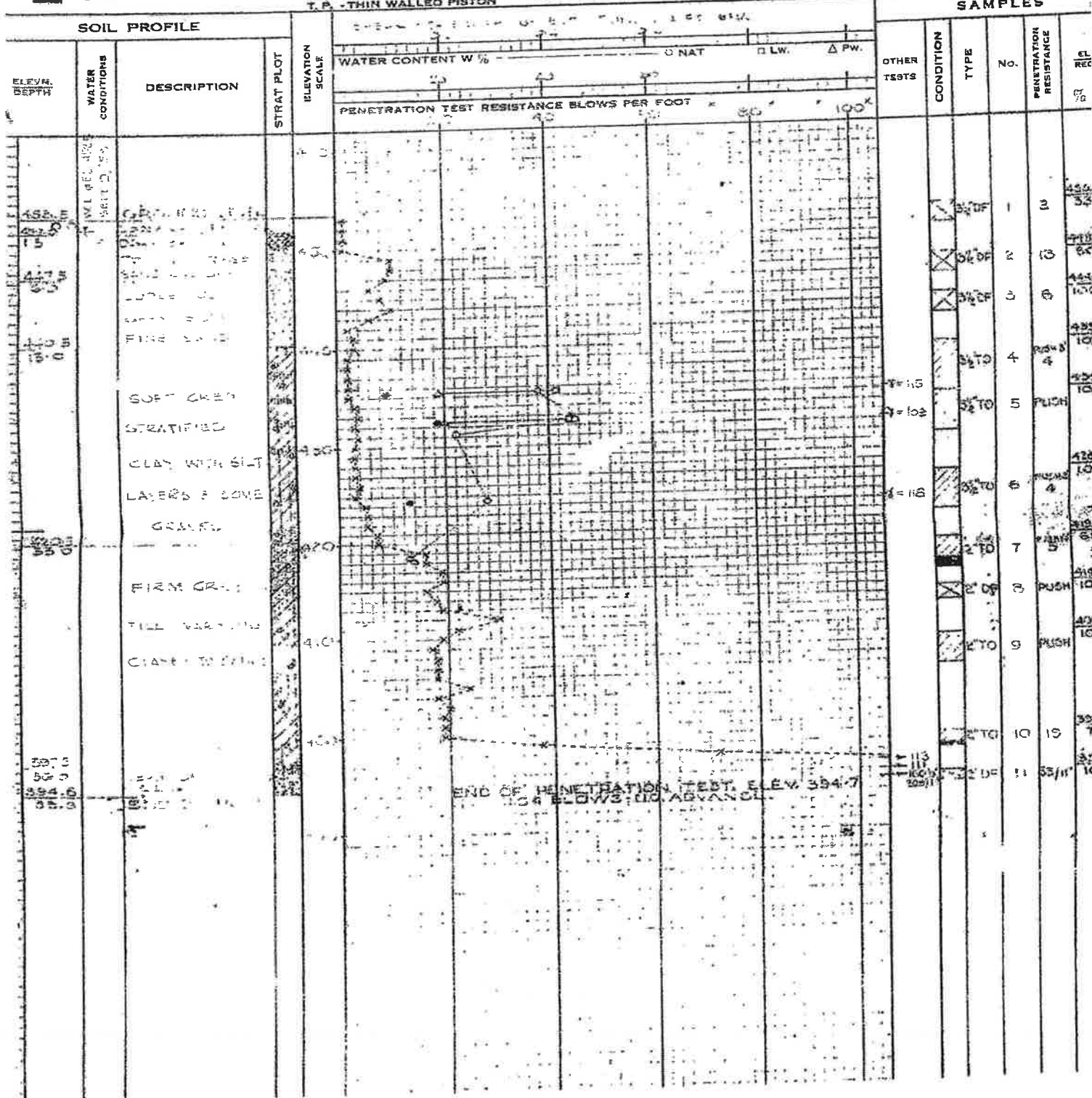
DRILL RIG: _____ JOB: _____ BORING # C-11
 CASING: _____ (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM: _____ DATE REPORT: SEP 23 1953
 SAMPLER HAMMER, WT. _____ DROP: _____ INCHES COMPILED BY: _____ CHECKED BY: _____ BORING DATE: SEP 23 1953



SAMPLE CONDITION
 DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES
 C.S. - CHUNK
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 D.P. - DRIVE PISTON
 T.O. - THIN WALLED OPEN
 T.P. - THIN WALLED PISTON
 F.S. - FOIL SAMPLE
 S.A. - BARREL AUGER
 S.A. - SPIRAL AUGER
 W.S. - WASHED SAMPLE
 R.C. - ROCK CORE

ABBREVIATIONS
 V. - IN-SITU VANE SHEAR TEST
 M. - MECHANICAL ANALYSIS
 U. - UNCONFINED COMPRESSION
 QC. - TRIAXIAL CONSOLIDATED QUICK
 Q. - TRIAXIAL QUICK
 S. - TRIAXIAL SLOW
 7. - UNIT WEIGHT
 K. - PERMEABILITY
 C. - CONSOLIDATION
 CA. - CASING
 WL. - WATER LEVEL IN CASING
 WT. - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. _____

CASING 4 1/2 (STANDARD SAMPLERS TO FIT UNLESS NOTED) INCH

SAMPLER HAMMER WT. 350 DROP 11 1/2 INCHES

JOB_

DATUM —

COMPILED BY...مفتی محمد رفیع

BORING # 12 (G12)

DATE REPORT SEP 1 1959

DATE REPORTED 12/12/2011
BORING DATE 12/12/2011

SAMPLE CONDITION



DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES

C.S. - CHUCK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
D.P. - DRIVE PISTON
T.O. - THIN WALLED OPEN
T.P. - THIN WALLED PISTON

PES
F. S. - FOIL SAMPLE
B. A. - BARREL AUGER
S. A. - SPIRAL AUGER
W. S. - WASHED SAMPLE
R. C. - ROCK CORE

ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST 7. - UNIT WEIGHT
 M. - MECHANICAL ANALYSIS K. - PERMEABILITY
 U. - UNCONFINED COMPRESSION C. - CONSOLIDATION
 Qc. - TRIAXIAL CONSOLIDATED QUICK CA. - CASING
 Q. - TRIAXIAL QUICK WL. - WATER LEVEL IN CASING
 Qs. - TRIAXIAL SLOW WT. - WATER TABLE IN SOIL

SOIL PROFILE				WATER CONTENT W %			PENETRATION TEST RESISTANCE BLOWS PER FOOT		SAMPLES					
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	W. %	W. %	W. %	BLOWS PER FOOT	OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	REMARKS
45.0	W. % 12.1, 45.0	SOFT CLAY		45.0								1	83	45.0
44.0		STRATIFIED CLAY, W. %		44.0								2	8	44.0
43.0		CLAY LAYERS		43.0								3	8	43.0
42.0		SOME CLAY		42.0								4	PUSH	42.0
41.0				41.0								5	PUSH	41.0
40.0		FIRM CLAY		40.0								6	PUSH	40.0
39.0		TILL VARYING		39.0								7	PUSH	39.0
38.0		CLAY, TO		38.0								8	PUSH	38.0
37.0		SANDY		37.0								9	PUSH	37.0
36.0				36.0								10	PUSH	36.0
35.0				35.0								11	100	35.0
34.0				34.0								12	117	34.0
33.0				33.0								13	151	33.0
32.0				32.0								14	160	32.0
31.0				31.0								15	277	31.0
30.0				30.0								17	1012	30.0

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG NO. 1 JOB 57142 BORING 13 (C13)
 CASING 1" 57 (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM CLAY LINE DATE REPORT SEP 11 1957
 SAMPLER HAMMER WT. 175 DROP 15 INCHES COMPILED BY J.C. CHECKED BY J.C. BORING DATE SEP 11 1957

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
D.P. - DRIVE PISTON
T.O. - THIN WALLED OPEN
T.P. - THIN WALLED PISTON

F.S. - FOIL SAMPLE
B.A. - BARREL AUGER
S.A. - SPIRAL AUGER
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

V. - IN-SITU VANE SHEAR TEST
M. - MECHANICAL ANALYSIS
U. - UNCONFINED COMPRESSION
QC. - TRIAXIAL CONSOLIDATED QUICK
Q. - TRIAXIAL QUICK
S. - TRIAXIAL SLOW

Y. - UNIT WEIGHT
K. - PERMEABILITY
C. - CONSOLIDATION
CA. - CASING
WL. - WATER LEVEL IN CASING
WT. - WATER TABLE IN SOIL

SOIL PROFILE

LEVEL DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE
0.00		CLAY		440.0
0.50		CLAY		439.5
1.00		CLAY		439.0
1.50		CLAY		438.5
2.00		CLAY		438.0
2.50		CLAY		437.5
3.00		CLAY		437.0
3.50		CLAY		436.5
4.00		CLAY		436.0
4.50		CLAY		435.5
5.00		CLAY		435.0
5.50		CLAY		434.5
6.00		CLAY		434.0
6.50		CLAY		433.5
7.00		CLAY		433.0
7.50		CLAY		432.5
8.00		CLAY		432.0
8.50		CLAY		431.5
9.00		CLAY		431.0
9.50		CLAY		430.5
10.00		CLAY		430.0
10.50		CLAY		429.5
11.00		CLAY		429.0
11.50		CLAY		428.5
12.00		CLAY		428.0
12.50		CLAY		427.5
13.00		CLAY		427.0
13.50		CLAY		426.5
14.00		CLAY		426.0
14.50		CLAY		425.5
15.00		CLAY		425.0
15.50		CLAY		424.5
16.00		CLAY		424.0
16.50		CLAY		423.5
17.00		CLAY		423.0
17.50		CLAY		422.5
18.00		CLAY		422.0
18.50		CLAY		421.5
19.00		CLAY		421.0
19.50		CLAY		420.5
20.00		CLAY		420.0
20.50		CLAY		419.5
21.00		CLAY		419.0
21.50		CLAY		418.5
22.00		CLAY		418.0
22.50		CLAY		417.5
23.00		CLAY		417.0
23.50		CLAY		416.5
24.00		CLAY		416.0
24.50		CLAY		415.5
25.00		CLAY		415.0
25.50		CLAY		414.5
26.00		CLAY		414.0
26.50		CLAY		413.5
27.00		CLAY		413.0
27.50		CLAY		412.5
28.00		CLAY		412.0
28.50		CLAY		411.5
29.00		CLAY		411.0
29.50		CLAY		410.5
30.00		CLAY		410.0
30.50		CLAY		409.5
31.00		CLAY		409.0
31.50		CLAY		408.5
32.00		CLAY		408.0
32.50		CLAY		407.5
33.00		CLAY		407.0
33.50		CLAY		406.5
34.00		CLAY		406.0
34.50		CLAY		405.5
35.00		CLAY		405.0
35.50		CLAY		404.5
36.00		CLAY		404.0
36.50		CLAY		403.5
37.00		CLAY		403.0
37.50		CLAY		402.5
38.00		CLAY		402.0
38.50		CLAY		401.5
39.00		CLAY		401.0
39.50		CLAY		400.5
40.00		CLAY		400.0
40.50		CLAY		399.5
41.00		CLAY		399.0
41.50		CLAY		398.5
42.00		CLAY		398.0
42.50		CLAY		397.5
43.00		CLAY		397.0
43.50		CLAY		396.5
44.00		CLAY		396.0
44.50		CLAY		395.5
45.00		CLAY		395.0
45.50		CLAY		394.5
46.00		CLAY		394.0
46.50		CLAY		393.5
47.00		CLAY		393.0
47.50		CLAY		392.5
48.00		CLAY		392.0
48.50		CLAY		391.5
49.00		CLAY		391.0
49.50		CLAY		390.5
50.00		CLAY		390.0
50.50		CLAY		389.5
51.00		CLAY		389.0
51.50		CLAY		388.5
52.00		CLAY		388.0
52.50		CLAY		387.5
53.00		CLAY		387.0
53.50		CLAY		386.5
54.00		CLAY		386.0
54.50		CLAY		385.5
55.00		CLAY		385.0
55.50		CLAY		384.5
56.00		CLAY		384.0
56.50		CLAY		383.5
57.00		CLAY		383.0
57.50		CLAY		382.5
58.00		CLAY		382.0
58.50		CLAY		381.5
59.00		CLAY		381.0
59.50		CLAY		380.5
60.00		CLAY		380.0
60.50		CLAY		379.5
61.00		CLAY		379.0
61.50		CLAY		378.5
62.00		CLAY		378.0
62.50		CLAY		377.5
63.00		CLAY		377.0
63.50		CLAY		376.5
64.00		CLAY		376.0
64.50		CLAY		375.5
65.00		CLAY		375.0
65.50		CLAY		374.5
66.00		CLAY		374.0
66.50		CLAY		373.5
67.00		CLAY		373.0
67.50		CLAY		372.5
68.00		CLAY		372.0
68.50		CLAY		371.5
69.00		CLAY		371.0
69.50		CLAY		370.5
70.00		CLAY		370.0
70.50		CLAY		369.5
71.00		CLAY		369.0
71.50		CLAY		368.5
72.00		CLAY		368.0
72.50		CLAY		367.5
73.00		CLAY		367.0
73.50		CLAY		366.5
74.00		CLAY		366.0
74.50		CLAY		365.5
75.00		CLAY		365.0
75.50		CLAY		364.5
76.00		CLAY		364.0
76.50		CLAY		363.5
77.00		CLAY		363.0
77.50		CLAY		362.5
78.00		CLAY		362.0
78.50		CLAY		361.5
79.00		CLAY		361.0
79.50		CLAY		360.5
80.00		CLAY		360.0
80.50		CLAY		359.5
81.00		CLAY		359.0
81.50		CLAY		358.5
82.00		CLAY		358.0
82.50		CLAY		357.5
83.00		CLAY		357.0
83.50		CLAY		356.5
84.00		CLAY		356.0
84.50		CLAY		355.5
85.00		CLAY		355.0
85.50		CLAY		354.5
86.00		CLAY		354.0
86.50		CLAY		353.5
87.00		CLAY		353.0
87.50		CLAY		352.5
88.00		CLAY		352.0
88.50		CLAY		351.5
89.00		CLAY		351.0
89.50		CLAY		350.5
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94.50		CLAY		345.5
95.00		CLAY		345.0
95.50		CLAY		344.5
96.00		CLAY		344.0
96.50		CLAY		343.5
97.00		CLAY		343.0
97.50		CLAY		342.5
98.00		CLAY		342.0
98.50		CLAY		341.5
99.00		CLAY		341.0
99.50		CLAY		340.5
100.00		CLAY		340.0

WATER CONTENT W% NAT. PLW Δ PW.
 PENETRATION TEST RESISTANCE BLOWS PER FOOT 10 20 30 40 50 60 70 80 90 100

SAMPLES

OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECC
			1	17	438.5
			2	42	438.0
			3	34	437.5
			4	0	437.0
			5	2	436.5
			6	2	436.0
			7	5	435.5
			8	2	435.0
			9	2	434.5
			10	2	434.0
			11	42	433.5
			12	52	433.0

END OF PENETRATION TEST. ELEV. 388.3

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. - MAJOR

DRILL RIG. MANUEL (STANDARD SAMPLERS TO FIT UNLESS NOTED)
CASING 1 1/2 INCH

CASING 4 1/2 STANDARD SAM. 4
SAMPLER HAMMER, WT. 35.0 DROP 11 1/2 INCHES

JOB - CIVIL
 DATUM - GEO

DATUM - GEODETIC

DATE 12-1-68 COMPILED BY John De CHECKED BY John De BORING DATE 12-1-68

COMPILED BY W. L. D. L. S. O. N. ABBREVIATIONS

ABBREVIATIONS

V. - IN-SITU VANE SHEAR TEST	7. UNIT WEIGHT
M. - MECHANICAL ANALYSIS	K. PERMEABILITY
U. - UNCONFINED COMPRESSION	C. CONSOLIDATION
QC. - TRIAXIAL CONSOLIDATED	CA. - CASING
Q. - TRIAXIAL QUICK	WL. - WATER LEVEL IN CASING
S. - TRIAXIAL SLOW	WT. - WATER TABLE IN SOIL

SAMPLES

SAMPLES

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. 34210
CASING 1 1/2 (STANDARD SAMPLERS TO FIT UNLESS NOTED)
SAMPLER HAMMER WT. 342 DROP 1 INCHES

JOB _____ BORING # 15 (Gr 15)
 DATUM GROUND DATE REPORT SEPT 24/53
 COMPILED BY NAW CHECKED BY W. H. H. BORING DATE SEPT 24/53

ABBREVIATIONS

ABBREVIATION	
V. IN-SITU VANE SHEAR TEST	T. UNIT WEIGHT
M. MECHANICAL ANALYSIS	K. PERMEABILITY
U. UNCONFINED COMPRESSION	C. CONSOLIDATION
Q.C. TRIAXIAL QUICK	CA. CASING
Q. TRIAXIAL QUICK	WL. WATER LEVEL IN CASING
S. TRIAXIAL SLOW	WT. WATER TABLE IN SOIL

SAMPLE CONDITION

DISTURBED
 FAIR
 GOOD
 LOST

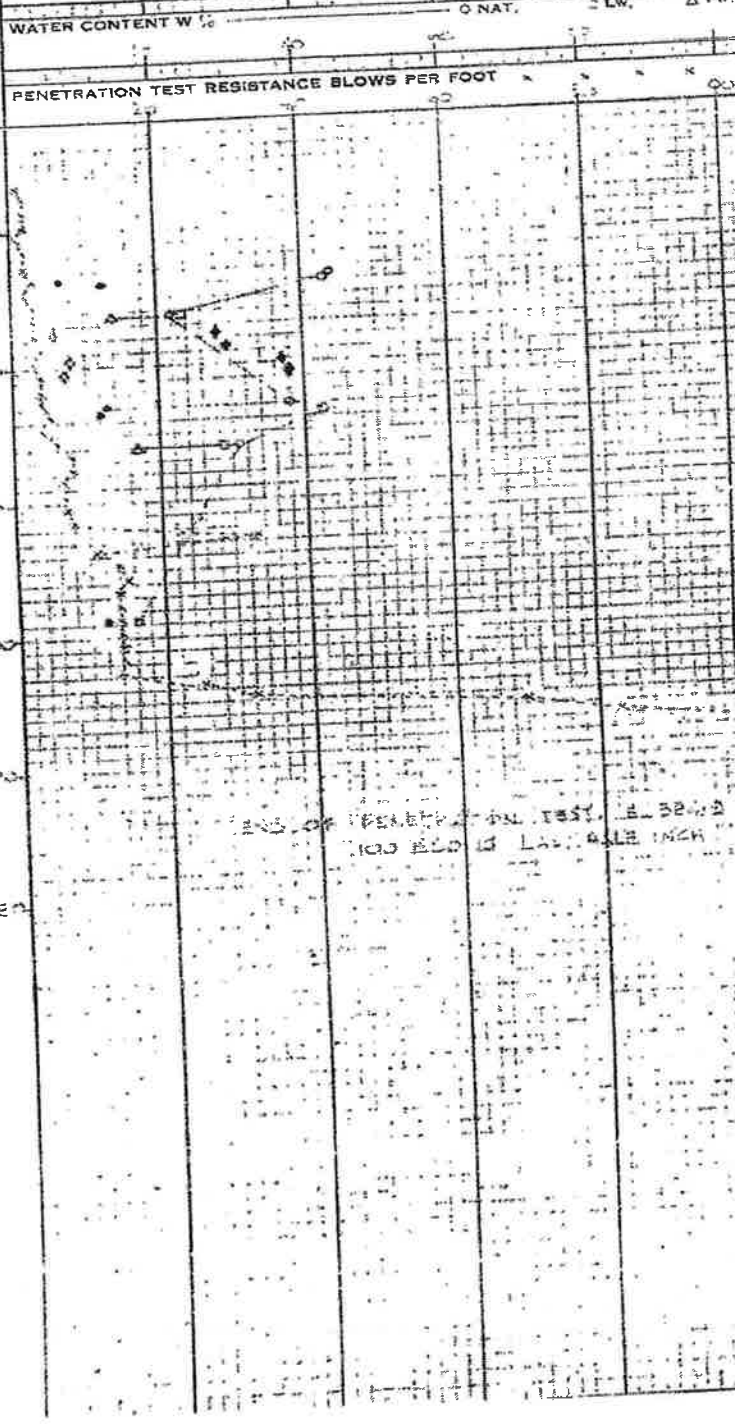
SAMPLE TYPES

C.S. - CHUCK
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
D.P. - DRIVE PISTON
T.O. - THIN WALLED OPEN
T.P. - THIN WALLED PISTON

PES
F. S. - FOIL SAMPLE
B. A. - BARREL AUGER
S. A. - SPIRAL AUGER
W. S. - WASHED SAMPLE
R. C. - ROCK CORE

SOIL PROFILE

LEV. N. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE
1.00 0.00	W. L. @ 2.0
1.00 0.00	W. L. @ 2.0	...		
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SAMPLES

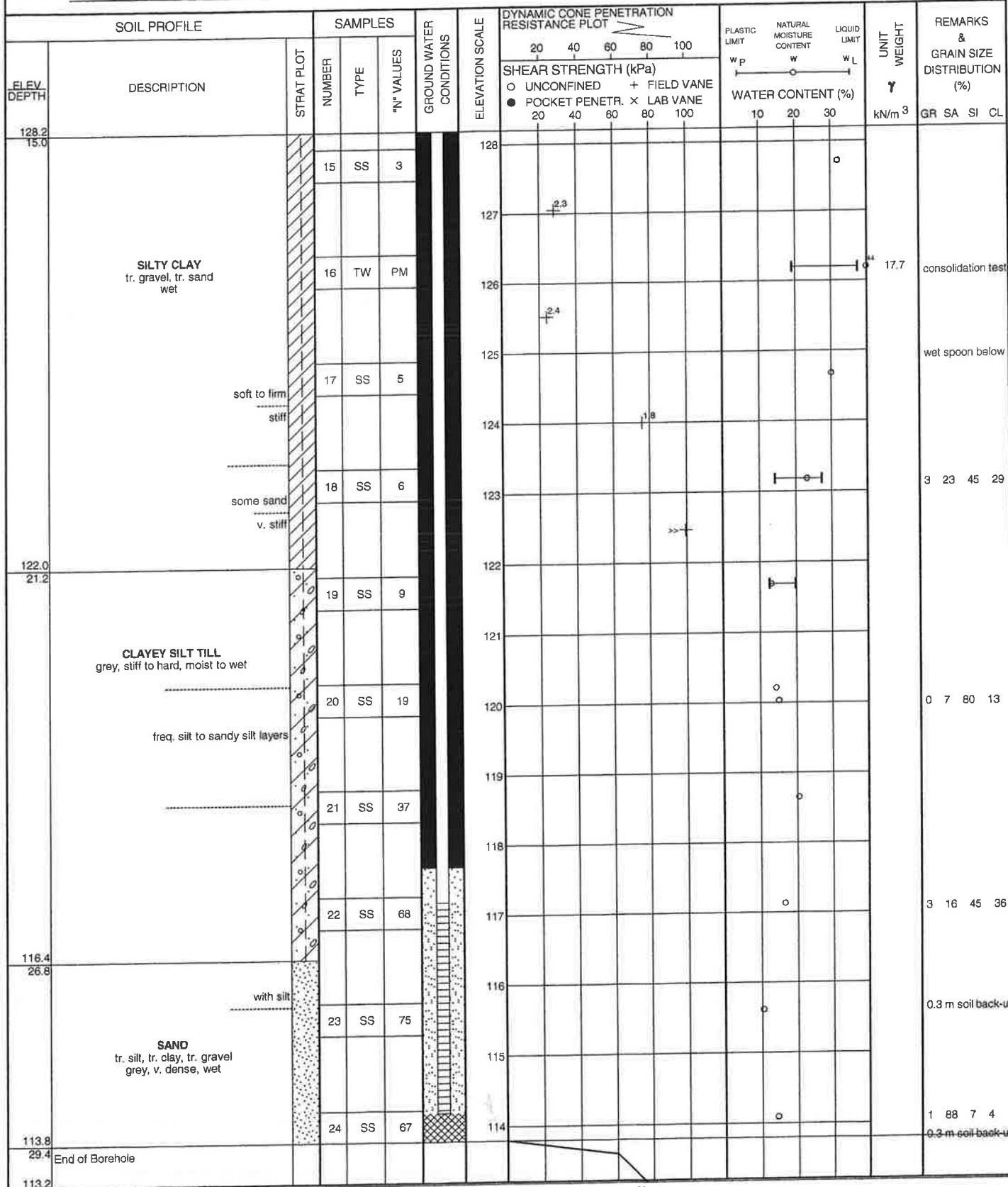
OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	RECORD
					57
					58
					59
					60
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					68
					69
					70
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					97
					98
					99
					100

RECORD OF BOREHOLE No E1

2 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-E Ramp of Leslie Street to 401 (Northing 4847320.9 & Easting 315808.7) ORIGINATED BY RK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers, DCPT COMPILED BY SK
 DATUM Geodetic DATE 1/4/2010 CHECKED BY ZO



Continued Next Page

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

RECORD OF BOREHOLE No E1

3 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-E Ramp of Leslie Street to 401 (Northing 4847320.9 & Easting 315808.7) ORIGINATED BY RK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers, DCPT COMPILED BY SK
 DATUM Geodetic DATE 1/4/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80
113.2 30.0	Dynamic Cone Penetration Test															
112.1 31.1	End of DCPT Dynamic Cone Penetration Test (DCPT) performed in the borehole from 29.6 to 31.1 m Piezometer installed to 29.0 m depth Piezometer water level records : Jan. 5, 2010 20.7 m Jan. 6, 2010 17.7 m Jan. 7, 2010 14.3 m Feb. 4, 2010 7.1 m															

+ 3 . × 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No E2

1 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-E Ramp of Leslie Street to 401 (Northing 4847329.4 & Easting 315835.3) ORIGINATED BY RK
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers, DCPT COMPILED BY SK
DATUM Geodetic DATE 1/6/2010 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE	WATER CONTENT (%)					
140.4	GROUND SURFACE													
0.0	tr. rootlets, some gravel		1	SS	56		140							
	v. dense compact		2	SS	27		139							
	v. loose to loose		3	SS	5		138							
	FILL: SILTY SAND tr. gravel, tr. clay brown, moist		4	SS	4		137							1 53 37 9
	compact to dense		5	SS	21		136							spoon wet
			6	SS	34		135							spoon wet
135.3			7	SS	11		134							spoon wet 0 12 79 9
5.1	SILT some fine sand grey, dilatant, loose to compact, wet		8	SS	8		133							spoon wet
			9	SS	14		132							
			10	SS	9		131							0 6 35 59
132.2							130							
8.2	SILTY CLAY tr. sand, tr. gravel grey, firm to stiff, wet		11	SS	2		129							
			12	TW	PM		128							
			13	SS	2		127							
	freq. sand zones		14	SS	3		126							
125.4														

Continued Next Page

+ 3 . × 3

Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No E2

2 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-E Ramp of Leslie Street to 401 (Northing 4847329.4 & Easting 315835.3) ORIGINATED BY RK
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers, DCPT COMPILED BY SK
DATUM Geodetic DATE 1/6/2010 CHECKED BY ZO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH (kPa)					
125.4 15.0	SILTY CLAY tr. sand, tr. gravel firm to stiff some sand tr. to some sand and gravel		15	TW	PM								
124			16	SS	4								
123			17	SS	8								
122			18	SS	7								
119.9 20.5	CLAYEY SILT TILL with sand grey, stiff to hard, moist to wet freq. silt to sandy silt layers		19	SS	27								
118			20	SS	40								
117			21	SS	84								
116			22	SS	12								
115.4 25.0	SAND tr. silt, tr. gravel grey, compact, wet compact												
114													
111.6 28.8	End of borehole and DCPT water level in open borehole @ 19.8 m (not stabilized)* upon completion before auger pull out. water level in borehole @ 8.2 m after auger pull out and caved in @ 8.5 m												

Continued Next Page

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

GWP	<u>2008-E-0012</u>	LOCATION	<u>N-E Ramp of Leslie Street to 401 (Northing 4847329.4 & Easting 315835.3)</u>	ORIGINATED BY	<u>RK</u>
DIST	<u> </u> HWY <u>401</u>	BOREHOLE TYPE	<u>Hollow Stem Augers, DCPT</u>	COMPILED BY	<u>SK</u>
DATUM	<u>Geodetic</u>	DATE	<u>1/6/2010</u>	CHECKED BY	<u>ZO</u>

[illegible] $+^3 \times^3$

Numbers refer to Sensitivity

20
15--5
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No N1

1 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847431.6 & Easting 315689.2) ORIGINATED BY RK
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
DATUM Geodetic DATE 11/23/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR.	+ FIELD VANE × LAB VANE						
146.9	GROUND SURFACE						20 40 60 80 100	10 20 30							
0.0	230 mm ASPHALT 0.2 m FILL : SAND 200 mm CONCRETE			AS											
			1	SS	100/ 23 cm										
	gravelly		2	SS	45		146								
	some clay		3	SS	49		145							auger grinding @ 2.0 m	
			4	SS	54		144								
			5	SS	39		143							11 40 (49)	
	FILL: SILTY SAND tr. to some gravel compact to v. dense		6	SS	32		142								
			7	SS	36		141								
			8	SS	33		140								
	cobbles		9	SS	19		139								
			10	SS	21		138								
	tr. rootlets, cobbles		11	SS	58		137								
	moist wet		12	SS	10		136							3 80 (17)	
	dark grey tr. to some organics tr. rootlets						135								
136.1			13	SS	20		134							spoon wet	
10.8	tr. organics						133								
	SAND some silt grey, compact, wet						132								
135.0			14	SS	40									spoon wet 0 19 65 16 non-plastic (LL=17.2%)	
11.9	SILT some sand, some clay grey, dilatant, dense, wet														
134.1			15	SS	6										
12.8	SILTY CLAY tr. to some sand, tr. gravel grey, stiff, wet														
131.9															

Continued Next Page

+ 3 x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No N1

2 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847431.6 & Easting 315689.2) ORIGINATED BY RK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
 DATUM Geodetic DATE 11/23/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
131.9 15.0	SILTY CLAY grey, wet							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE					
			16	SS	3		131	2.8 3.1						
							130							1 14 34 51
							129	2.5						
			17	TW	PM		128							
							127	2.2						
			18	SS	10		126							
							125							
			19	SS	6		124							
							123							
121.8 25.1	CLAYEY SILT TILL some sand grey, hard, moist		20	TW	PM		122							
			21	SS	8		121							
							120							
			22	SS	92		119							1 17 50 32
	SAND some silt, some gravel, tr. clay grey, v. dense, wet						118							
			23	SS100 / 25 cm			117							
			24	SS100 / 29 cm										14 68 12 6

Continued Next Page

+ 3, × 3

Numbers refer to Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

TRANETOB01245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N1

3 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847431.6 & Easting 315689.2) ORIGINATED BY RK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
 DATUM Geodetic DATE 11/23/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
116.9 30.0	SAND some silt, some gravel, tr. clay grey, v. dense, wet		25	SS 100/10 mm			116							
116.3 30.6	End of Borehole water level @ 17.7 m (not stabilized)* upon completion before auger pull out borehole caved-in @ 15.2 m upon completion after auger pull out													

+ 3, x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No N2

1 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847425.8 & Easting 315670.7) ORIGINATED BY RK
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
DATUM Geodetic DATE 11/16/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE	WATER CONTENT (%)					
142.3 0.0	GROUND SURFACE													
	1 cm Topsoil		1	SS	58		142							
	some fragments of concrete and asphalt		2	SS	35		141							
	FILL: SILTY SAND tr. to some gravel, some clay brown, compact to dense, moist		3	SS	13		140							22 50 (28)
139.8 2.6			4	SS	18		139							
	some clay		5	SS	47		138							
	SAND tr. to some gravel, some silt brown, compact to dense, wet		6	SS	49		137							spoon wet 0 87 (13)
137.2 5.1			7	SS	22		136							spoon wet
	SILT tr. to some fine sand, tr. to some clay dilatant, loose to compact, wet		8	SS	15		135							
	brown		9	SS	9		134							0 7 88 5
	grey						133							
	some clay		10	SS	28		132							
134.0 8.3							131							
	SILTY CLAY tr. to some sand, tr. gravel grey, wet		11	SS	2		130							3 23 38 36
			12	TW	PM		129							
			13	SS	3		128							
			14	TW	PM									
127.3														

Continued Next Page

+ 3 × 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

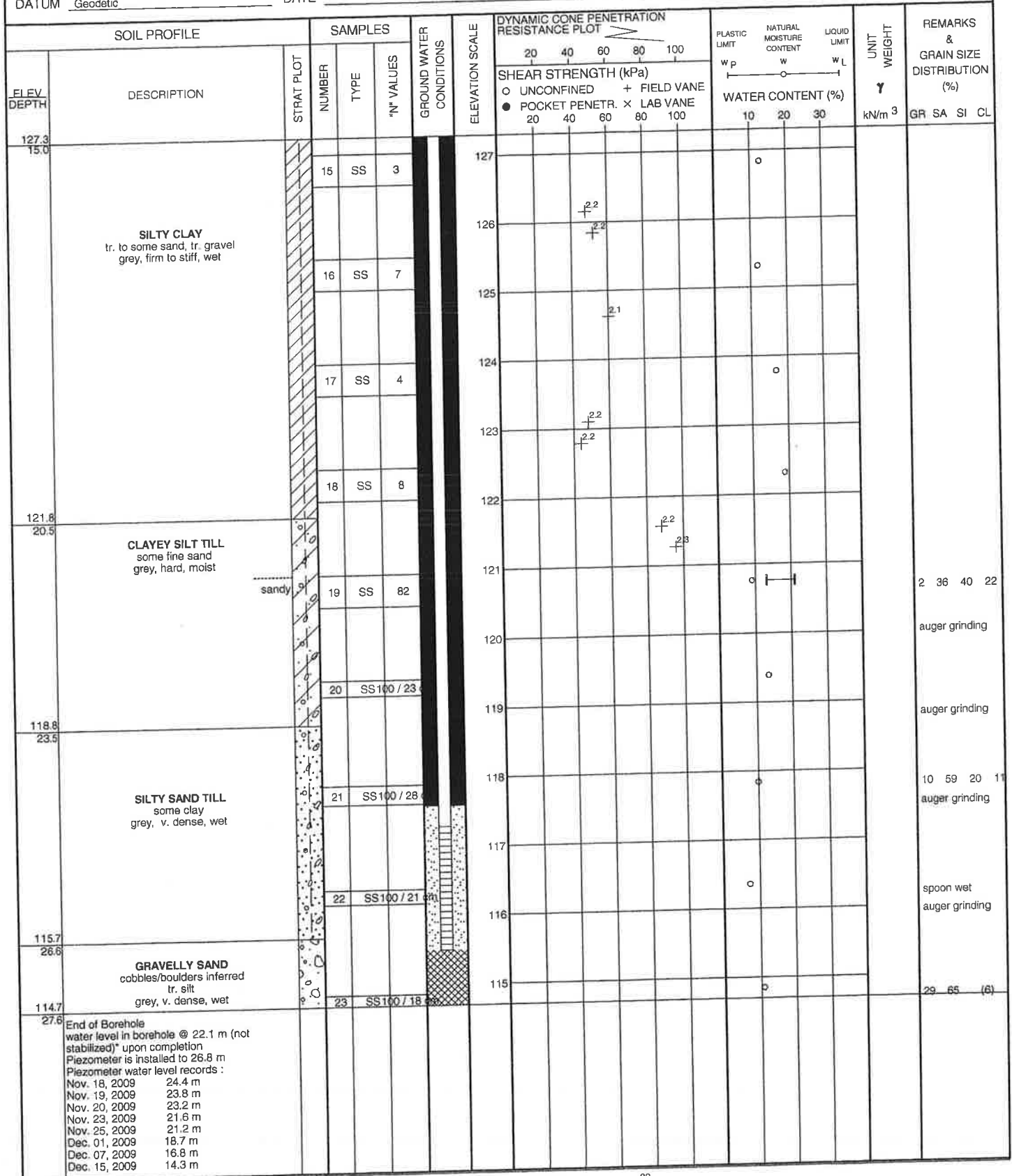
TRANETOBO1245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N2

2 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847425.8 & Easting 315670.7) ORIGINATED BY RK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
 DATUM Geodetic DATE 11/16/2009 CHECKED BY ZO



TRANETOB01245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N2

3 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847425.8 & Easting 315670.7) ORIGINATED BY RK
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
 DATUM Geodetic DATE 11/16/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
112.3	Dec. 16, 2009 14.2 m Dec. 21, 2009 13.0 m Dec. 22, 2009 13.0 m Jan. 06, 2010 10.1 m Feb. 04, 2010 6.4 m																

+³ . x³

Numbers refer to
Sensitivity

20
15
10

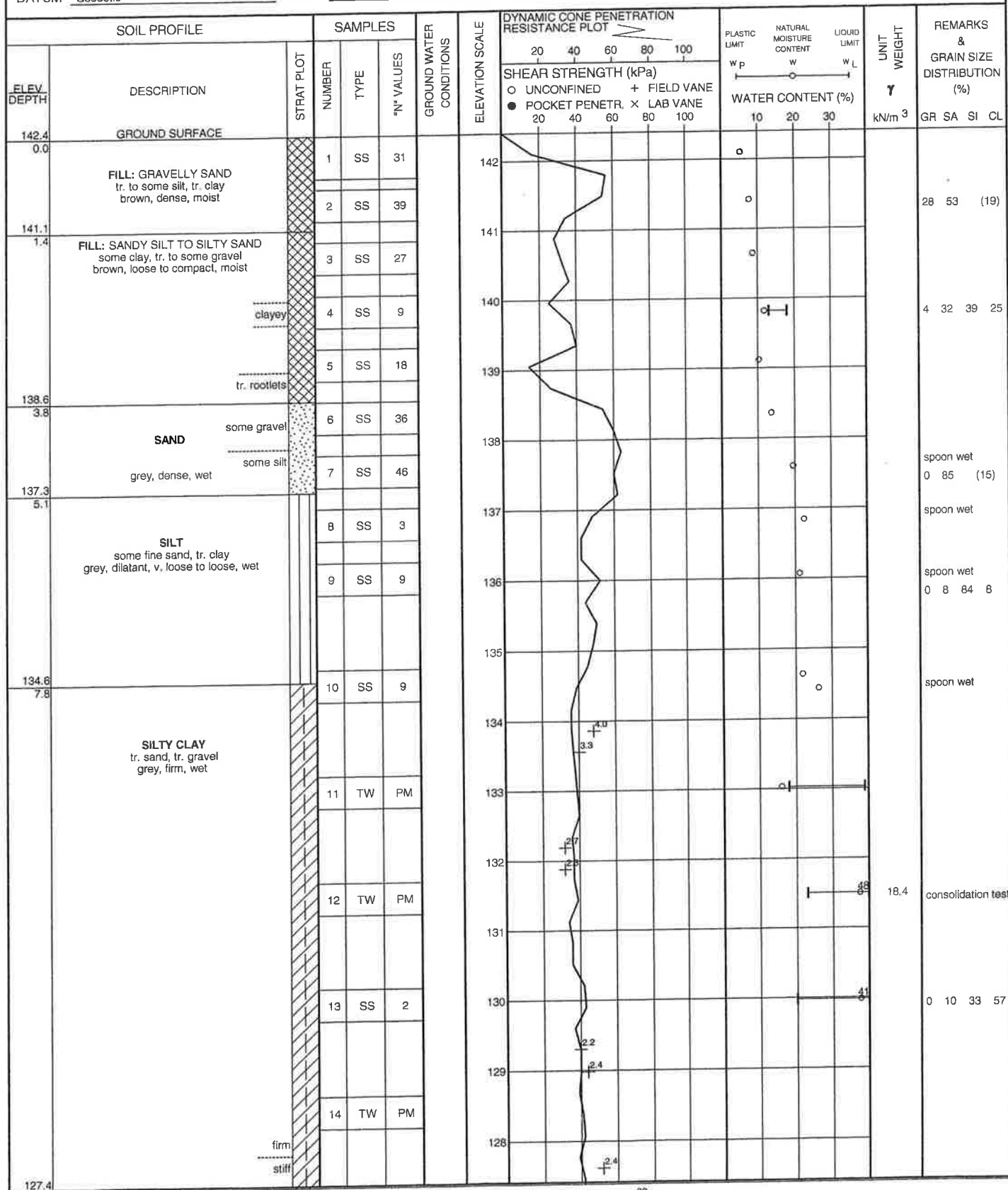
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No N3

1 OF 2

METRIC

GWP	<u>2008-E-0012</u>	LOCATION	<u>N-W Ramp of Leslie Street to 401 (Northing 4847420.9 & Easting 315653.1)</u>	ORIGINATED BY	<u>RK</u>
DIST	<u> </u> HWY <u>401</u>	BOREHOLE TYPE	<u>Hollow Stem Augers</u>	COMPILED BY	<u>SK</u>
DATUM	<u>Geodetic</u>	DATE	<u>11/18/2009</u>	CHECKED BY	<u>ZO</u>



Continued Next Page

 $+^3 \times^3$

Numbers refer to
Sensitivity


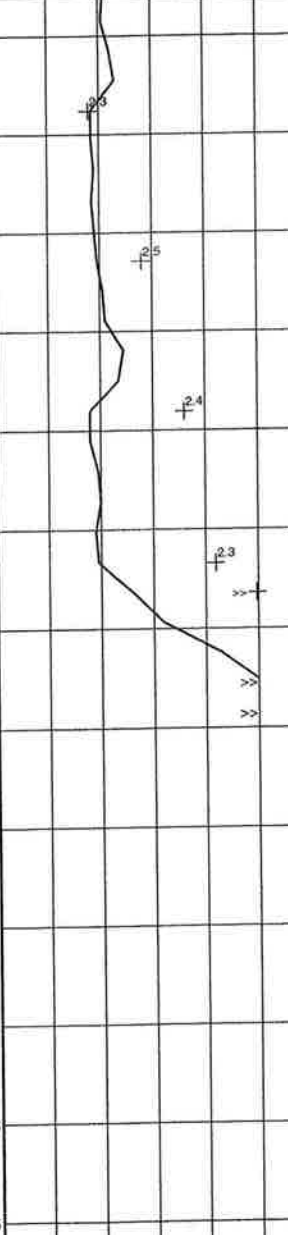
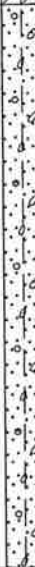
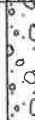
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No N3

2 OF 2

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847420.9 & Easting 315653.1) ORIGINATED BY RK
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK
DATUM Geodetic DATE 11/18/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE		WATER CONTENT (%) w _p w w _L				
127.4 15.0	SILTY CLAY tr. gravel, tr. sand grey, wet		15	SS	3		127						1 24 39 36	non-plastic (LL=15.0%) 10 39 31 20
			16	SS	9		126							
			17	SS	5		125							
			18	SS	6		124							
			19	SS	20		123							
121.5 20.9	SILTY SAND TILL tr. to some clay grey, v. dense, wet		20	SS100 / 25 cm		122						23 38 (39)	auger grinding at 24.4, 25.3 and 25.6 m	
			21	SS100 / 13 cm	121									
			22	SS100 / 20 cm	120									
					119									
					118									
115.8 26.6	GRAVELLY SAND grey, v. dense, wet					117						auger grinding		
							116							
114.5 27.9	End of Borehole water level in borehole @ 16.8 m (not stabilized)* upon completion before auger pull out borehole caved-in @ 4.3 m after auger pull out Dynamic Cone Penetration Test (DCPT) performed adjacent to borehole from ground surface to 22.3 m						115							

+ 3 x 3

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

Appendix B

Test Results

TABLE B-1

COMPARISON OF METHODS OF TESTING OF SAMPLES OF SOIL MATERIAL

C-7142 - NOVEMBER 1959

Sample No.	Depth of specimen, ft.	Moisture content, %	Consistency Limits	Unconfined Compressive Strength, lb./sq. in.	Unconfined Compression Index	Unit wet weight, lb./cu. ft.	Remarks	
4	21'-21'6"	36.8	37.5	18.9	18.6	0.39	0.05	118
19'-21'1") Recovery of 4'6")								
6	26'-26'6"	35.8	37.5	18.9	18.6	0.22	0.16	112
(24'-29')								
11	34'-10"-35' 4"	17.1	21.6	12.2	9.4	0.16	0.20	132
(34'-37'8") (Recovery of 2'4")								
14	50'6"-51'52'8")	45.8	50.0	22.6	27.4	0.29	0.08	103.5
(49'-52'8")								
16	59'6"-60'59'-61'9")	35.6 13.2						

TABLE B-2 c)

TABULATION OF RESULTS OF TESTS ON SAMPLES OF SOIL MARKED

C-7142 - BOREHOLE NO. 11

Sample No.	Depth of specimen ft.	Water content %	Consistency Limits Liquid Plastic Limit Index	Unconfined Compressive Strength as rec'd. strain received	Unit wt. lb/cu.ft.	Remarks
4 (14'-19')	18'-18'6"	33.0	40.7 19.9 20.3	0.13	0.14	115
						Stratified clay, grey; from 14' to 14'10"; numerous inclusions of silt from 14'10" to 16'9"; 3/4" to 2" layers of clay alternating with 1" to 2" layers of silty clay containing numerous inclusions and thin layers of silt, also few small pebbles; from 17'8" to 17'6"; subangular gravel-size particles, sand and silt; from 17'6" to 19': 1/2" to 2" layers of clay alternating with 1/2" to 1-1/4" layers of silty clay containing thin layers of silt.
5 (19'-23')	21'-21'6"	45.5	45.3 20.0 25.3	0.37	0.04	109
						From 19' to 22'4": clay, faintly varved, 1/8" to 1/2" layers of clay alternating with 1/2" to 1" layers of silty clay with inclusions of silt, also 1" layer of silt at 19' from 22'4" to 23'; stratified, numerous thin layers and inclusion of silt.
6 (27'-31')	29'-29'6"	28.0		0.27	0.09	118
						From 27' to 29'6": 1/2" to 1" layers of subangular gravel-size particles sand and silt alternating with 1/4" to 1-1/2" layers of clay; from 29' to 31': subangular gravel-size particles, sand and silt with 1/4" layer silt at 30'

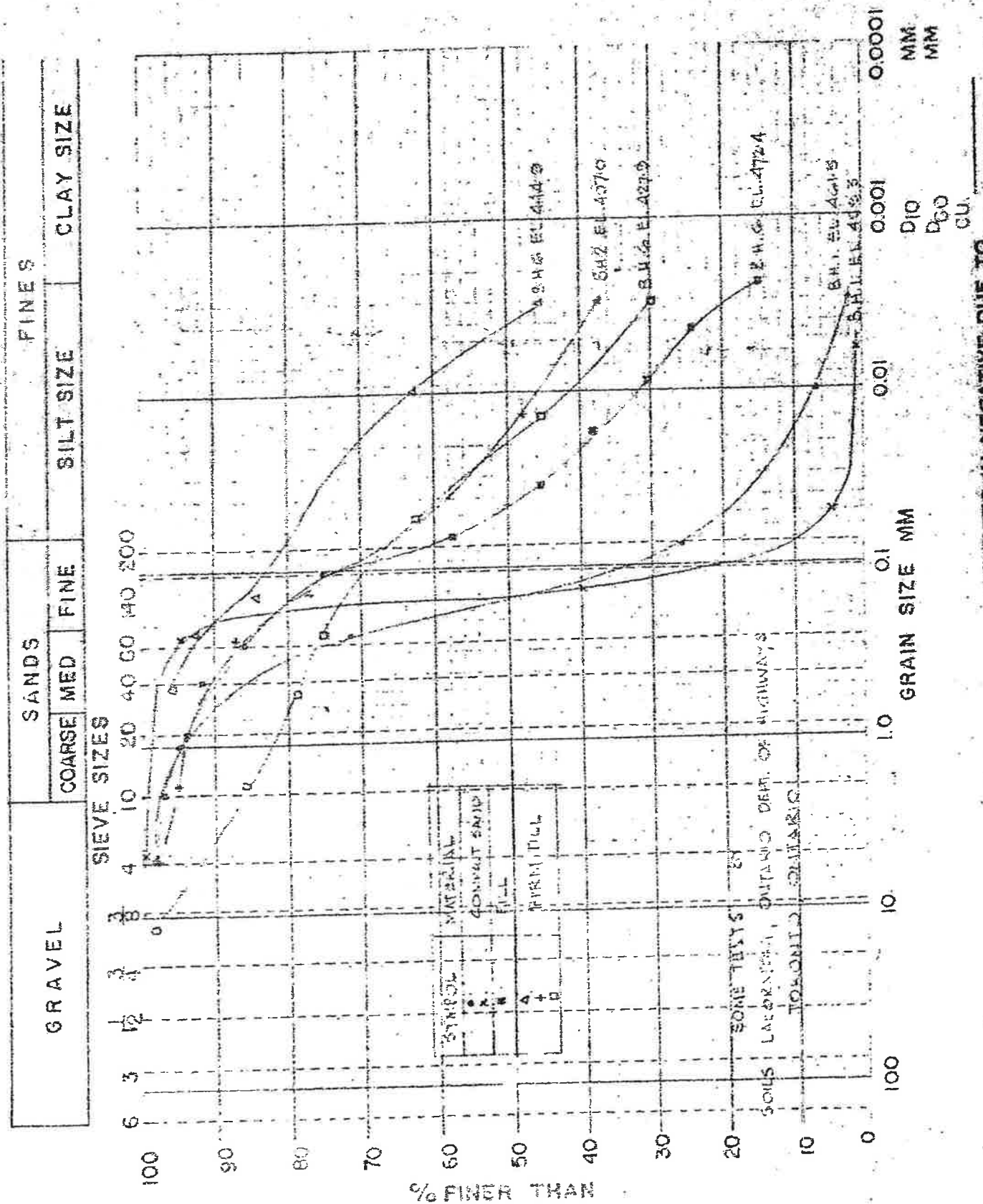
TABLE B-2 (2)

- 2 -

BOREHOLE NO. 11

Sample No.	Depth of specimen, ft.	Water content %	Consistency Limits		Unconfined Compression Strength as received	Corresponding strain	Unit wet weight as received	Remarks
			Limit	Index				
9 (44'-47') (Recovery of 2'6")	45'6"-46'	14.1	15.9	9.6	6.3	0.29	0.29	142
								From 44' to 45'; subangular grain size particles, sand and silt; clay, with 2" layer of clay; From 45' to 46'6": silty clay with numerous inclusions of silt, few small pebbles.

FIGURE 1
PROJECT CT-42



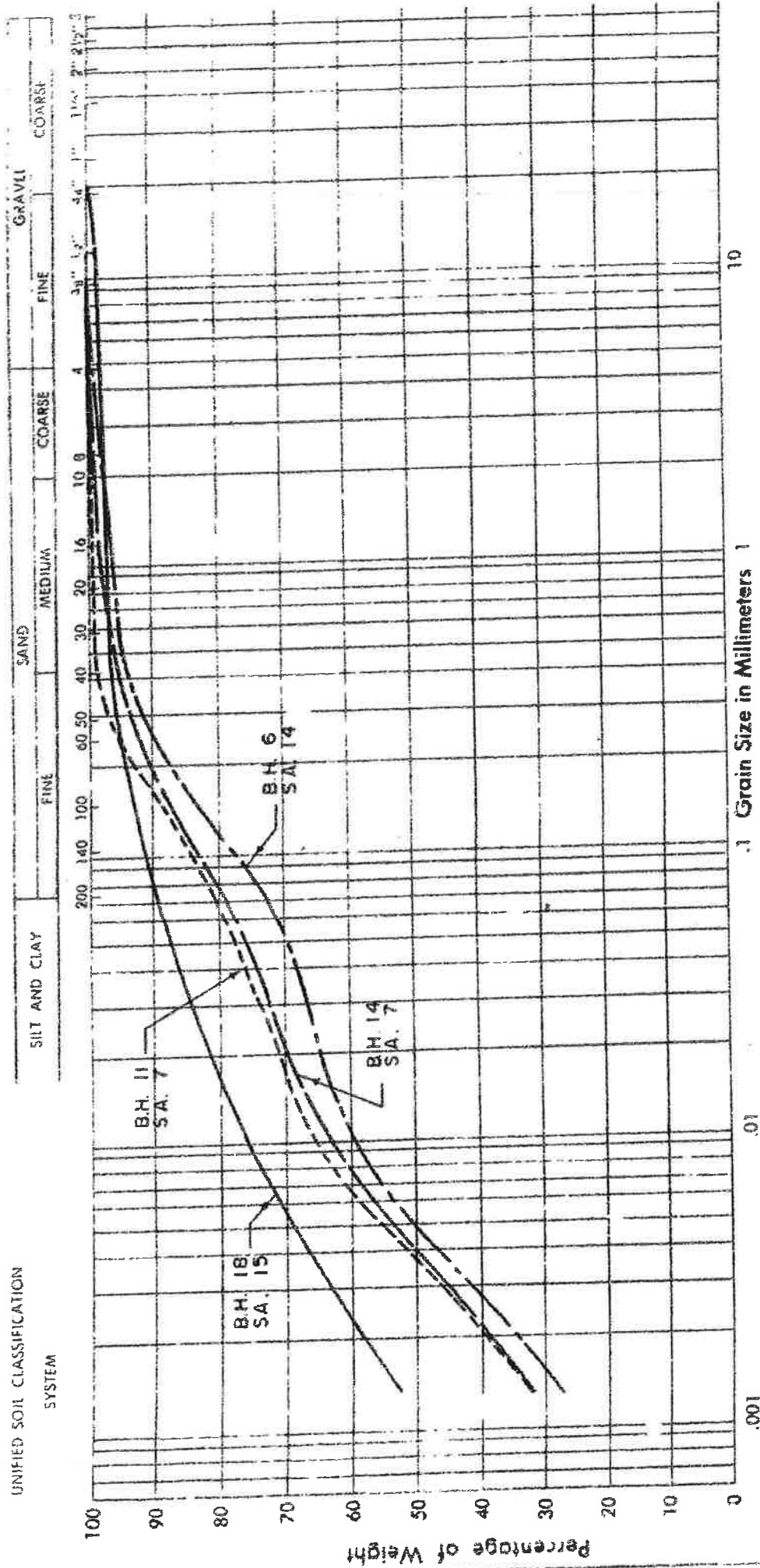
DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

MIT. GRAIN SIZE SCALE

FIGURE B-2

DOMINION SOIL INVESTIGATION LIMITED GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 6-12
Your Ref. NW/P 266-6



PROJECT: HWY. NO. 401 & LESLIE ST.

LOCATION:

BOREHOLE NO. 6 11 14 18

SAMPLE NO.: 14 7 7 15

DEPTH OF SAMPLE:

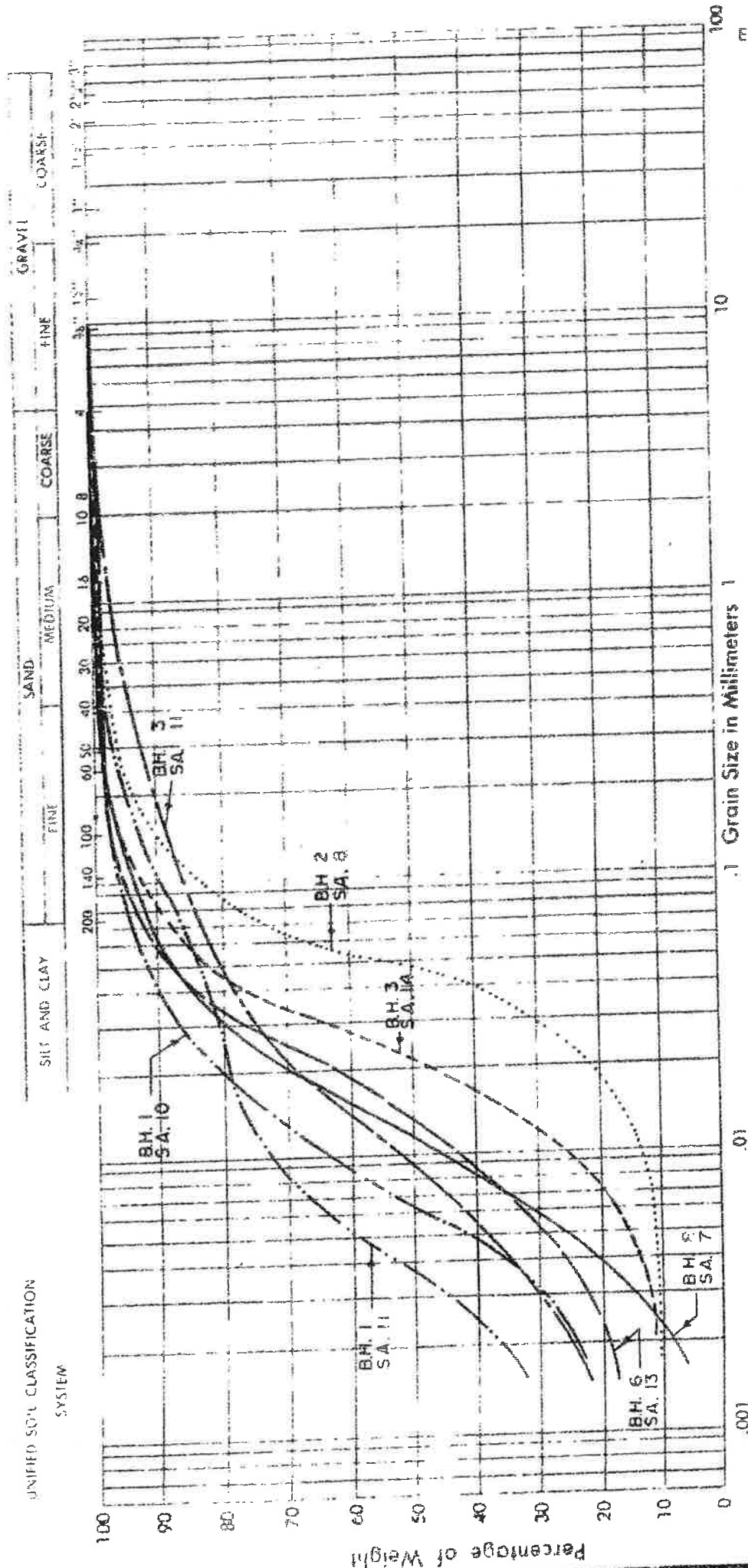
ELEVATION OF SAMPLE:

Classification of Sample and Group Symbol:
CLAYEY SILT to SILTY CLAY
with some FINE SAND

Figure B-3

DOMINION SOIL INVESTIGATION LIMITED GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 6-12-1
Your Ref. No. W.P. 266-61



Enclosure No.

PLASTIC PROPERTIES

LIQUID LIMIT	%
PLASTIC LIMIT	%
PLASTICITY INDEX	%
MOISTURE CONTENT	%
ACTIVITY	%

Classification of Sample and Group Symbol:

SILT with a trace to some CLAY and FINE SAND

COEFFICIENT OF UNIFORMITY
COEFFICIENT OF CURVATURE

PROJECT HWY NO 401 & LESLIE ST.

LOCATION

BOREHOLE NO 1, 2, 3, 3, 6, 8

SAMPLE NO 10, 11, 8, 11, 14, 13, 7

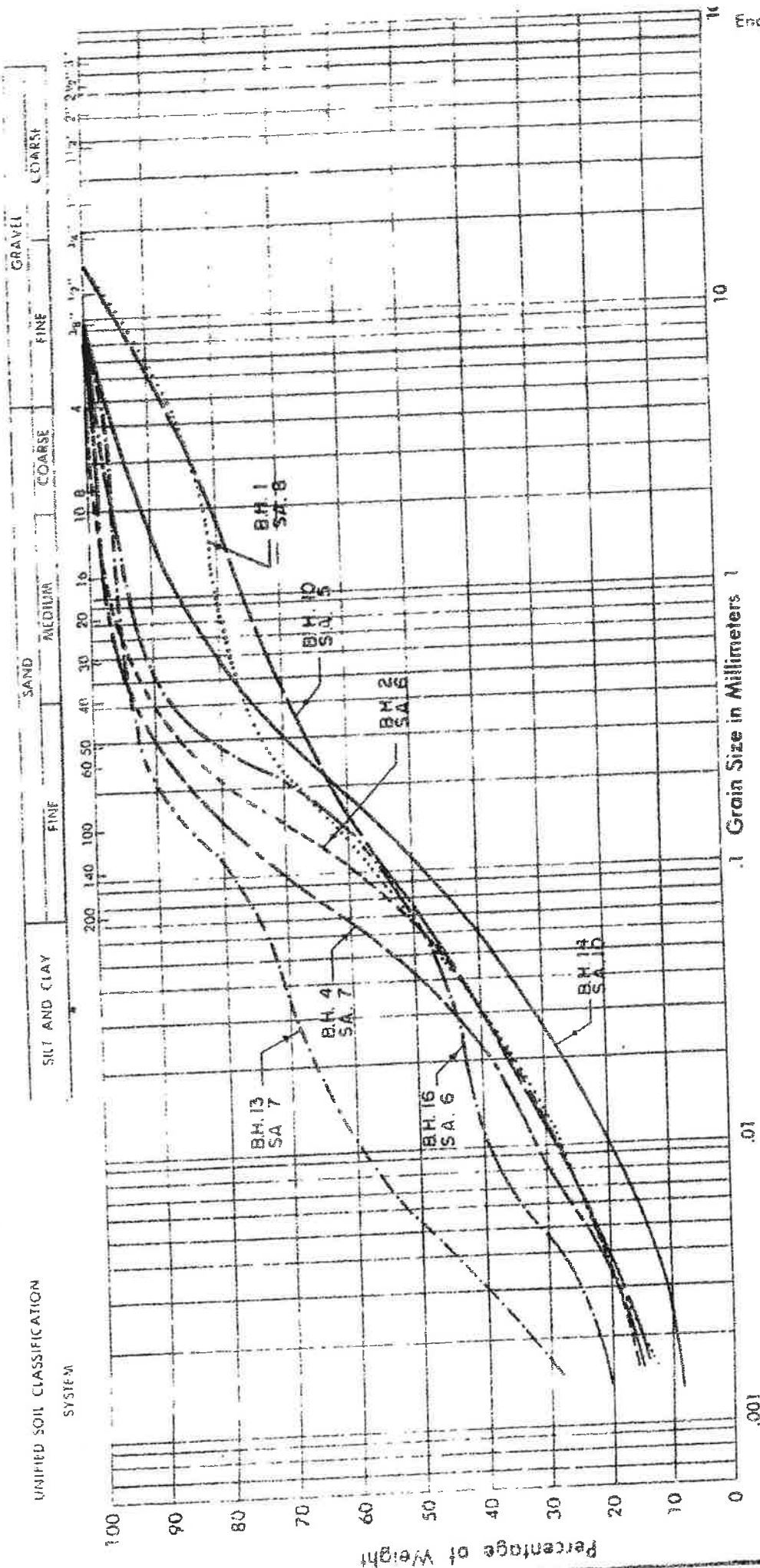
DEPTH OF SAMPLE

ELEVATION OF SAMPLE

FIGURE B-4

DOMINION SOIL INVESTIGATION LIMITED GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 6-12-1
Your Ref. No. W.P. 256-61



Enclosure No.

PLASTIC PROPERTIES:

LIQUID LIMIT	%
PLASTIC LIMIT	%
PLASTICITY INDEX	%
MOISTURE CONTENT	%
ACTIVITY	%

COEFFICIENT OF UNIFORMITY
COEFFICIENT OF CURVATURE

Classification of Sample and Group Symbol:
SAND and SILT with some CLAY
and a trace of GRAVEL

PROJECT: HWY. NO 401 & LESLIE ST.

LOCATION

BOREHOLE NO 1 2 4 10 13 14 16

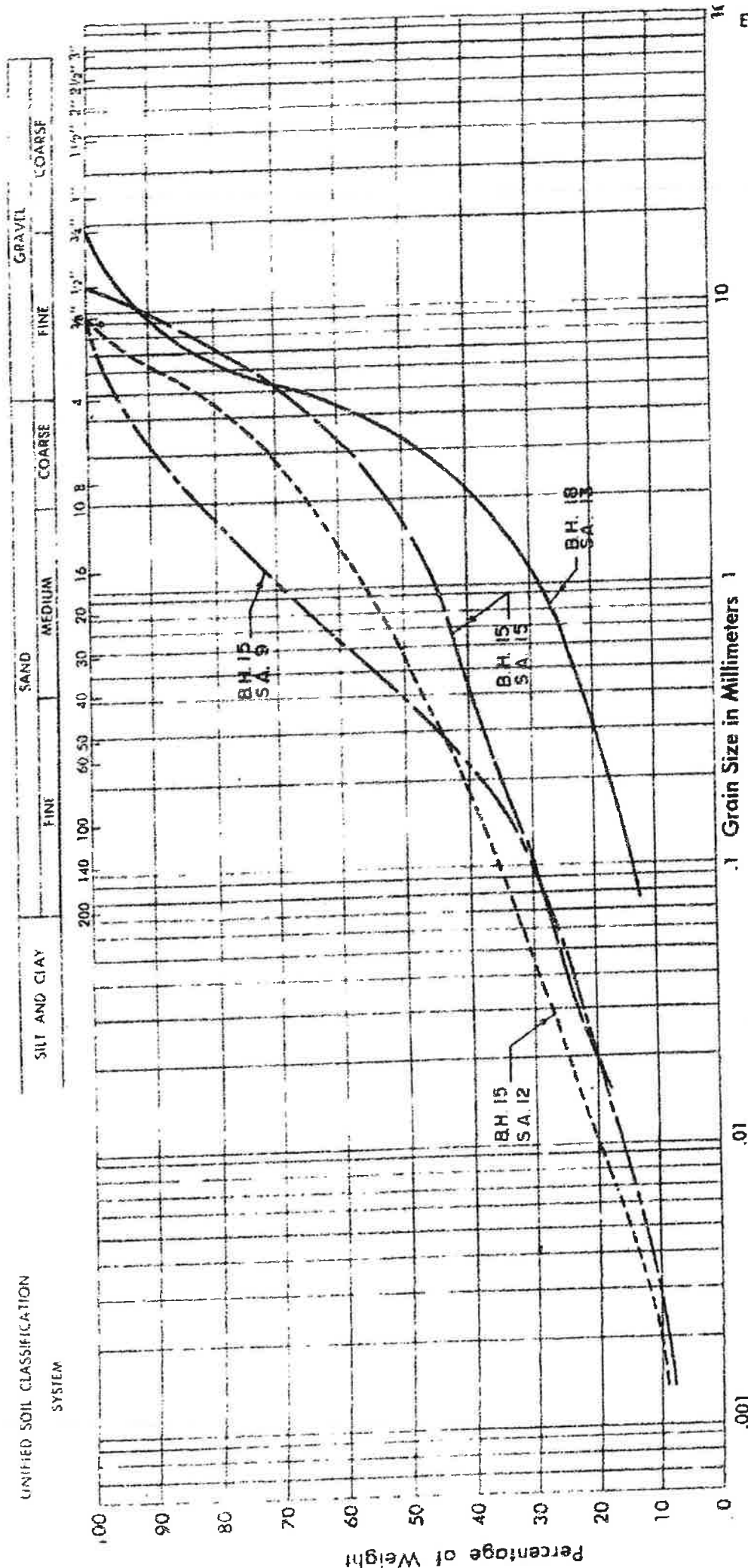
SAMPLE NO 8 6 7 5 7 10 6

DEPTH OF SAMPLE

ELEVATION OF SAMPLE

DOMINION SOIL INVESTIGATION LIMITED
GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 6-12-1
Your Ref. No. W.P. 266-61



COEFFICIENT OF UNIFORMITY
COEFFICIENT OF CURVATURE

PLASTIC PROPERTIES:

LIQUID LIMITED	==	%
PLASTIC LIMIT	==	%
PLASTICITY INDEX	==	%
MOISTURE CONTENT	==	%
ACTIVITY	==	%

PROJECT HWY: NO 401 & LESLIE ST.

2011

BOREHOLE NO. 15 15 15 18

SAMPLE NO	9	12	15	13
1				
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100				

DEPT. OF SAMPLE

ELEVATION OF SAMPLE

Classification of Sample and Group Symbol:
GRAVELLY SAND with some SILT
and a trace of CLAY

Enclosure No.

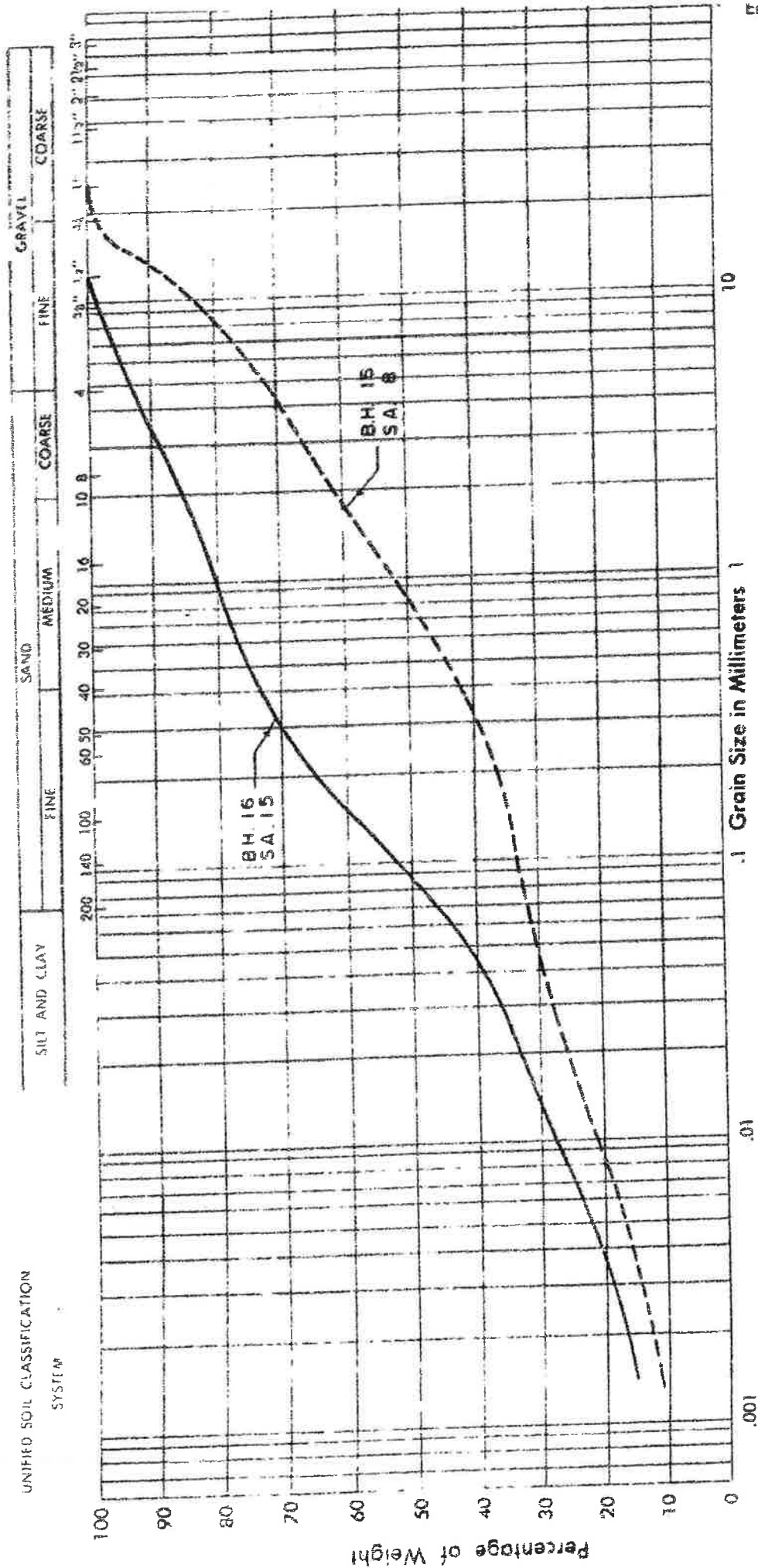
and a trace of CLAY

FIGURE B-6

DOMINION SOIL INVESTIGATION LIMITED

GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO. 6-12-1
Your Ref. No. W.P. 265-61



PLASTIC PROPERTIES:

LIQUID LIMIT %

PLASTIC LIMIT %

PLASTICITY INDEX %

MOISTURE CONTENT %

ACTIVITY %

COEFFICIENT OF UNIFORMITY

COEFFICIENT OF CURVATURE

PROJECT HWY. NO. 401 & LESLIE ST.

LOCATION

BOREHOLE NO. 16 16

SAMPLE NO. 8 15

DEPTH OF SAMPLE

ELEVATION OF SAMPLE

Classification of Sample and Group Symbol:

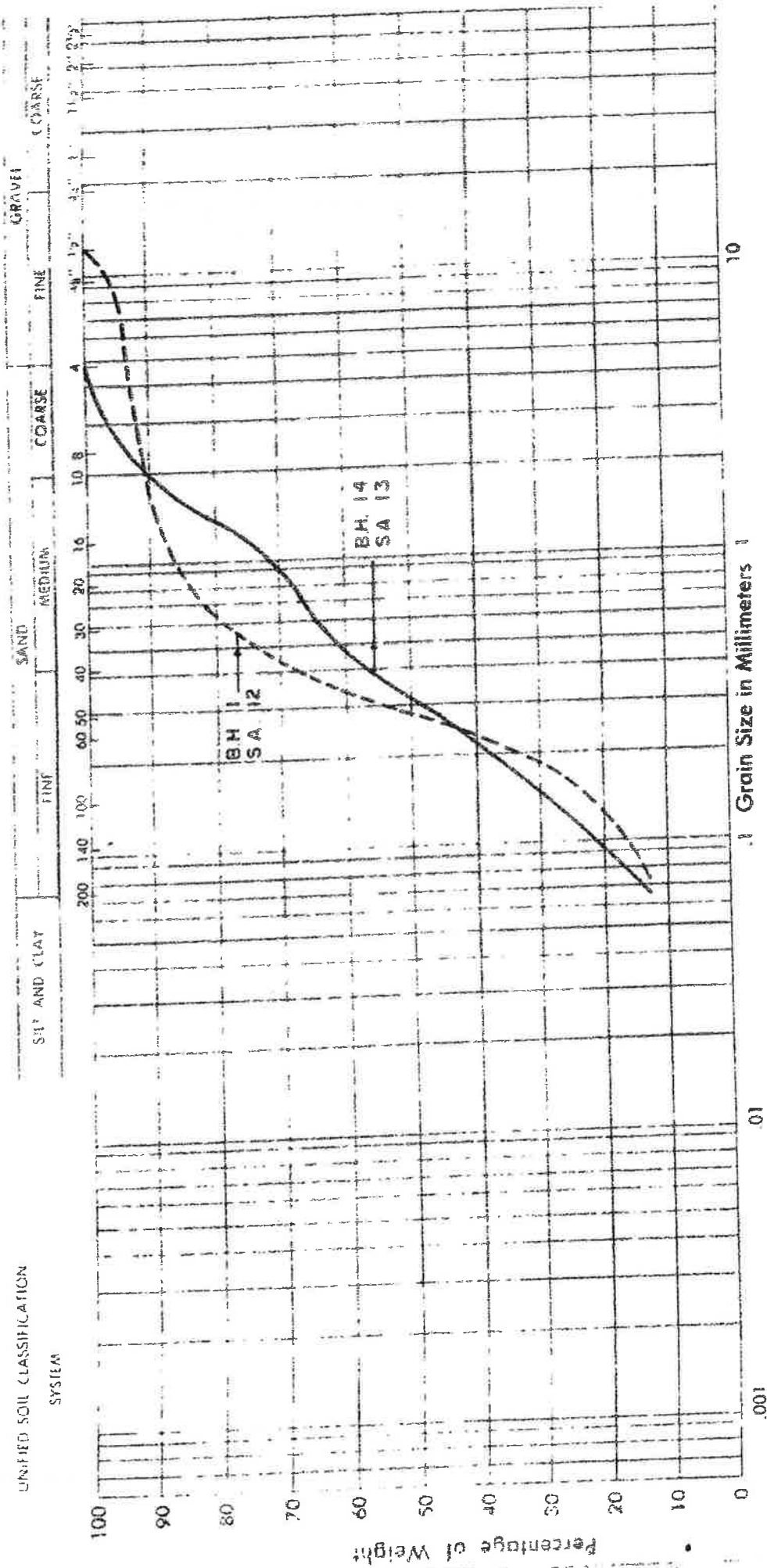
SAND and SILT with some GRAVEL and CLAY

Enclosure No.

Figure B-7

DOMINION SOIL INVESTIGATION LIMITED GRAIN SIZE DISTRIBUTION

OUR REFERENCE NO 6-1
Your Ref. No WP 266-B



PLASTIC PROPERTIES:

LIQUID LIMIT
PLASTIC LIMIT
PLASTICITY INDEX
MOISTURE CONTENT
ACTIVITY

COEFFICIENT OF UNIFORMITY
COEFFICIENT OF CURVATURE

PROJECT HWY NO 401 @ LESLIE ST

LOCATION

BORSHOLE NO 1 14

SAMPLE NO 12 13

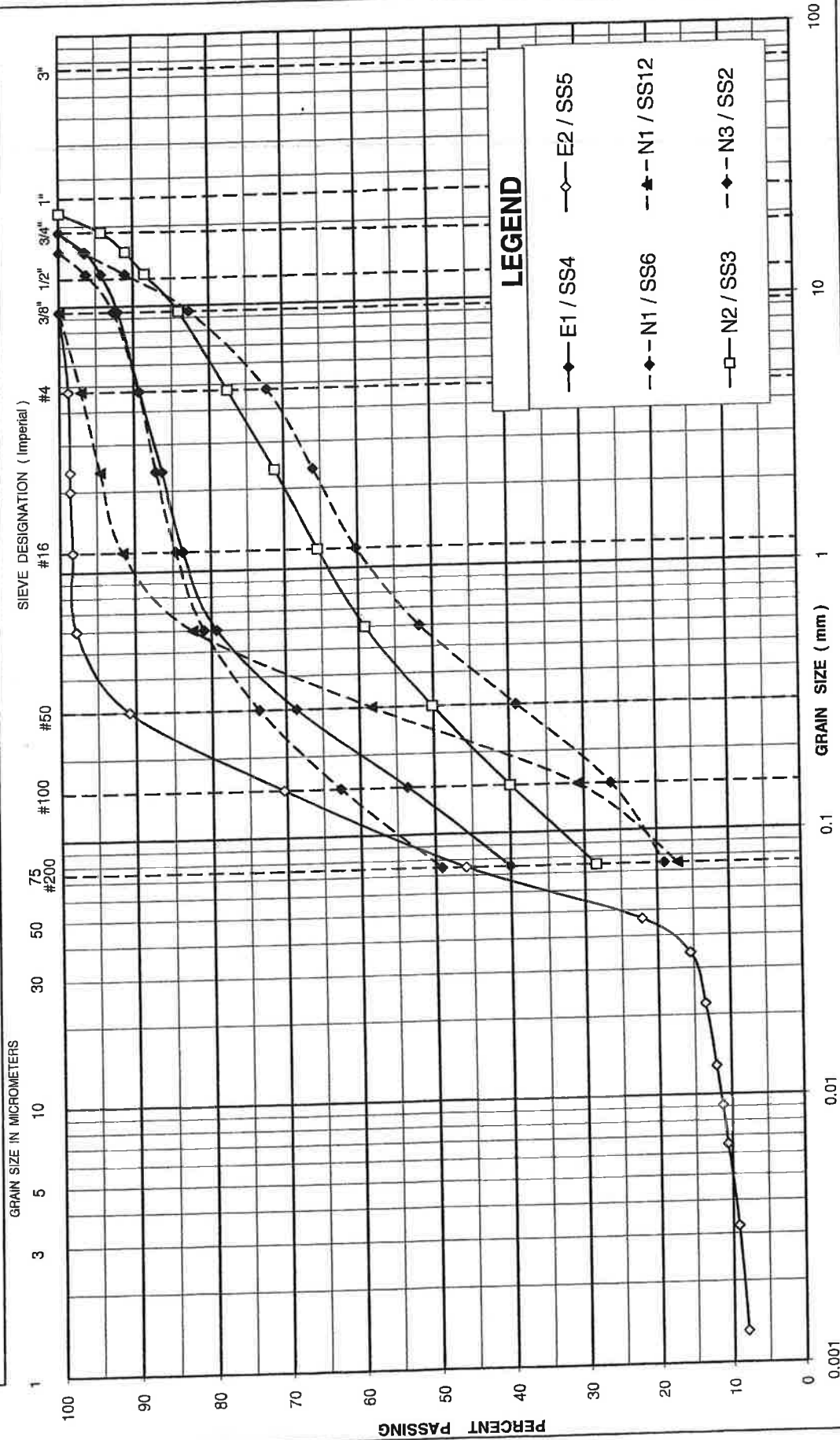
DEPTH OF SAMPLE

ELEVATION OF SAMPLE

Classification of Sample and Group Symbol:
FINE to MEDIUM SAND
with some SILT

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



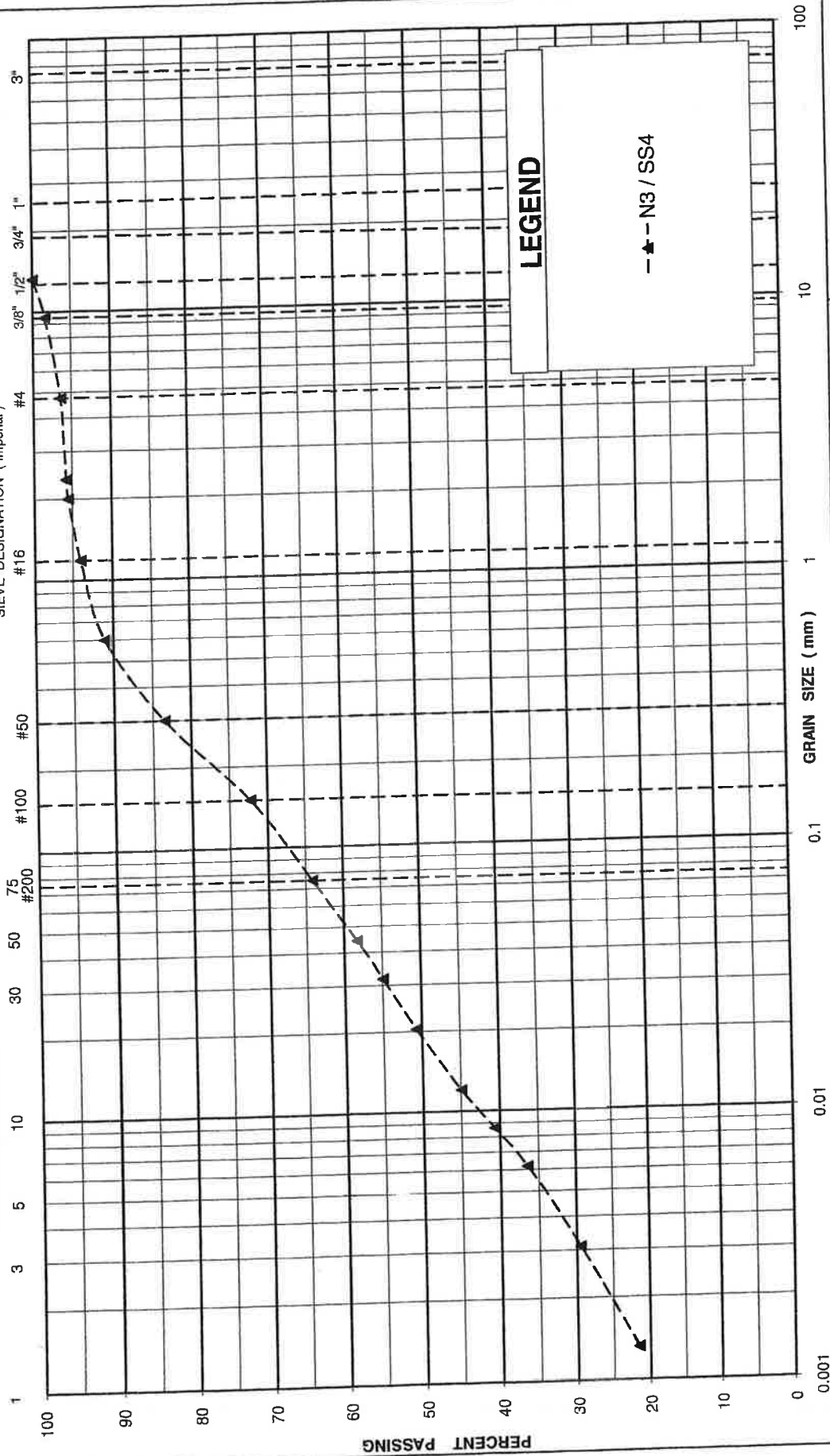
GRAIN SIZE DISTRIBUTION
 FILL: Sandy Silt to Silty Sand

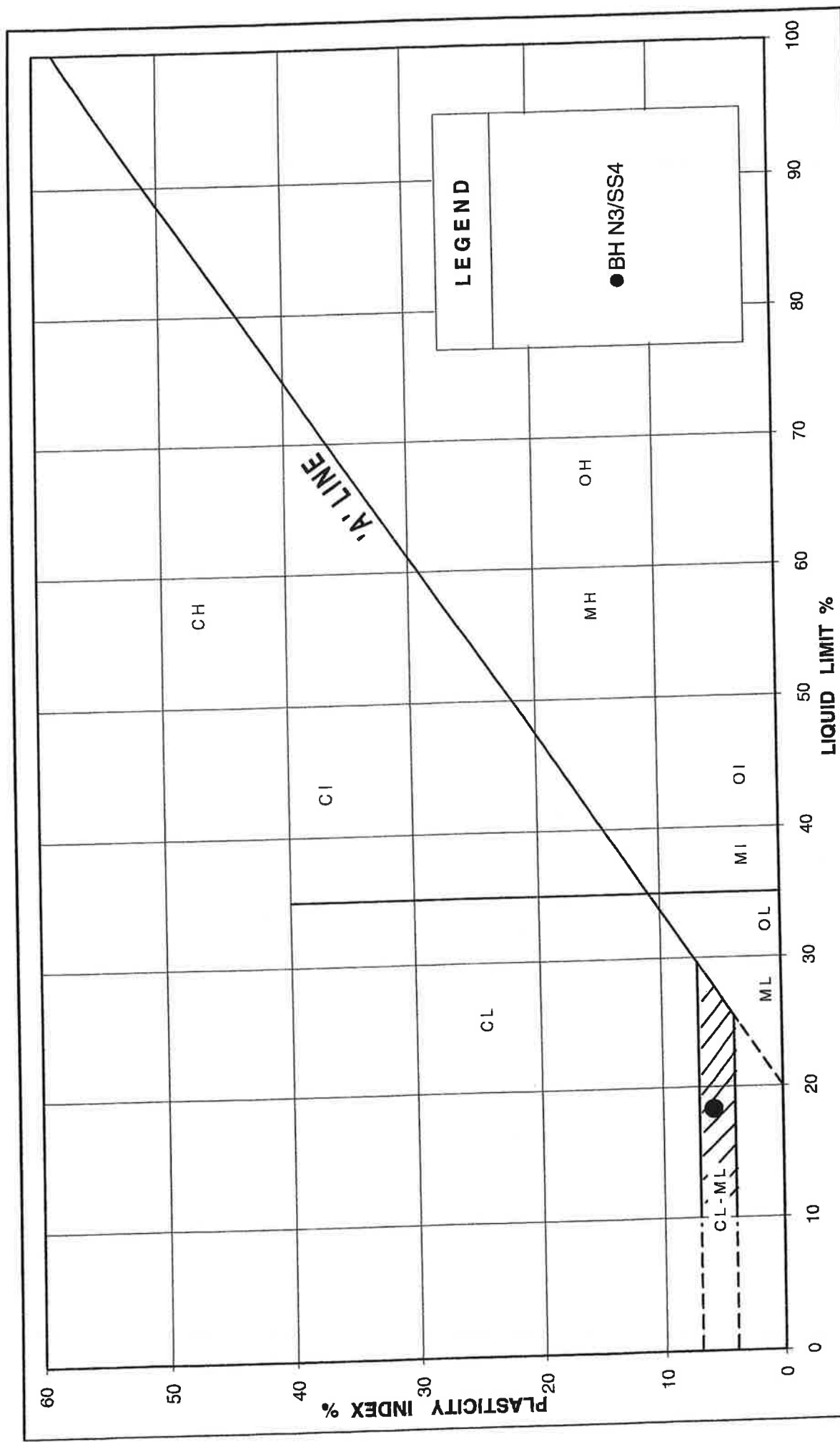
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)





PLASTICITY CHART FILL : sandy silt to silty sand with clay		FIGURE No. B-10
		REF. No. TRANETOB01245AA
		DATE MARCH, 2010

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT

GRAVEL

SAND

Coarse

Fine

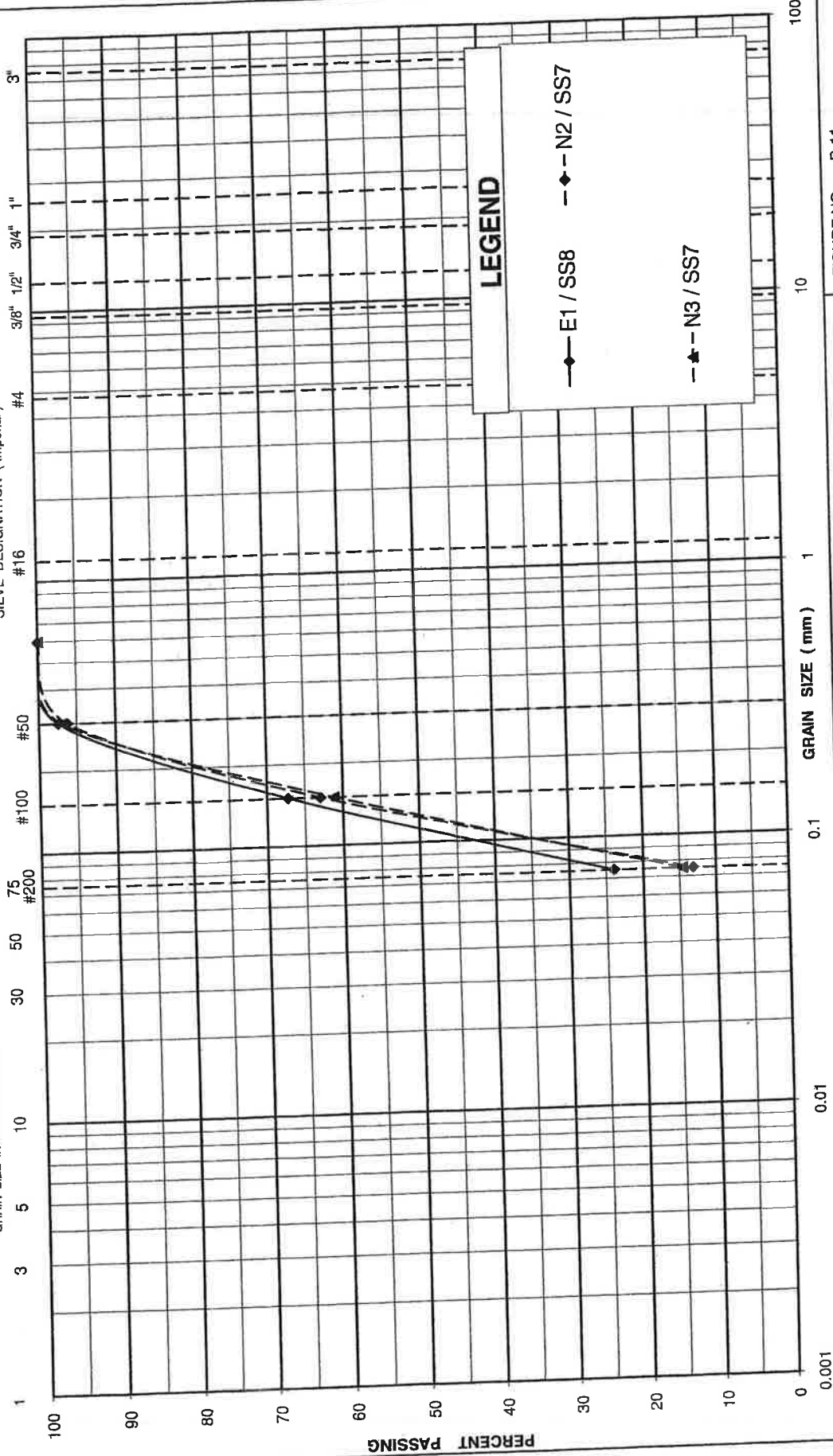
Coarse

Medium

Fine

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



LEGEND

—●— E1 / SS8 --◆-- N2 / SS7

--▲-- N3 / SS7

GRAIN SIZE DISTRIBUTION
SAND

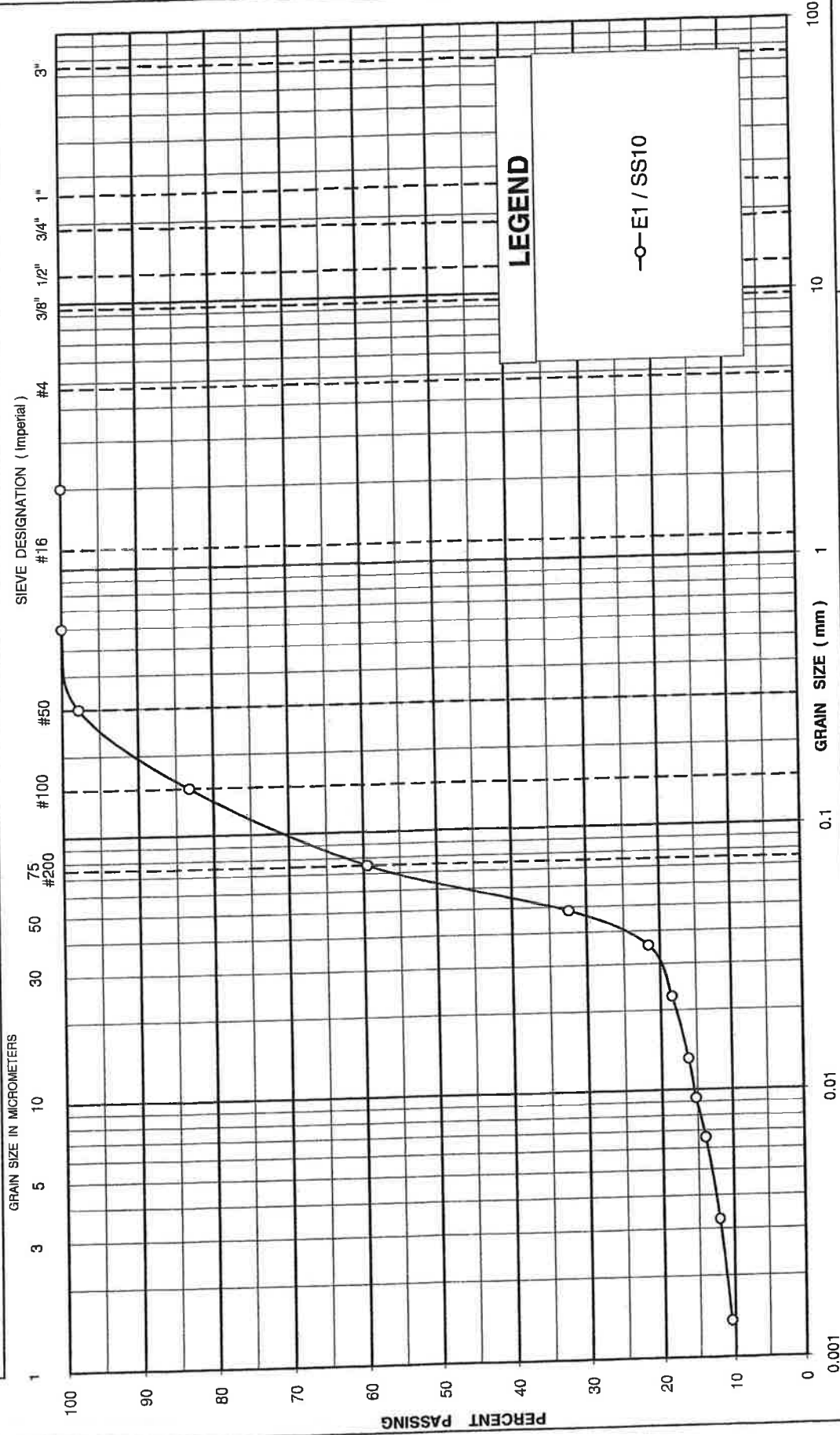
FIGURE NO.: B-11

PROJECT NO: TRANETOBO1245AA

DATE: MAR. 2010

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



GRAIN SIZE DISTRIBUTION SILTY SAND TO SANDY SILT

FIGURE NO.: B-12
PROJECT NO: TRANETOBO1245AA
DATE: MARCH, 2010

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

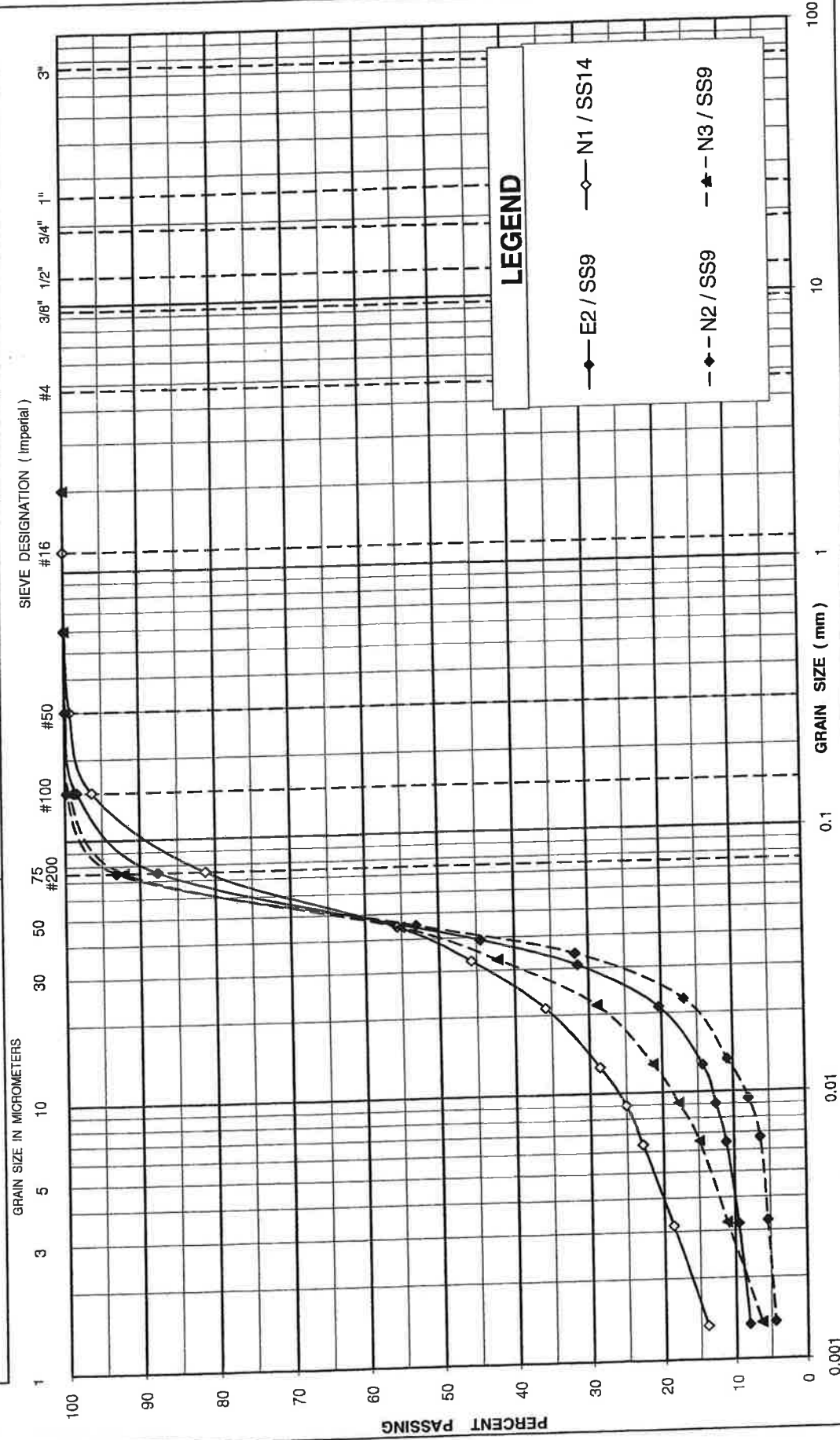


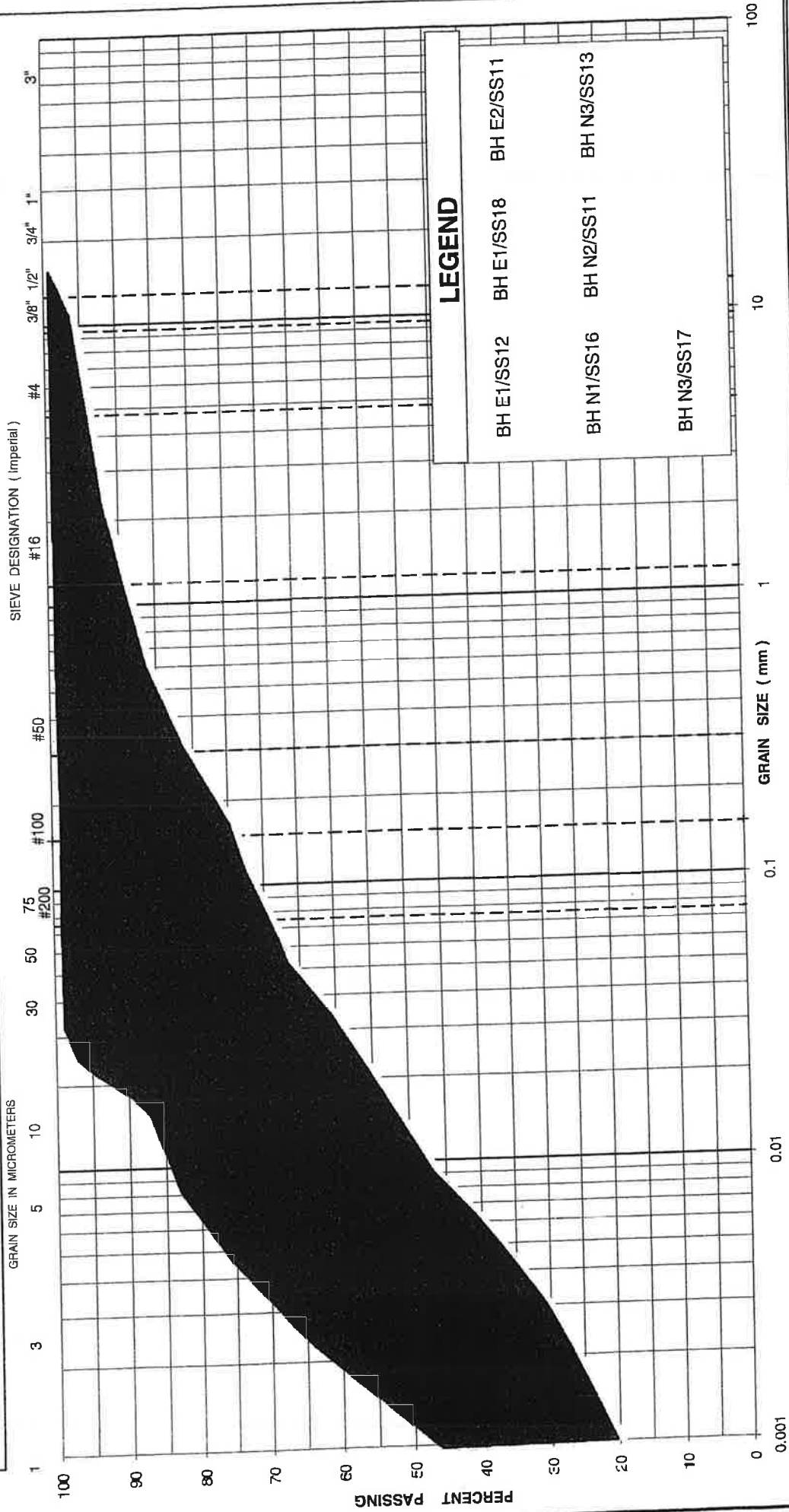
FIGURE NO.: B-13

PROJECT NO: TRANETOBO1245AA

DATE: MARCH, 2010

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

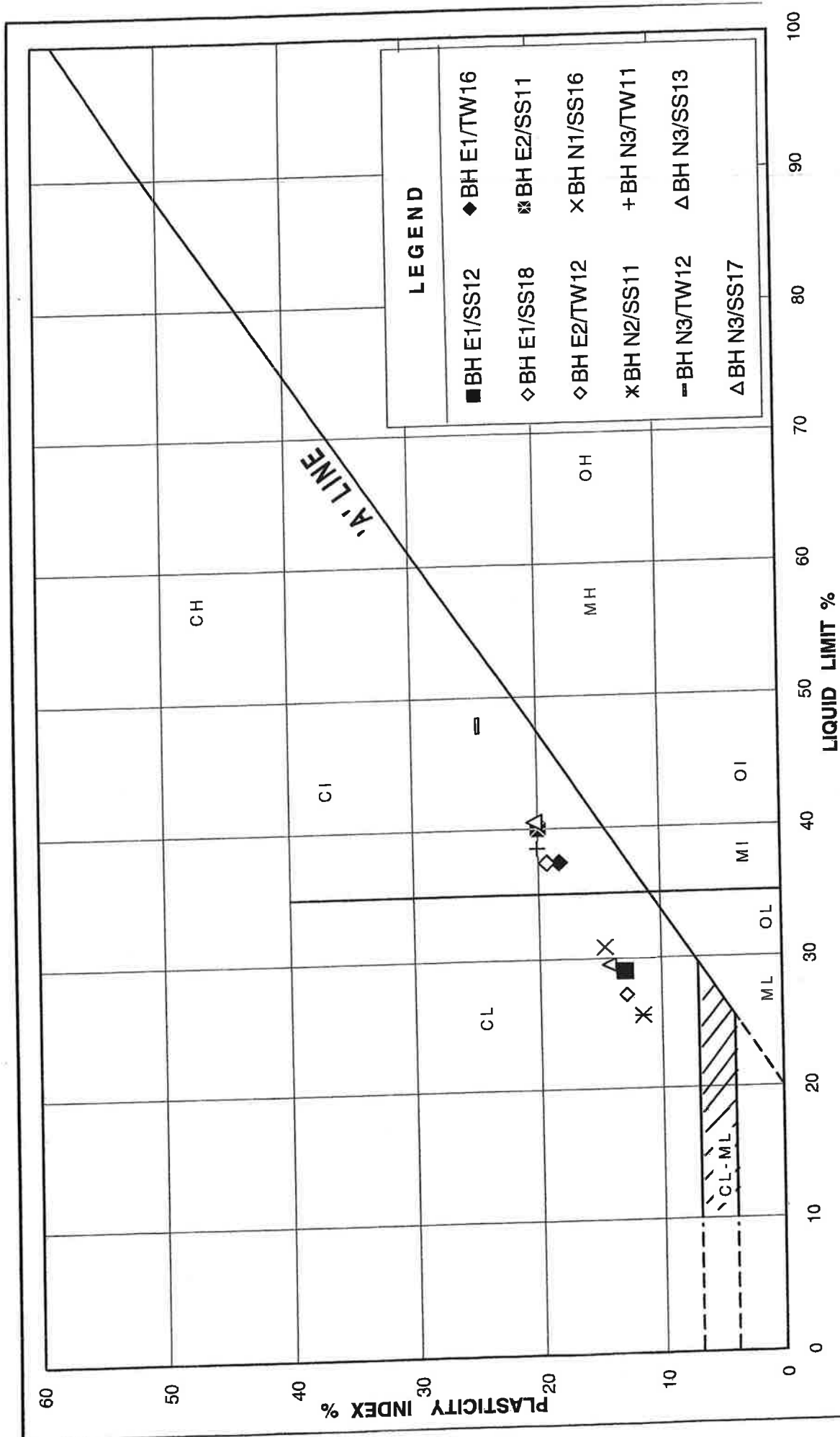


GRAIN SIZE DISTRIBUTION SILTY CLAY

FIGURE NO.: B-14

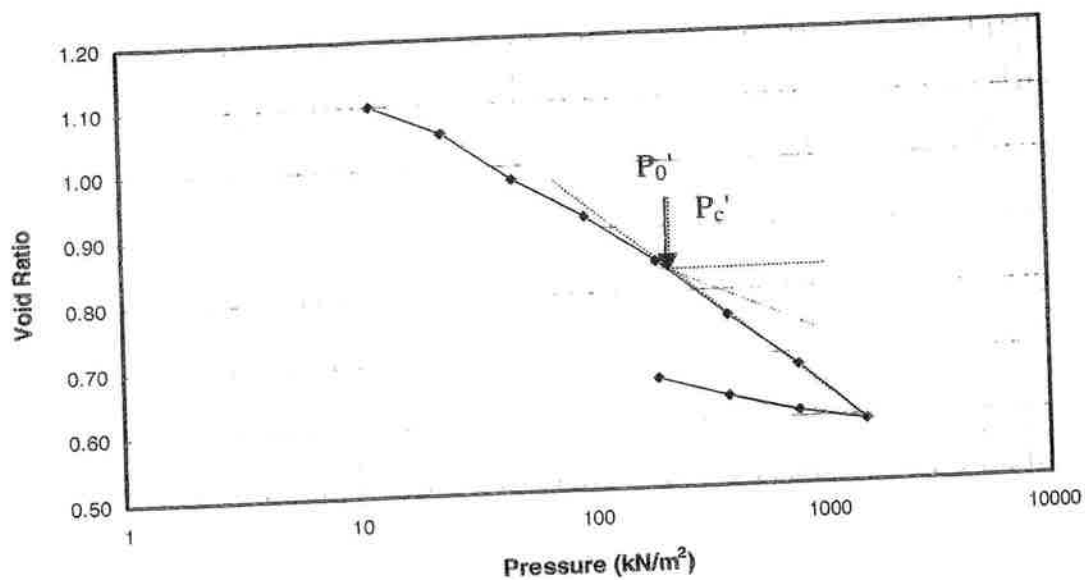
PROJECT NO: TRANETOBO1245AA

DATE: MARCH, 2010



Consolidation test- Borehole E1 TW 16

Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

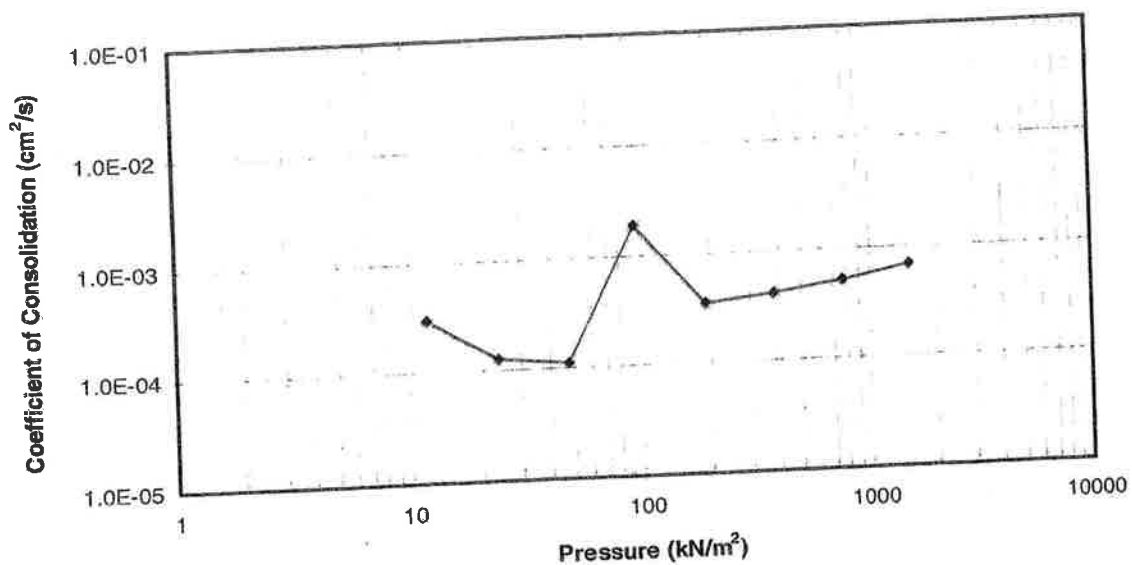
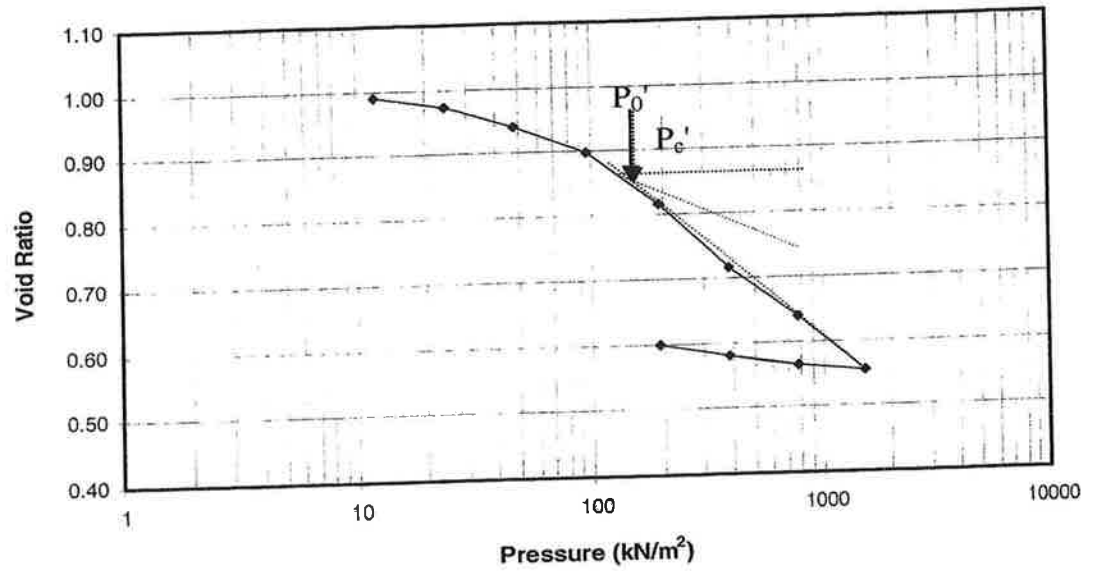


Figure B-16

Consolidation test- Borehole N3 TW 12

Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

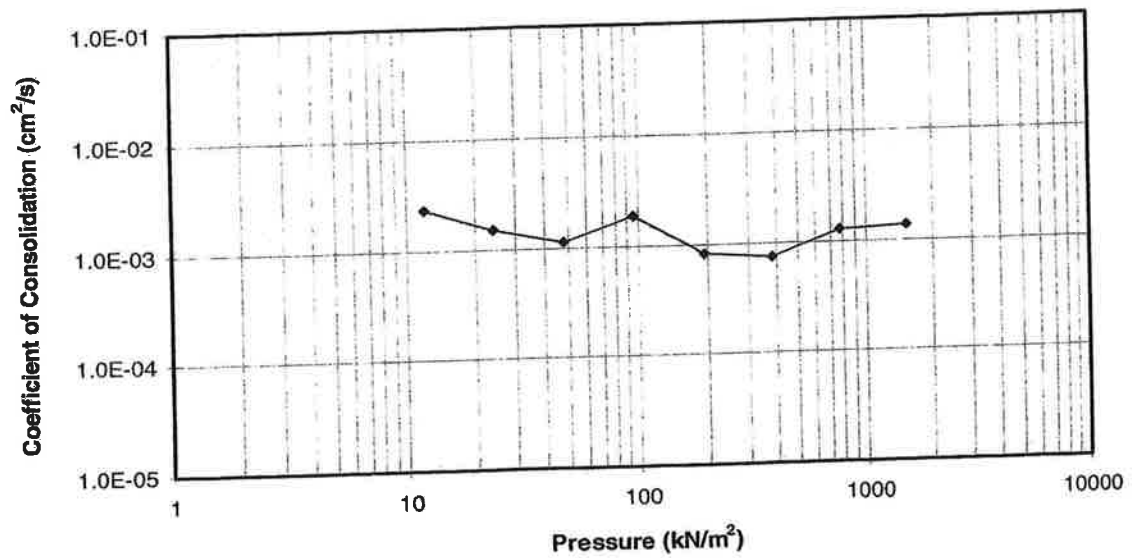


Figure B-17

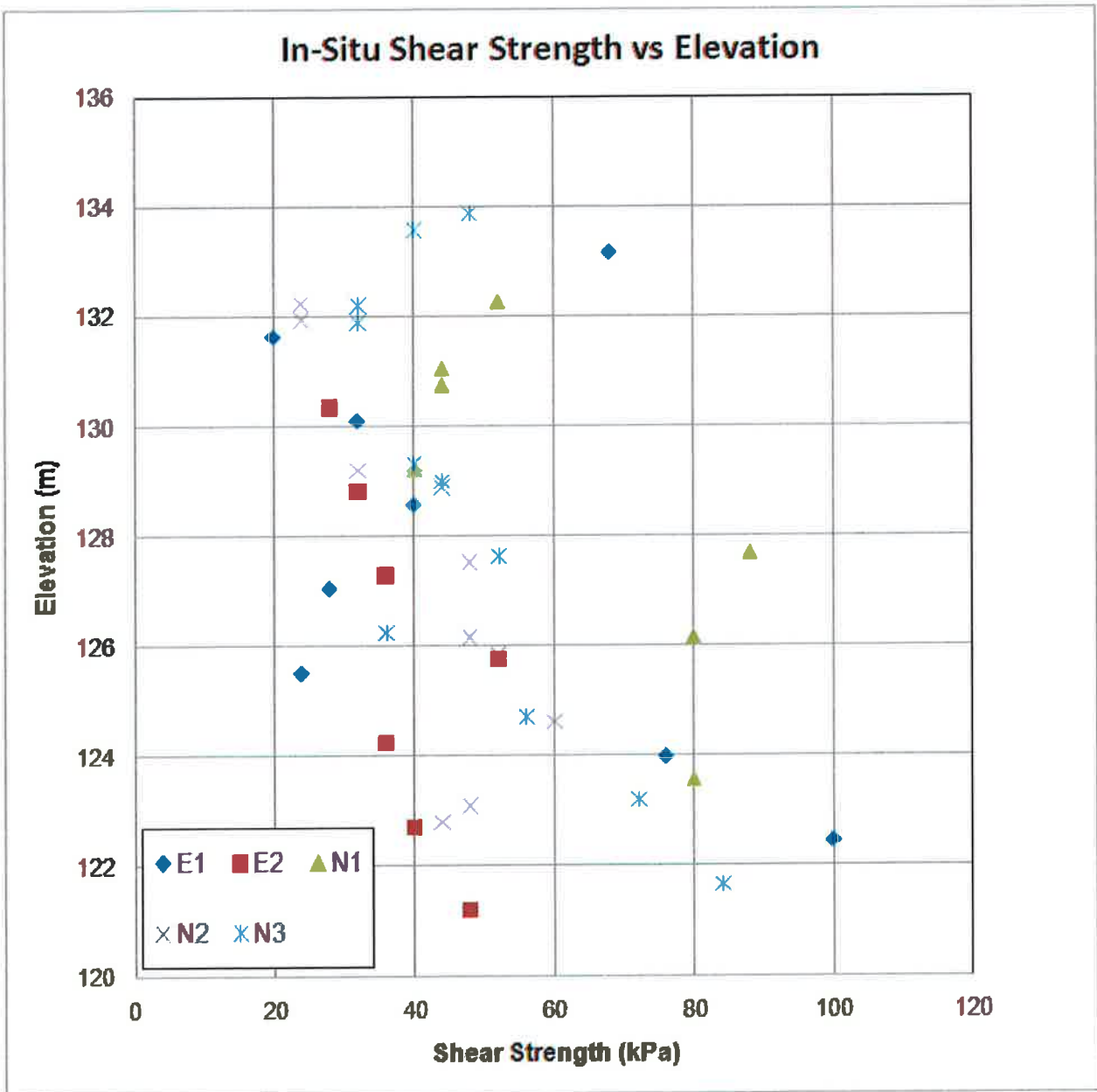
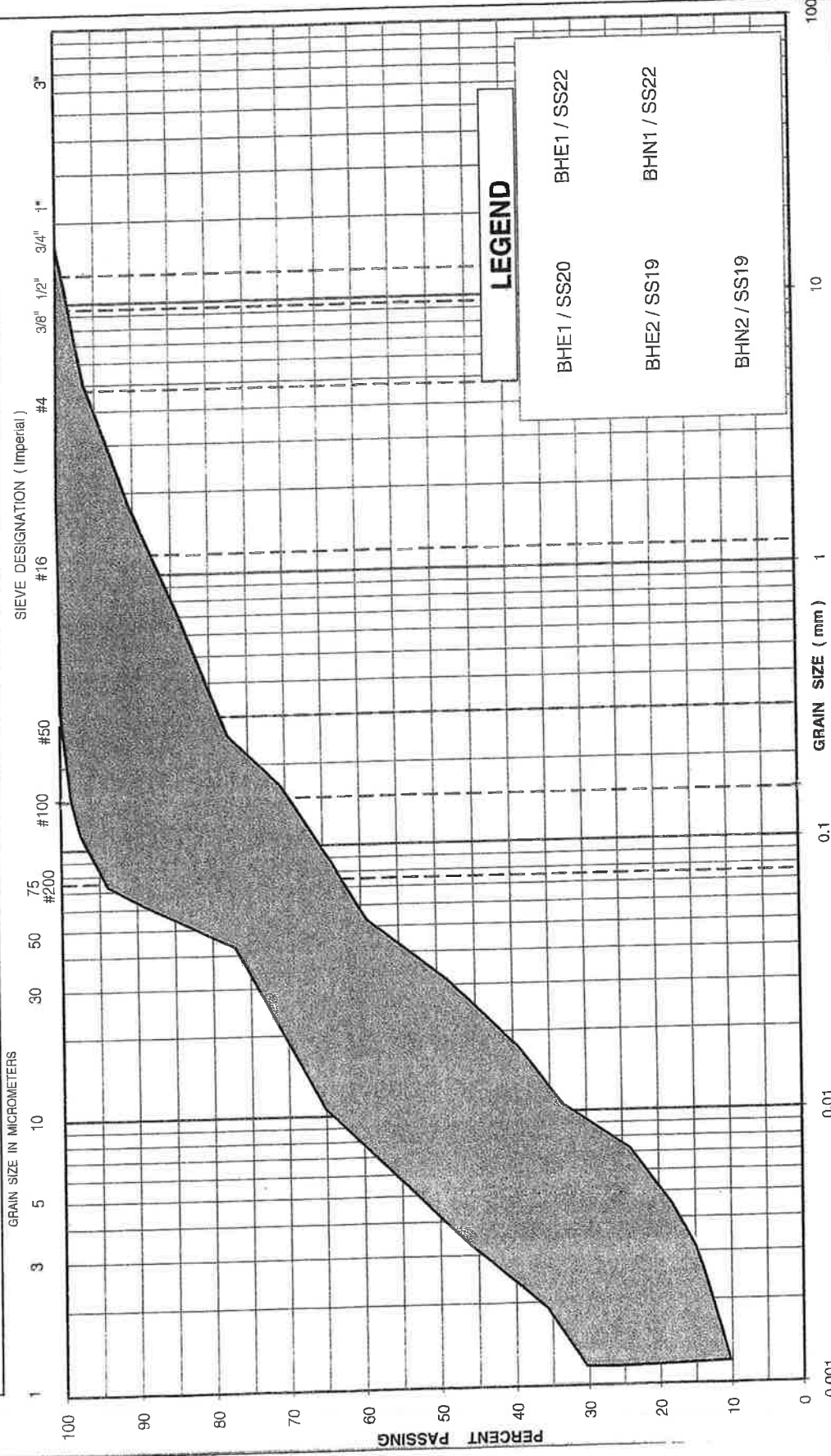


Figure B-18 In-situ shear strength vs Elevation

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



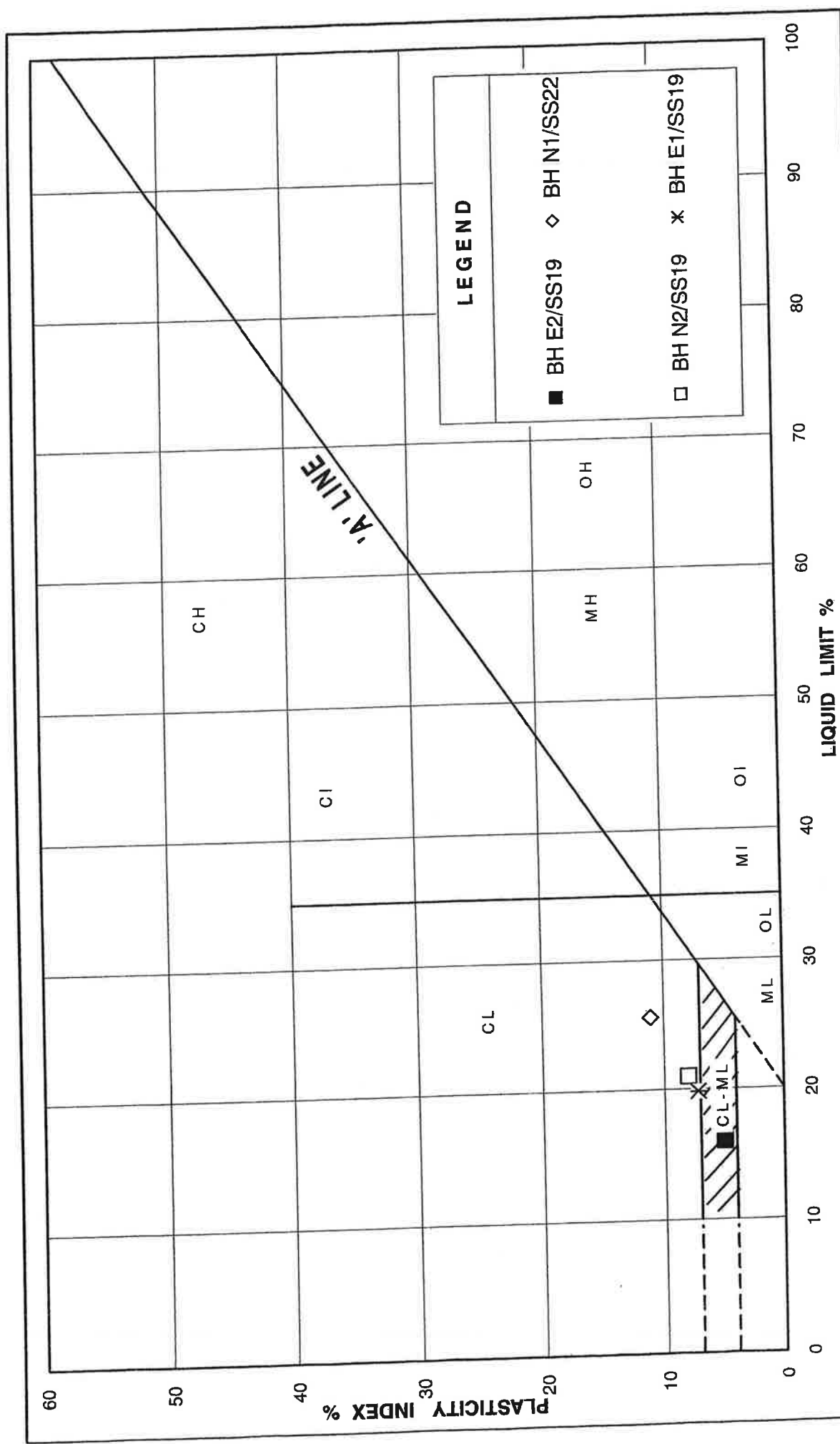


FIGURE No. B-20

REF. No. TRANETOB01245AA

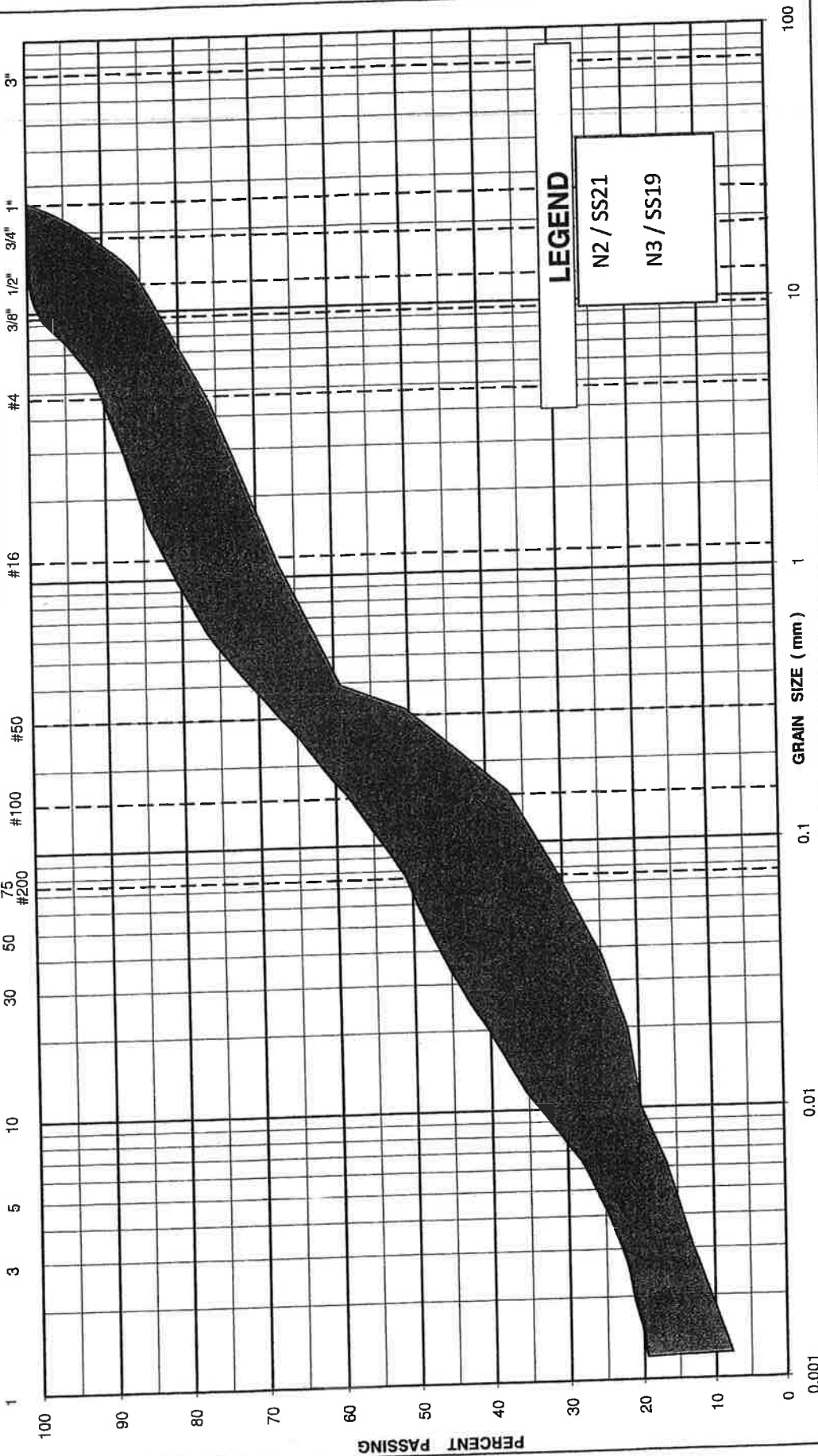
DATE MARCH, 2010

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT				SAND				GRAVEL			
				Fine	Medium	Coarse		Fine	Coarse		

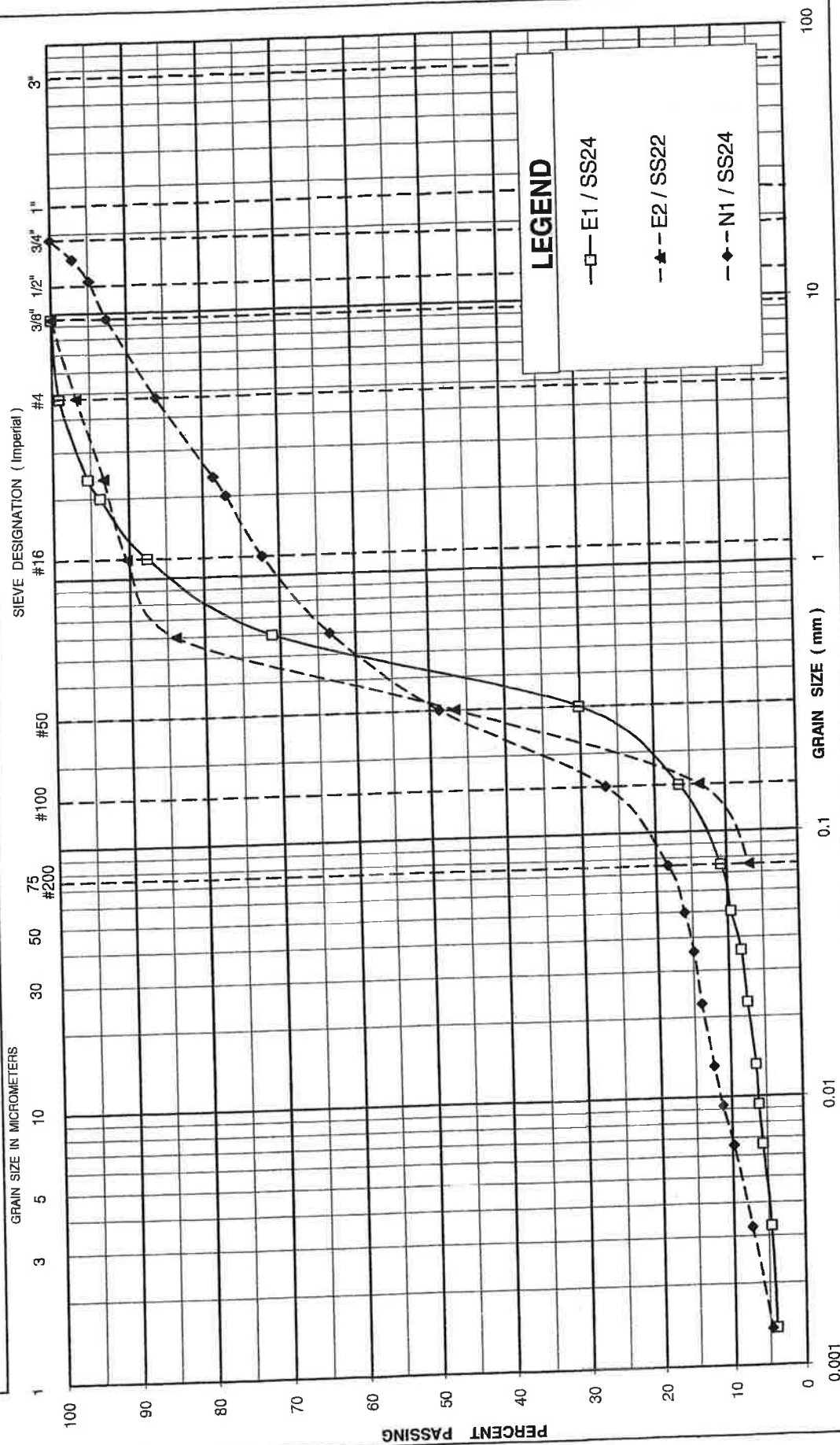
GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

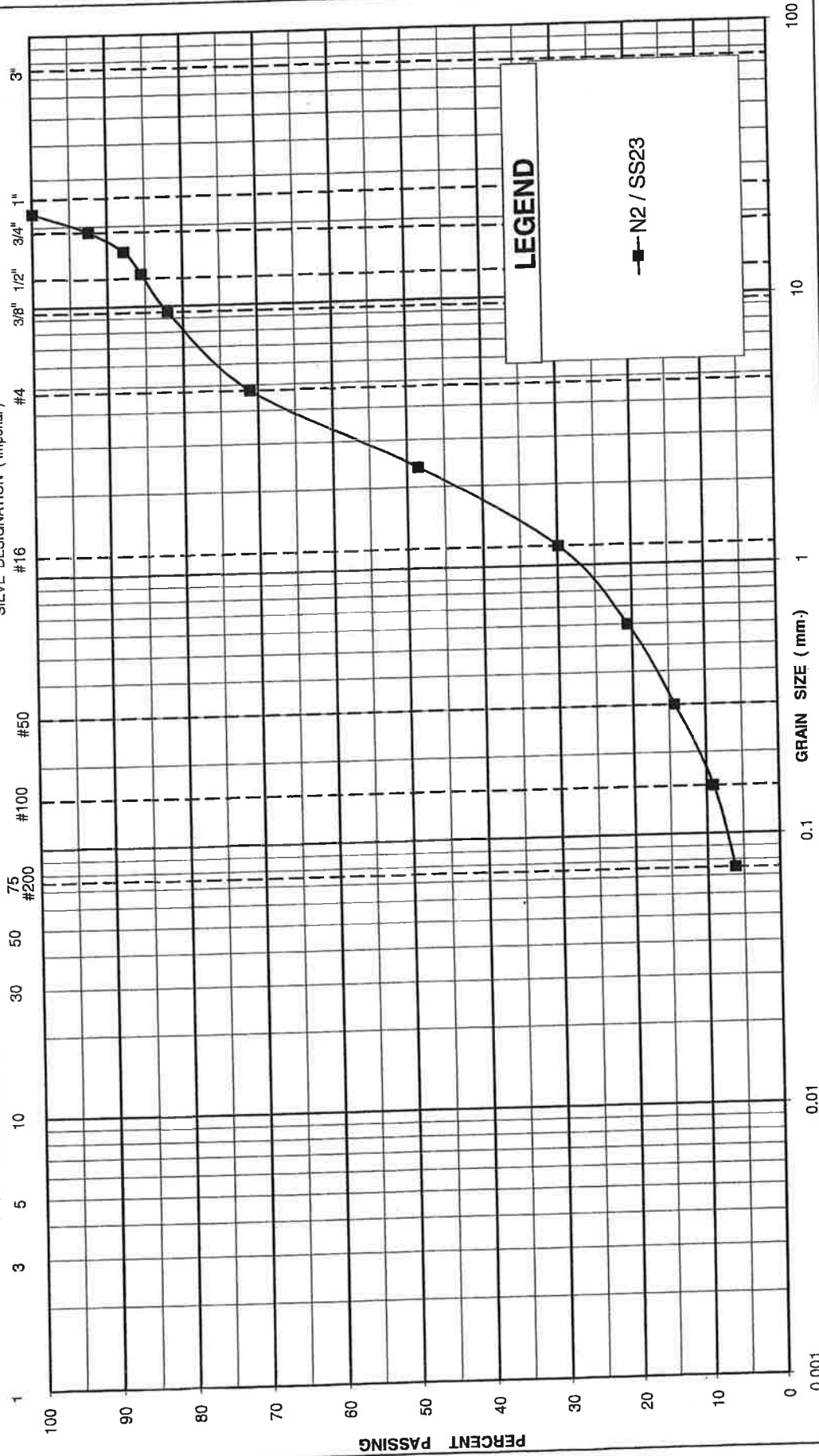


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



GRAIN SIZE DISTRIBUTION BASAL GRAVELLY SAND

FIGURE NO.: B-23
PROJECT NO: TRANETO01245AA
DATE: March, 2010

Appendix C

**Stratigraphic Contacts - Highway 401 and Oriole GO Parking Overpass Site
(Past Investigations)**

ORIOLE PARKING

References	Borehole Designation and	Ground Surface (ft)	Top of Sand (ft)	Top of Clay (ft)	Top of Till (ft)	Top of Lower Sand (ft)	Water Table (ft)	Ground Surface (m)	Top of Sand (m)	Top of Clay (m)	Top of Till (m)	Top of Lower Sand (m)	Water Table (m)	Depth to Till (m)	Depth to Sand (m)	Thickness						Flakiness																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
																Sand			Clay			Till			Fill or Clay, Silts, Organics			Sand			Clay			Till																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
																(ft)	(m)	(%)	(ft)	(m)	(%)	(ft)	(m)	(%)	(ft)	(m)	(%)	(ft)	(m)	(%)	(ft)	(m)	(%)	(ft)	(m)	(%)	(ft)	(m)	(%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
Department of Highways Ontario, 1967	Minimum:	433.8	433.5	424.0	387.5	369.0	428.5	132.2	132.0	129.2	118.2	112.5	130.2	11.4	26.1		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Appendix D

Site Photographs



Oriole Go Station



Oriole Go Parking lot (looking west from Leslie Street)



Oriole Go Parking lot (looking south from Oriole Go Parking Entrance on Leslie Street)



Oriole Go Parking lot

Appendix E

Explanation of Terms Used in the Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, 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 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 COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
SPACING					
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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_a	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
σ'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = 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_p	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p) / I_p$	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	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**PRELIMINARY FOUNDATION DESIGN
REPORT, ORIOLE GO PARKING
OVERPASS STRUCTURE
HIGHWAY 401 REHABILITATION FROM
LESLIE STREET TO WARDEN AVENUE
MTO CENTRAL REGION, G.W.P. 2130-01-00
GEOCRES 30M14-333**

Delcan Corporation
Project: TRANETOB01245AA-AC
September 30, 2011

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**PRELIMINARY FOUNDATION DESIGN REPORT
ORIOLE GO PARKING OVERPASS STRUCTURE,
HIGHWAY 401 REHABILITATION
FROM LESLIE STREET TO WARDEN AVENUE
MTO CENTRAL REGION, G.W.P. 2130-01-00**

4 DISCUSSION AND RECOMMENDATIONS

Rehabilitation of Highway 401 between Leslie Street and Warden Avenue is proposed. The original scope of the project was to rehabilitate the following structures:

<u>Structure Name</u>	<u>MTO Structure Number</u>
Highway 401 Overpass at Leslie Street/C.N.R. Ramp W-N/S	37-206/5
Highway 401 Overpass at Leslie Street/C.N.R. Ramp N-E	37-206/6
Highway 401 Overpass at Leslie Street/C.N.R. Ramp N-W	37-206/7

Subsequently, the project scope was revised as follows:

- C.N.R. overpass structure (single span rigid frame structure)
- Structure(s) over the existing Oriole GO Parking Lot
- Viaduct (north west quadrant of Highway 401 and Leslie Street interchange)
- Leslie Street overpass structure (two span rigid frame structure)

The project scope was changed after the borehole drilling portion of our foundation investigation was completed (between late 2009 and early 2010). Consequently, we were asked to submit the reports for this preliminary foundation investigation based on the available subsurface information, including the recently drilled boreholes by Coffey Geotechnics Inc (Coffey). It should be pointed out that the borehole coverage may not consistent with normal MTO procedures due to this fact. Foundation investigation recommendations for detail design are included in Section 5. This report provides preliminary foundation recommendations for the proposed Go Parking overpass structure. The proposed structure details and general arrangement drawing were not available at the time of preparing this report but we understand that a three-cell, rigid frame type structure is being contemplated. Consideration is also being given to a slab on steel girder option, which is similar to the existing structure. For the slab on steel girder option, the existing structure will be replaced with a new structure and additional new structure will be constructed on the south side of the existing Highway 401, based on the information provided to us by Delcan (see Construction plan drawing in Appendix H). In any case, the proposed GO Parking overpass replacement will be carried out in stages to accommodate the Highway 401 traffic. It is our understanding that details of construction staging will be developed during detail design.

In general, the sub-surface stratigraphy comprises fill materials and surficial non-cohesive to cohesive (typically non-cohesive) soil deposits overlying silty clay, which are in turn underlain by cohesive and non-cohesive glacial till deposits. The glacial deposits are further underlain by basal granular soils, within the depths of the previous and present investigations. The previous and present investigations indicate similar

overall subsurface conditions at the site. As various construction activities have taken place over the past many decades at the site, it is difficult to estimate embankment fill heights at the site. However, based on the existing topography and assuming that the existing CN track level is at about the original ground level (i.e. El. 141 m, based on the available contract drawings), the embankment fill heights at the Oriole GO Parking overpass structure site are estimated to be about 3 to 4 m.

4.1 Foundations

The replacement of the existing structures over Oriole GO Parking Lot is required as part of the proposed bridge replacement and realignment of the Highway 401 and Leslie Street interchange. Details of the new Highway 401 structures over the Oriole GO parking lot, and the applicable foundation details, will be developed during detail design. The proposed structure(s) will have similar minimum vertical clearance as the existing structure.

Construction will be carried out in stages and we understand that the construction will start with the demolition of the existing structure from one side (either north or south) and the remaining Highway 401 structures (lanes) will be retained for the Highway 401 traffic during the construction. After the construction of one segment, the construction will subsequently move to the other segments (towards the other end) and the newly built structure(s) and the remaining Highway 401 structures will support the Highway 401 traffic.

Based on the available MTO Geocres information, the existing Oriole Go parking lot overpass structure appears to be supported by caisson foundations and driven steel H-pile foundations (including battered piles at abutment locations near the Leslie Street). The available contract drawings indicate that the foundations were designed to be extended into dense to very dense till deposit (Department of Highway, Province of Ontario, Contract No. 65-205, Contract Drawings, dated 1965, Book 4 of 5). Available details for the existing foundations are summarized in Table G-1 in Appendix G. 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. It also appears that where driven H-piles were used, the pile caps cover the entire structure foot-print. Available information of the existing foundations is summarized in Table G-1 in Appendix G. We feel that these details of the existing structure foundations will likely play a significant role in the selection and construction of the foundations of the new structure(s).

Based on the results of past and recent investigations, we have considered a number of foundation options as follows:

- *Shallow foundations*
- *Drilled caisson foundations*
- *Driven steel H pile foundations*
- *Driven steel tube pile foundations*
- *Micropile foundations*
- *Continuous Flight Auger (CFA) pile foundations*

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

The following paragraphs present a discussion on these foundation options. The foundation details and technical options discussed below should be further studied during detail design and detailed foundation recommendations will be made at that time.

4.1.1 Spread Footing Foundations

Based on the prevailing subsurface conditions, the use of spread footings is not considered feasible at this location. This is primarily due to the fact that the existing surficial granular soils have a variable relative density and the underlying clay deposit is weak and compressible.

4.1.2 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) can be a feasible foundation option for the Oriole GO parking overpass structure. It is noted that some of the existing Highway 401 structures over the existing Oriole GO parking are supported on drilled caisson foundations. Since the site is located in the City of Toronto (i.e. close to residential areas and hospitals), drilled caisson is typically considered to be a favourable deep foundation option because of reduced noise and vibration generated during the construction, in comparison with driven piles. The existing structure needs to be removed prior to the installation of the caissons, with a staging plan.

Based on the local design practice, caissons are socketed into the very dense/hard till ($N > 50$ blows/0.3 m) a sufficient distance, whereby the caissons are designed for a combination of shaft friction/adhesion and end bearing resistance in competent till. For this particular project, for the sake of simplicity, we recommend that the caissons be designed for an end bearing resistance of 2000 and 3000 kPa at SLS and ULS, respectively, for a minimum of 2.0 m embedment (socket) in the very dense/hard till, plus a friction/adhesion factor. If very dense to hard till thickness is found to be less than 2.0 m (i.e. Boreholes 8A, 10A, 11A, 12A, 13A, E1 and E2), the above mentioned resistances would be revised. For preliminary design purposes, revised caisson capacities with 1 m penetration into very dense to hard till option is also presented in Table 5.1.2.1. Less than 1 m penetration is not recommended. As well, more than 2 m penetration into the till may also be objectionable, unless the till below the base of the caisson is sufficiently thick, due to the observed artesian conditions. In summary, we recommend that, with the available subsurface data, the design be based on a minimum 1 m and a maximum 2 m penetration into the very dense/hard glacial till deposit, subject to possible revision in light of the findings of the detailed foundation investigation. It is therefore recommended that additional deeper boreholes with piezometric instrumentation be advanced in detail design phase to confirm groundwater and subsoil conditions at the proposed caisson locations.

The following table presents our preliminary recommendations on commonly used caisson sizes, which are recommended for this project.

Table 4.1.2.1 Recommended Caisson Resistances

Caisson Diameter	Penetration length into the very dense or hard glacial till (m)	Recommended SLS value using base area only* (kPa)	Recommended Factored ULS value using base area only* (kPa)	Recommended Friction/Adhesion SLS value using circumferential area (kPa)	Recommended Factored Friction/Adhesion value at ULS using circumferential area (kPa)	Corresponding Caisson Resistance kN/Pile
0.91 m (36-inch)	2.0	2000	3000	75	120	1730 (SLS)* 2650 (ULS)
	1.0	1700	2550	60	100	1300 (SLS)* 2000 (ULS)
1.07 m (42-inch)	2.0	2000	3000	75	120	2300 (SLS) * 3500 (ULS)
	1.0	1700	2550	60	100	1800 (SLS)* 2700 (ULS)
1.22 m (48-inch)	2.0	2000	3000	75	120	2900 (SLS)* 4400 (ULS)
	1.0	1700	2550	60	100	2200 (SLS)* 3400 (ULS)

* SLS for 25 mm settlement

These caisson sizes are recommended for efficiency in installation in consideration of the prevailing subsurface conditions. Higher caisson resistances would be available for greater penetration into the competent till (where the very dense or hard glacial till is relatively thicker) but as mentioned earlier, this is not recommended with due consideration of the excess hydrostatics pressures which prevail at the site.

A sample calculation in determining the caisson resistances is as follows. A 1.07 m (42-inch) diameter caisson will have a base area of $(1.07/2)^2 \times \pi = 0.90 \text{ m}^2$. When designed for a value of 2000 kPa, a resistance of $2000 \text{ kN/m}^2 \times 0.90 \text{ m}^2 = 1800 \text{ kN}$ is obtained at SLS plus a resistance of 75 kPa for the additional 2.0 m penetration, giving $75 \text{ kN/m}^2 \times 1.07 \text{ m} \times \pi \times 2.0 \text{ m} = 500 \text{ kN}$. When added, the resulting resistance at SLS is $1800 \text{ kN} + 500 \text{ kN} = 2300 \text{ kN/pile}$.

The recommended clear distance between any two adjacent caissons is not less than 2.5 diameters centre to centre. It should be noted that the installation of new caissons may be difficult due to the presence of densely distributed existing piles and caissons supporting the existing structure at the site. Locations and sizes of new caissons need to be carefully selected considering the existing foundations (i.e. pile cap, battered pile, large diameter caisson etc).

Table 5.1.2.2 presents the anticipated caisson depths/elevations at recent investigated Coffey Boreholes E1, E2, N1, N2 and N3.

Table 4.1.2.2 Anticipated Caisson Depths/Elevations

Borehole No.	Existing Ground Elevation (m)	Anticipated Caisson Depth (m)	Anticipated Caisson Bottom Elevation (m)	Anticipated Socket and Base Subgrade Type
E1	143.2	26.3	116.9	Clayey Silt Till*
E2	140.4	24.5	115.9	Clayey Silt Till*
N1	146.9	27.9	119.0	Clayey Silt Till*
N2	142.3	23.3	119.0	Clayey Silt Till
N3	142.4	24.7	117.7	Silty Sand Till

*About 1 m to 1.5 m penetration into hard glacial till.

The minimum caisson diameter would be 0.76 m to enable the cleaning and inspection of the base of the caisson.

The ULS geotechnical resistance (unfactored) to lateral loading can be calculated using passive earth pressure theory outlined in Section C 6.8.7 of the *Commentary* to the CHBDC. For a single pile/caisson in non-cohesive soil, the passive resistance may be estimated by calculating passive earth pressure over an equivalent wall area having a depth from the ground surface equal to six times the pile/caisson diameter, and a width equals to three times the pile/caisson diameter. The pile/caisson diameter is the diameter of round pile/caisson or the average face to face distance of octagonal, hexagonal and square piles. For a pile/caisson in cohesive soils, the passive earth resistance should be limited to $2c_u$ (c_u =undrained shear strength) at the ground surface and increase linearly to $9c_u$ at a depth of three pile/caisson diameters and beyond. This pressure should be converted into a passive resistance by using a bearing width which equals the pile/caisson diameter. In accordance with CHBDC, a resistance factor 0.5 is to be applied in calculating factored ULS resistance. The ULS lateral resistance of a pile/caisson group may be estimated as the sum of the single pile/caisson resistance across the face of the pile/caisson group, perpendicular to the direction of the applied lateral load. We will be pleased to give you preliminary soil parameters regarding this aspect, if they are needed at this preliminary stage of foundation design.

Alternatively, for preliminary design purposes, the lateral resistance of SLS can be taken as between 5% and 8% of the axial caisson resistance for about 10 mm deformation at serviceability state based on our local experience.

Consideration can be given to the use of battered caissons to resist lateral loads similar to batter piles. A temporary (or permanent) liner (or casing) may be required to maintain the hole open during the installation, and it may not be easy to maintain the specified inclination. In our opinion, in practice, the installation of battered caissons with the prevailing site conditions will be difficult, requiring high degree of skilled workmanship. Consequently, this option is believed to be both risk adverse and costly and is therefore not recommended. However, the constructability of battered caisson can be discussed with a specialist contractor who has experience in inclined boring and caisson installation.

Difficulties may be encountered during the installation of the caissons due to the presence of surficial granular overburden below groundwater table, the observed boulder (Borehole 16A) in the silty clay and the anticipated presence of cobbles and boulders in the glacial till deposits. This can be discussed with a specialist contractor in relation to cost vs. caisson diameter. Dewatering may be required during the

installation of the caissons due to the observed high groundwater table. These aspects will need to be red flagged in the contract documents to minimize construction claims. An NSSP should be issued to alert the presence of cobble and boulder and potential basal and sidewall instability during the caisson installation. Temporary steel casing would be required to be installed during the construction of the caisson holes to prevent caving. The casing would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'. If a permanent casing is to be adopted, the above mentioned geotechnical resistances will need to be revisited. To prevent the disturbance of the base of the caisson, the concrete must be poured expeditiously without delay after the cleaning of the base and its inspection and approval. Dewatering process may be required to successfully install caissons in this respect. This may range from dewatering the upper perched water table to pressure relief measures to reduce the hydrostatic uplift condition at the base of the caisson. Tremie concreting of the caisson can be also considered to reduce dewatering requirements for the installation of the caissons. Based on the available subsurface information, tremie concreting may be a favourable option for this project.

The removal of the existing structure and its foundations may be required for installation of new caissons where they interfere with the existing foundations. This aspect needs to be carefully investigated at the detail design phase.

Additional deeper boreholes with piezometric instrumentation are recommended to reduce potential risk factors in caisson design and installation.

4.1.3 Driven Steel Piles

4.1.3.1 Steel H-Piles

From the geotechnical point of view, the boreholes show that the subsurface conditions at the site are suitable for the use of driven steel H-piles to support the proposed Oriole Go parking overpass structures. The borehole data also show that with the prevailing subsurface conditions, the use of a low displacement pile, such as steel H-pile with a heavy section (e.g. HP 310 x 110 or 310 x 125) would be better suited than other pile types (e.g. steel tube piles, steel H-piles with lighter sections or precast concrete piles). However, due to the expected noise and vibration induced by pile driving, this option may not be a favourable option for this project for environmental reasons as the site is located close to residential areas and hospitals in the City of Toronto. The vibration monitoring programme should be carried out during pile driving. Special Provisions for vibration monitoring was included in Appendix I.

If piles are to be used, the existing immediate adjacent overpass structures will need to be removed prior to driving the piles. Steel H-piles (HP310 x 110) driven to practical refusal in the competent glacial till materials (at about El. 122 to 112 m, average El. 119 m) can be designed for 1700 kN/pile for factored U.L.S. and 1250 kN/pile for S.L.S (for 25 mm settlement). These values can be increased by 50 kN/pile for HP 310 x 125 piles, due to the increase in the steel cross-sectional area. Normally, somewhat higher resistances are available for the pile sizes recommended. However, in view of the possible upward gradients, and artesian pressures and known past problems experienced at this interchange, the use of higher capacities is not recommended. The anticipated approximate pile tip (refusal) elevations at Boreholes E1, E2, N1, N2 and N3 are given in Table 4.1.3.1.1.

Table 4.1.3.1.1 Anticipated Pile Tip Depths/Elevations

Borehole No.	Existing Ground Elevation (m)	Anticipated Pile Tip Depth (m) below the existing ground	Anticipated Pile Tip Elevation (m)
E1	143.2	26.7	116.5
E2	140.4	24.8	115.6*
N1	146.9	28.0	119.3
N2	142.3	23.3	119.0
N3	142.4	24.2	118.2

*The pile may possibly penetrate by as much as 3 m below the quoted elevation.

The pile cap will need to be installed below the frost depth and therefore the anticipated pile lengths would typically be about 22 to 27 m.

The pile tip elevations provided are for preliminary estimating purposes only. Due to potentially variable soil conditions, the actual pile tip elevation may vary. The piles should be driven into the competent glacial till or the very top portion of the basal granular soil deposits, using a suitably heavy hammer capable of delivering a suitable rated energy. The possibility of piles encountering cobbles and boulders in the till should be anticipated. In view of this, as well as the very dense and/or hard nature of the till, the tips of the piles should be stiffened as per OPSD-3000.100 (or similar, such as Titus point) to minimize damage to the piles in anticipation of heavy driving conditions. Based on the present information, it is our opinion that flange plate stiffening (Type I) should suffice. This aspect should however be revisited based on the detailed investigation findings related to possible artesian conditions. This is because flange reinforcement plates may promote easier upward seepage of groundwater from the lower aquifer, along the steel H-pile. It is also our opinion that stiffening will not be required for the heavier HP 310x125 size steel H-Piles. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded). This is an important aspect as there has been extensive pile damage due to boulders in adjacent sites when somewhat lighter piles were used.

If the piles encounter refusal before sufficiently penetrating into the competent materials, then pile capacities may need to be revisited and alternative measures sought. Therefore, pile driving records will need to be kept and if refusal is met above the recommended bearing zone, a geotechnical engineer would review the driving records to assess the axial resistance. It is also possible that the piles may be driven some distance below the estimated pile tip elevations to achieve the desired capacity. We recommend that over driving of the piles more than about 1 m into the basal granular soils underlying the glacial tills should be avoided due to the anticipated artesian pressure within the basal granular soil deposits. This aspect should be kept in mind when conducting the detailed investigation.

As mentioned before, the use of light-weight (e.g. HP 310 x 79) piles is not recommended as lighter piles are more vulnerable to damage. 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 or hard strata.

Horizontal forces can be resisted by battered piles. While in the past, MTO has successfully installed batters of between 12V:1H and 3V:1H, in our experience a batter steeper than 4V:1H is difficult to install in

practice. We recommend, therefore, that the batter be no steeper than about 4V:1H. Pile caps should have a minimum of 1.2 m permanent frost protection.

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

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

- Horizontal Resistance at U.L.S. = 120 kN/pile
- Horizontal Resistance at S.L.S.* = 50 kN/pile

*for a lateral displacement of 10 mm at the pile head with reference to Section C6.8.7.1 of CHBDC

The removal of the existing structure and foundations may be required prior to driving piles where the existing structure and the existing foundations may interfere with the new foundations. In this regard, the use of steel H or tube piles may have a slight advantage compared to caisson foundations due to a smaller area (cross-section) of steel H or tube piles (less chance to hit the existing foundation) and flexibility of pile layout within the proposed pile cap. This aspect needs to be carefully considered for this particular project.

As mentioned before, the use of driven H-piles close to residential areas and hospitals may be subject to a noise and vibration study. Stability of the existing embankments and structure foundations under the anticipated vibrations induced by pile driving should be taken into consideration.

4.1.3.2 Steel Tube Piles

The use of steel tube piles is another option but their disadvantage is that they are higher displacement piles in comparison with H-piles and, as such, vibrations generated during pile driving may present a bigger issue. On the other hand, steel tube piles have the advantage that they can be inspected after driving and prior to pouring the concrete for possible damage that may have incurred while driving the pile.

The pile should have a 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 area of the piles (i.e. should not project beyond the circumference of the pile) so that adhesion/friction is not adversely affected. As well, a larger plate may promote easier water upflow along the pile in the case of excess hydrostatic pressure/artesian conditions, as such may present a greater risk in this respect. Tube piles will need to be filled with concrete after their installation and inspection for possible damage. In addition to being higher displacement piles in comparison with steel H-piles, one other disadvantage of tube piles, as mentioned before, is their greater vulnerability against soil loss in artesian conditions, in comparison with steel H-piles.

Steel tube piles of 300 mm nominal diameter with a suitable steel thickness (e.g. 324 mm x 9.4 mm), driven at least 1 m to 2 m into the very dense/hard soil, can be expected to provide a Factored Axial Resistance at U.L.S. of 1050 kN and an Axial Resistance at S.L.S. equal to 700 kN at about the tip elevations (approximately 0.6 m higher) quoted for steel H-piles given in Table 4.1.3.1.1, in Section 4.1.3.1.

Similar to steel H-piles, pile lengths may be different than the estimated values and, therefore, this aspect will need to be considered in the contract documents and when ordering piles.

If battered piles are required to sustain horizontal loads, then, as was mentioned before, we would suggest that the batter be limited to a reasonable value (e.g. say no steeper than 5V:1H), as in practice greater batter may be difficult to install.

As mentioned before, the use of driven piles close to residential areas and hospitals may be subject to a noise and vibration study. As well, the stability of the existing embankments and possible damage to structure in the immediate vicinity, due to the anticipated vibrations induced by pile driving should be taken into consideration. The vibration monitoring programme should be carried out during pile driving. Special Provisions for vibration monitoring was included in Appendix I.

4.1.4 Micropiles

Another alternative which may be considered is the use of micropiles to support the new overpass structure(s).

A micropile is constructed by drilling a borehole, placing a steel reinforcing bar, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in most soil and rock types and ground conditions. Micropiles can be installed through obstacles such as cobbles, boulders and even through the existing structures or foundations, and anticipated hydrostatic uplift (if encountered) can be counter-balanced by high pressure grout. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing structure. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout & ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter (typically 160 to 260 mm), end-bearing contributions in micropiles are generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

Based on the prevailing subsurface conditions at the site, geotechnical resistance of micropile primarily depends on the bond length within the competent glacial till and basal granular deposits and the type of micropile/installation method. For preliminary design purposes, axial resistances of up to about 500 kN/micropile may be available at ULS (for 260 mm diameter micropile) with a penetration not less than 6 m into the competent soil deposits (i.e. very dense soils). The lateral resistances would depend on the diameter and reinforcement of micropile and needs to be checked with a specialist contractor.

As mentioned before, the use of micropiles are likely to be less economical than other conventional deep foundations due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install conventional deep foundations. However, use of micropiles may shorten the construction period because micropiles can be installed under the existing structure without traffic disruption. As well in view of the congested nature of existing foundations, micropiles may provide a more economical solution. In this respect you may wish to check with a

specialized contractor if the micropiles can be installed through caissons containing reinforcing and/or through steel H-piles.

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

4.1.5 Continuous Flight Auger Piles (CFA)

CFA piles are a type of drilled foundation in which the pile is drilled to the final depth in one continuous process using a continuous flight auger are filled with soil, providing lateral support and maintaining the stability of the hole. As the auger is withdrawn from the hole, concrete or a sand/cement grout is placed by pumping the concrete/grout mix through the hollow center of the auger pipe to the base of the auger. Simultaneous pumping of the grout or concrete and withdrawing of the auger provides continuous support of the hole. Reinforcement for steel-reinforced CFA piles is placed into the hole filled with fluid concrete/grout immediately after withdrawal of the auger. CFA piles are typically installed with diameters ranging from 0.3 m to 0.9 m (12 in to 36 in), but locally typical diameters are 0.5 m to 0.6 m (20 in to 24 in) lengths of up to about 24 m are locally available, although longer piles have occasionally been used. This maximum CFA pile length should be discussed with local contractors, if you wish to use CFA option. The reinforcement is often confined to the upper 10 to 15 m of the pile for ease of installation and also due to the fact that in many cases, relatively low bending stresses are transferred below these depths. In some cases, full-length reinforcement is used, as is most common with drilled shaft foundations. CFA piles can be constructed as single piles (similar to drilled shafts), for example, for noise wall or light pole foundations. For bridges or other large structural foundations, CFA piles are most commonly installed as part of a pile group in a manner similar to that of driven pile foundations. Similar to driven piles, the top of a group of CFA piles is terminated with a cap. Typical minimum center-to-center spacing is 3 to 5 pile diameters, preferably 5.

CFA piles differ from conventional drilled shafts or bored piles, and exhibit both advantages and disadvantages over conventional drilled shafts. The main difference is that the use of casing or slurry to temporarily support the hole is avoided. Drilling the hole in one continuous process is faster than drilling a shaft excavation, an operation that requires lowering the drilling bit multiple times to complete the excavation. In contrast, the torque requirement to install the continuous auger is high compared with a conventional drilled shaft of similar diameter; therefore, the diameter and length of CFA piles are generally less than drilled shafts, as well as limiting the depths. The use of a continuous auger for installation also limits CFA piles to soil or very weak rock profiles, while drilled shafts are often socketed into rock or other very hard bearing materials. Because CFA piles are drilled and cast-in-place rather than being driven, as are driven piles, noise and vibration due to pile installation are minimized. CFA piles also eliminate splices and cutoffs. Soil heave due to driving can be eliminated when non-displacement CFA piles are used. Hydrostatic uplift conditions at the bottom of the borehole (if any) can be counter-balanced with concrete or a sand/cement grout. A disadvantage of CFA piles compared to driven piles is that the available QA methods to verify the structural integrity and pile bearing capacity for CFA piles are less reliable than those for driven piles. Another disadvantage of CFA piles is that CFA piles generate soil spoils that require collection and disposal. Handling of spoils can be a significant issue when the soils are contaminated or if limited room is available on the site for the handling of material. Depending on the diameter and depth of

the CFA pile, resistance values up to the order of about 1700 kN/pile (factored) at ULS and 1100 kN/pile at SLS would likely be available.

CFA piles have been used worldwide and also in the U.S. commercial development, but have not been used frequently for support of transportation structures in the North America. This underutilization of CFA is a result of perceived difficulties in quality control and of the difficulties associated with incorporating a rapidly developing technology into the traditional. Recent advances in automated monitoring and recording devices will alleviate concerns of quality control. Also, CFA can be installed for low headroom condition or in confined space with segmental augers in some countries. Availability of equipment and construction details should be discussed with a specialist contractor and we will be pleased to expand on this further should MTO wish to pursue this option in detail design phase.

4.1.6 Use of Existing Foundations

Another alternative would be to look into the possibility to fully or partially utilize the existing foundations to support the new structure(s), especially for the slab on steel girder option. This may involve supplementing the existing foundations with additional foundations such as micropiles. In our experience, however many Structural Engineers do not favor the use of composite foundations on the basis of their load response characteristics and compatibility issues. If this option is to be pursued, a thorough study of the existing foundations needs to be made, along with as built information such as caisson/H-pile installation records, base elevations, etc.

In any case, if the existing piles are considered to be re-used to support the newly proposed structure(s), residual capacity and integrity of the existing piles will need to be carefully evaluated/verified. Detailed information about the existing piles must be available and sufficient to provide confidence in their re-use. If non-destructive tests are under consideration, following tests are the available non-destructive tests listed in American Society of Testing and Materials (ASTM) standards:

Table 4.1.6.1

ASTM number	Title
D4945	Standard Test Method for High-Strain Dynamic Testing of Piles
D5882	Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations
D6760	Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing
D7383	Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations

If new piles to be installed supplement the capacity of the existing piles, spacing between the existing and new piles needs to be taken into account to evaluate new capacity of the hybrid foundation system. Pile spacing in between the existing and new piles should be maintained in accordance with CHBDC S6-06 Section 6.8 considering pile type, length and load carrying mechanism. In this particular case, in view of the relatively long nature of driven piles, we recommend that distance between driven piles be no less than 5 times the pile diameter, centre to centre. Interference of the existing foundations with the new foundations should be carefully investigated prior to laying out the new foundations. Especially for a long pile, verticality (or batter) should be precisely maintained under any circumstances. Differential settlement between the existing and new foundations also needs to be considered.

4.1.7 Recommended Preliminary Foundation Option

From a geotechnical engineering point of view, H-pile, drilled caisson and micropile foundations can be used for the prevailing subsurface conditions, with H piles being the preferred option from geotechnical and economics point of view. However, due to noise and vibrations issue this may not be feasible for environmental concerns in an urban setting. In that case caissons would be preferable. Considering all environmental, construction (including staging) and traffic disruption issues, the use of micropiles can be a preferred option for the rigid frame option but to assess this a cost comparison will need to be performed regarding the installation costs versus advantages provided by micropiles while working in confined spaces and with due regard to the existing underground pile caps, caissons etc.. The use of existing foundations option for the slab on steel girder option should be carefully investigated in the detail design phase, if it is to be used. The use of CFA foundations is largely dependent upon availability of equipment with the local contractors. Advantages, disadvantages, risk & consequences and cost of each foundation option were listed in Table F-1 (Appendix F). The foundation treatment preliminary recommendations should be confirmed by the TPM Consultant, once all the other aspects such as cost, staging and environmental issues are considered.

4.2 Preliminary Recommendations for the Proposed Retaining Walls

We understand that the project includes the construction of retaining walls on the south side of the existing highway if the slab on steel girder option is adopted. It is our understanding that details of the proposed retaining wall will be developed during the detail design phase. The height of the walls can be expected to be of the order of 5 to 7 m (i.e. similar to the existing structure height). The walls will need to be high performance and high appearance type of retaining structure (i.e. vertical concrete wall).

Typical retaining wall options are as follows;

- Conventional Cast-in-place Reinforced Concrete Retaining Walls
- Mechanically Stabilized Earth /Retained Soil System (MSE/RSS) Walls

These options based on the available subsurface data, are discussed in the following paragraphs to cover possible retaining walls at the Highway 401-Leslie interchange area.

4.2.1 Conventional Cast-in-place Reinforced Concrete Retaining Walls

The available borehole data show that the possible retaining wall locations are probably underlain by fill deposits which generally range in thickness from 3 to 6 m. The fill is underlain by fine grained granular soils which typically range in thickness from 3 to 7 m, bringing the combined thickness of these surface or near surface deposits generally from about 6 to 11 m, but typically about 8 to 9 m. Historical boreholes by MTO, located in the vicinity of the possible retaining wall locations, were advanced without sampling and hence there are no N-values that are available. Six of the boreholes drilled by Coffey for this present investigation are located in the reasonable proximity of the site and these give the indication that the fill generally consists of fine grained granular soils with typical N-values ranging from about 6 to 30 blows/0.3 m (i.e. generally loose to compact but typically compact). The underlying fine grained granular natural soils are typically in a compact condition. As with elsewhere at the site, these deposits are underlain by a weak and compressible silty clay deposit. As was mentioned before, these conditions are unfavorable for the

foundation support of structures. Unless the stresses can be substantially distributed surficial soils overlying the silty clay, excessive settlements can occur. For this purpose, a comprehensive settlement analysis should be made when the details are known.

With the presently available data, in general, conventional cast-in-place reinforced concrete retaining walls extending to 5 to 7 m height can not be supported on conventional spread footing foundations with the prevailing subsurface conditions.

Available deep foundation options for the proposed retaining walls are as follows;

- Driven Steel H piles
- Driven Steel tube piles
- Cast-in-place concrete piles
- Continuous auger flight piles
- Micropiles

If the retaining walls and their backfill will induce an additional stress in founding soils, downdrag on the deep foundations should be considered.

If the proposed retaining wall will be placed on a sloping ground (i.e. embankment or berm side slope), stability of the existing slope should be maintained during the construction.

Founding depth/elevation of deep foundation needs to be decided when structural requirements of the proposed retaining walls are available. Battered pile may be required to resist the lateral loads on the retaining structure(s).

Alternatively, depending on the retaining wall configuration, lightly loaded structure may be feasible by utilizing light weight fill e.g. EPS (Expanded Polystyrene) behind the wall or where grade raise is required. In this instance it may be possible to use spread footing foundations using a procedure similar to the following approach. The upper 1.5 to 2.0 m of the existing fill beneath the footing may need to be removed within a strip of about 3 m and replaced with well compacted granular engineered fill under geotechnical supervision to provide a uniform and reliable support. In that case the use of geotechnical resistances of up to 240 kPa at factored ULS and a resistance of the order of 150 kPa at SLS may be feasible for footing widths of up to 2 m. These aspects need to be verified depending on the location and details of the retaining walls. For this purpose a comprehensive detailed geotechnical investigation would be required.

4.2.2 Retained soil system (RSS)

Consideration can also be given to the use of retained soil system (RSS) walls for the retaining structure provided there is sufficient horizontal space to implement this option. Vertical wall facing segmental concrete panel RSS with reinforcement installed within backfill (i.e. Tensar/Nilex Acres, Terrafix Terrafort) may shorten the construction period. Typically, this type of RSS wall is supported on a granular bearing pad. In our opinion this type of system will not be suitable due to the presence of the weak silty clay deposit, but this should be confirmed when details are known.

4.2.3 Retaining Wall Backfill

If and where grade raise using normal earth fill over the original grade (o.g.) levels is required for this project, the stresses induced can be expected to cause settlements in the upper granular soils (where these deposits are not competent enough due to their variable relative density as revealed by the presently available geotechnical data), as well as due to the presence of the underlying silty clay, which is generally compressible and prone to long term consolidation settlements.

Consideration can be given to light weight fill material such as EPS. Basically, EPS backfill can be used for embankment or retaining wall structure to minimize or eliminate additional loading to the existing ground. In most cases, stability and settlement concerns can be minimized or eliminated by using EPS. Construction period can also be shortened by using EPS.

Since RSS is required install its reinforcements (i.e. geogrid) generally within the earth backfill, this option may not be feasible for the proposed retaining wall.

Reinforced concrete retaining wall with EPS backfill may be a favorable option at this stage.

The above mentioned options should be carefully considered when details of the proposed retaining walls are available in the detail design phase.

4.3 Lateral Earth Pressures

Backfill behind the overpass structure and associated retaining structures (if any) should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150 and OPSD 3101.200.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B' Type I or Type II, with minus 0.075 mm sieve size material not exceeding 5%) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C. For design purposes, the following static parameters (unfactored) can be used.

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

Angle of Internal Friction, $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$

$K_o = 0.43$

Compacted Granular 'B' Type I

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$$K_a = 0.31$$

$$K_o = 0.47$$

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the rigid frame structure is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C. 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 C.H.B.D.C. When selecting the parameters vibrations from the train traffic should also be considered.

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 C.H.B.D.C. commentary can be consulted. We understand, however, that the present design of the rigid frame structure does not incorporate any wing walls.

Vibratory equipment for use behind the rigid frame structure and retaining structures should be restricted in size as per current MTO practice.

EPS type backfill will reduce the lateral pressure on the retaining wall structures, but such details need to be sorted out when structures are better defined in the detail design phase.

EPS will need to be well drained to avoid hydrostatic uplift and damage due to continuous exposure to water, as well as to prevent hydrostatic forces on the wall itself. Drainage for this type of structure is usually maintained by a vertical drainage sheeting such as MiraDRAIN (high-performance, high strength drainage composite consisting of a three-dimensional, high-impact polystyrene core, and a woven filter fabric) along with horizontal drains at appropriate levels.

4.3.1 Seismic Design Consideration

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-00). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S , for the site is 1.0. Table A3.1.1 of the CHBDC provides that Toronto has a Zonal Acceleration Ratio of 0.05 and Velocity Related Seismic Zone (Z_v) of zero. As site coefficient (S) is 1.0, and the zonal acceleration is 0.05, the design zonal acceleration ratio for the site can be taken as $A=0.05$. This bridge site can be classified as Seismic Performance Zone 1 based on the above values. Subsection 4.4.5.3 and Table 4.2 of the CHBDC indicate that seismic analysis is not required for bridge in Seismic Performance Zone 1. These should be reviewed by the structural engineer.

Evaluation of sliding and overturning stability of whole EPS block under earthquake excitation may be required in the detail design phase.

4.4 Construction Comments

We understand that the proposed pile cap (bottom) will be installed at least 1.2 m below the final grade and pile cap thickness will be about 1.0 m based on the GA Drawing provided to us by Delcan. It is envisaged that some dewatering will be required to provide a stable excavation base. Pumping from shallow filtered sumps along the perimeter ditches will probably suffice, if needed. However, type of the dewatering need to be decided after the details of construction are available.

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

OPSS 539 – Construction Specification for Temporary Protection Systems

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

The boreholes show that the excavations can be expected to extend through some fill materials, occasionally clayey silt, into the surficial granular (i.e. sand, silt, sandy silt to silty sand) deposits. These soils can be classified as follows:

Fill	Type 3 soil above water level.
	Type 4 soil below water level.
Surficial Clayey Silt	Type 3 soil.
Upper Sand, Silt, Sandy Silt to Silty Sand	Type 3 soil above water level.
	Type 4 soil below water level.

Temporary shoring may be required to support the deeper excavations, due to space limitations at the site and possible removal of the existing pile caps. Some dewatering may be required due to a perched groundwater condition within the fill and surficial granular soils. In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing/rakers). Tight interlocking sheeting is also used. The advantage of the latter is that dewatering effort within the interlocking system will be minimized. However vibrations generated during the sheet pile driving may be detrimental to the existing and newly built pile caps adjacent to it. The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required Performance Level is considered 2 depending on the details of the retained structure or embankment. The shoring system should be designed by a Professional Engineer, experienced in this type of work. All shoring should be in accordance with OPSS539.

Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Granular Fill	0.32	0.49	3.1	20.5
Other Fill	0.38	0.55	2.7	18.0
Clayey Silt	0.45	0.62	2.2	17.5
Silt	0.38	0.55	2.7	18.0
Silty Sand to Sandy Silt	0.33	0.50	3.0	19.5

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Sand	0.31	0.47	3.2	20.5

It should be pointed out that the presence of gravel and cobbles can possibly occur within the overburden, as well possibly in the fill. If encountered, these can cause some problems during the installation of shoring units.

It is also recommended that as a precaution, it would be prudent to monitor the vibrations during the driving of the shoring support units (e.g. sheet piling) close to the existing and newly built structure units. Special provision for vibration monitoring is given in the Appendix I. An NSSP may need to be issued in this respect.

We recommend that an NSSP be issued specifying that shoring piles will be cut off approximately 1.2 m below grade.

Regular Polystyrene products are combustible and protecting against fire hazard during construction and service is an important consideration.

4.5 Frost Protection

Design frost protection depth for the general area is about 1.2 m. Therefore, a permanent soil cover of about 1.2 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps.

5 RECOMMENDED SCOPE OF WORK REQUIRED FOR DETAILED FOUNDATION INVESTIGATION

As was mentioned before, the investigation was carried out for the rehabilitation option of some of the existing bridge structures. The recent Coffey investigation fell short of MTO requirements for the newly proposed structure. It may be prudent to advance additional boreholes to augment the information from this investigation. Regardless, additional boreholes should be drilled for the proposed structure, once it is finalized, during the detail design phase.

Followings are our recommendations for detail foundation investigation, as per typical MTO requirements.

Structure	Investigation requirements
Bridge (Overpass Structure)	not less than two boreholes per each foundation unit – minimum 3 m penetration into competent materials (SPT in excess of 100 blows/0.3 m materials. Piezometers should be installed.
Protection System (<100 m long)	One borehole is required at each end of the protection system. Maximum borehole spacing 50 m, borehole depth depending on the height of protection system.
Retaining Wall (<100 m long)	One borehole is required at each end of the structure. Maximum borehole spacing 50 m, borehole depth depending on the height of retaining wall (minimum to a depth equal to twice the height of the wall)
Embankment (<250 m long)	One borehole is required at each end of the embankment. Maximum borehole spacing 25 m, borehole depth depending on the height of embankment (minimum 3 m into the competent material)

The boreholes for the bridge structure foundations should be deep enough to verify the existence and the complete thickness of the dense to very dense sandy silt to silty sand till deposit. As well, the upward, excessive hydrostatic pressures in this deposit, as well as in the underlying granular deposits, should be carefully observed during the drilling and also by means of piezometer installations.

Existing foundations should also be carefully investigated during detail design phase to minimize the potential construction problems.

As well, details of the proposed structure (e.g. retaining wall) and construction staging plan should be available in detail design stage.

6 CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



Ramon Miranda, P.Eng.



Zuhtu Ozden, P.Eng.



Appendix F

Evaluation of Foundation Alternatives

Table F-1. Foundation Options for Go Oriole Parking. Overpass Structure

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations	-Not feasible from geotechnical point of view.		Low cost	-Not feasible due to the prevailing subsurface conditions
Driven steel H-pile foundations	<p>-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles</p> <p>-Driving piles close to the existing and newly built structure may not be desirable due to vibrations and noise in urban area (close to the residential area and nearby hospital)</p> <p>-Cannot be installed prior to removal of existing superstructures (traffic disruption)</p>	<p>-Some Interference of the existing structure foundation is expected depend on the proposed pile foundation layout</p> <p>-Cobbles, boulders may be encountered during the driving, which may present problems</p> <p>-Vibration and noise</p> <p>-Possible problems with slope instability due to vibrations during the installation of the piles</p>	Moderate cost	<p>-Feasible but may be unacceptable due to environmental considerations (i.e. noise and vibration generated by construction)</p> <p>-The use of driven piles will need further study due vibrations and noise generated</p>
Driven Steel Tube Pile Foundations	Similar to driven steel H-piles, except they are higher displacement piles in comparison with H-piles	Similar to driven steel H-piles except possible greater vibration generation	Moderate Cost	Similar to driven Steel H-piles
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	<p>-Less vibrations and noise created than driven piles</p> <p>-Cannot be installed prior to removal of existing superstructure (traffic disruption)</p> <p>-Disposal of spoils will present a problem if soil at the site is found to be contaminated</p>	<p>-Some Interference of the existing structure foundation is expected depend on the proposed caisson foundation layout</p> <p>-The presence of cobbles and boulders may present problems during the installation of drilled caisson foundations.</p> <p>-Basal heave possibility if extended deeper and/or close to the base generally sand sand layer. May require dewatering</p>	Moderate cost but more expensive than driven piles	-Feasible option but slightly less suitable for the prevailing subsurface conditions from Geotechnical point of view in comparison with driven steel piles but more acceptable from environmental point of view

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Micropile Foundations	<ul style="list-style-type: none"> -Minimizes vibrations and dewatering -Can be installed in low overhead conditions and reduces interference with existing foundations. Can be installed through existing pile caps. - Expensive due to special equipment / material and specialist contractor 	<ul style="list-style-type: none"> -Problems may arise during the installation due to cobbles, boulders but less likely an impact than caissons or H piles because of the smaller diameter of micropile and installation method -Less geotechnical resistance than caissons or piles 	Expensive	-Feasible but more expensive than driven H-piles and drilled caissons. May be an attractive solution in view of the existing structure and foundations.
CFA (continuous flight auger pile)	<ul style="list-style-type: none"> - Rapid installation accelerates foundation construction, which reduces project schedules -Hydrostatic uplift can be counter-balanced by concrete or a sand cement mix - Suitable for low headrooms or confined spaces if segmental augers are available -Limited installation noise and vibration for sensitive urban environments -Disposal of spoils will be problem if soil at the site is contaminated -Local installers may not have powerful enough equipment to reach desired depths below ground surface 	<ul style="list-style-type: none"> - Problems may arise during the installation due to cobbles, boulders due to the less torque of CFA machine than normal caisson installation machine. -Depth may be an issue -Quality control issue (automated monitoring equipment provides real-time quality control can overcome this issue) 	Moderate to high cost (more expensive than driven piles and less expensive than micropiles)	-Feasible upon acceptance of relatively new technology into the traditional and depending on the availability of suitable equipment capable of reaching desired depths.

Appendix G

Foundation Elements - Highway 401 and Leslie Street Interchange

Table G-1
Project Number: 37-206/1
Foundation Elements - Highway 401 and Leslie Street Interchange
TRANET0601245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	Estimated (ft)	Diameter (in)	Base Diameter vs Estimated based on table no ss-bulbs	Depth (ft)	Diameter (in)	Base Diameter (mm)	Battered	Notes
37-206/1	410	128P53 H-Pile	150	*			42.7			1 to 3	Legend and Notes Signifies values could place Not clear indicates data either not available or could not be read drawing. (Note data based on interpretation of drawings and/or tables. No as-built construction records were located. Two piles went to 42 ft depth during the installation
	418	128P53 H-Pile	88	*			25.8				
	419	Concrete Caisson	n/a	n/a	36		n/a			1 to 6	working load 250 Ton/Caisson
	420	128P53 H-Pile	n/a	n/a			n/a			1 to 5	Design load 60 Ton/Pile
	421	128P53 H-Pile	n/a	n/a			n/a				Design load 60 Ton/Pile
	422	Concrete Caisson	n/a	n/a	36						working load 250 Ton/Caisson
	423	Concrete Caisson	81	n/a	30	42	24.7	762	1067		working load 250 Ton/Caisson
	424	Concrete Caisson#2	78	n/a	30	42	23.8	762	1067		working load 250 Ton/Caisson
	425	128P53 H-Pile	82		30	42	25.0	762	1067		caisson blow-out/Replaced by H-Pile, 60 Ton/Pile
	426	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	427	Concrete Caisson#2	78		30	42	24.7	762	1067		working load 250 Ton/Caisson
	428	Concrete Caisson#1	77		30	42	23.8	762	1067		working load 250 Ton/Caisson
	429	Concrete Caisson#2	77		30	42	23.5	762	1067		caisson blow-out/Replaced by H-Pile, 60 Ton/Pile
	430	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	431	Concrete Caisson#2	76		30	42	24.1	762	1067		working load 250 Ton/Caisson
	432	Concrete Caisson#3	29		30	42	23.8	762	1067		working load 250 Ton/Caisson
	433	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	434	Concrete Caisson#2	77		30	42	23.5	762	1067		working load 250 Ton/Caisson
	435	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	436	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	437	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	438	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	439	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	440	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	441	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	442	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	443	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	444	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	445	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	446	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	447	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	448	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	449	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	450	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	451	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	452	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	453	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	454	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	455	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	456	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	457	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	458	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	459	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	460	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	461	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	462	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	463	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	464	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	465	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	466	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	467	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	468	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	469	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	470	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	471	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	472	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	473	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	474	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	475	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	476	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	477	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	478	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	479	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	480	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	481	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	482	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	483	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	484	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	485	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	486	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	487	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	488	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	489	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	490	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	491	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	492	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	493	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	494	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	495	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	496	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	497	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	498	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	499	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson
	500	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson

Table G-1
Project Number: TRANET0801245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated (ft)	Diameter (in)	Base Diameter ** Estimated based on cable no as-builts (in)	Depth (m)	Diameter (mm)	Base Diameter (mm)	Battered	Legend and Notes * Symbols shown/Touch state Not Clear indicates data either not available or could not be read from drawing. Note data based on interpretation of drawings and/or tables. No as-built construction records were located.	Notes
Hwy 401 over pass at Leslie St. WBL Collectors (B3 Spans)	101	12BP53 H-File	54	*	30	36	16.5	762	914	1 to 3	working load 250 Ton/caisson	
	102	Concrete Caisson	50	*	30	36	15.2	762	914		working load 250 Ton/caisson	
	103	Concrete Caisson	50	*	30	36	15.2	762	914		working load 250 Ton/caisson	
	104	Concrete Caisson	51	*	30	36	15.5	762	914		working load 250 Ton/caisson	
	105	Concrete Caisson	52.5	*	30	36	16.0	762	914		working load 250 Ton/caisson	
	106	Concrete Caisson	63	*	30	36	19.2	762	914		working load 250 Ton/caisson	
	107	Concrete Caisson	69	*	30	36	21.0	762	914		working load 250 Ton/caisson	
	108	Concrete Caisson	74	*	30	36	22.6	762	914		working load 250 Ton/caisson	
	109	Concrete Caisson	72	*	30	36	21.9	762	914		working load 250 Ton/caisson	
	110	Concrete Caisson	67	*	30	36	20.4	762	914		working load 250 Ton/caisson	
	111	Concrete Caisson	87	*	30	36	26.5	762	914		working load 250 Ton/caisson	
	112	Concrete Caisson	80.5	*	30	36	24.5	762	914		working load 250 Ton/caisson	
	113	Concrete Caisson	70.5	*	30	36	21.5	762	914		embankment failure north of the bent/working load 200 Ton/caisson	
	114	Concrete Caisson	70.5	*	30	36	21.5	762	914		embankment failure north of the bent/working load 200 Ton/caisson	
	115	Concrete Caisson	72	*	30	36	21.9	762	914		embankment failure north of the bent/working load 200 Ton/caisson	
	116	Concrete Caisson	72	*	30	36	21.9	762	914		embankment failure north of the bent/working load 200 Ton/caisson	
	117	Concrete Caisson	74	*	30	36	22.6	762	914		embankment failure north of the bent/working load 200 Ton/caisson	
	118	Concrete Caisson	80	*	30	36	24.4	762	914		embankment failure north of the bent/working load 200 Ton/caisson	
	119	Concrete Caisson									design load 60 Ton/pile	
	120	12BP53 H-File	n/a	n/a							working load 250 Ton/caisson	
121	121	12BP53 H-File	n/a	n/a	30	36		762	914		settlement problem, caisson replaced with 6 new tube piles of 230 mm diameter, also replaced the Pier	
	122	Concrete Caisson	77	*	30	36	23.5	762	914		working load 250 Ton/caisson	
	123	Concrete Caisson	76	*	30	36	23.2	762	914		working load 250 Ton/caisson	
	124	Concrete Caisson	77	*	30	36	23.5	762	914		working load 250 Ton/caisson	
	125	Concrete Caisson	78	*	30	36	23.8	762	914		settlement problem, caisson replaced with 6 new tube piles of 230 mm diameter, also replaced the Pier	
	126	Concrete Caisson	77	*	30	36	23.5	762	914		working load 250 Ton/caisson	
	127	Concrete Caisson	78	*	30	36	23.8	762	914		working load 250 Ton/caisson	
	128	Concrete Caisson	78	*	30	36	23.8	762	914		working load 250 Ton/caisson	
	129	Concrete Caisson	78	*	30	36	23.8	762	914		working load 250 Ton/caisson	
	130	Concrete Caisson	64	*	30	36	19.5			1 to 6		
	131	12BP53 H-File	64	*			19.5			1 to 6		
	132	12BP53 H-File	64	*			19.5			1 to 6		
	133	12BP53 H-File	64	*			19.5			1 to 6		
	134	12BP53 H-File	78	*			23.8			1 to 3		

Table G-1

Foundation Elements - Highway 401 and Leslie Street Interchange

Project Number:

TRANETO801245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated (ft)	Diameter (in)	Base Diameter ** Estimated based on table no as-builts (in)	Depth (m)	Diameter (mm)	Base Diameter (mm)	Battered	Notes
37-206/3 Hwy 401 overpass at Leslie St. EBL Express (16 Spans)	318	12BP53 H-File									
	319	Rect Clear									
	320	Rect Clear									
	321	Rect Clear									
	322	Rect Clear									
	323	Rect Clear									
	324	12BP53 H-File									
	325	Concrete Caisson#1	78	*	30	42	23.8	762	1067	not clear	Previously recognized as caisson, it pile design load 60 Ton
	326	Concrete Caisson#2	85		30	42	25.9	762	1067		working load 250 Ton/caisson
	327	Concrete Caisson#3	85		30	42	29.9	762	1067		working load 250 Ton/caisson
	328	Concrete Caisson#4	98		30	42	29.9	762	1067		working load 250 Ton/caisson
	329	Concrete Caisson#5	98		30	42	25.0	762	1067		working load 250 Ton/caisson
	330	Concrete Caisson#6	82		30	42	24.7	762	1067		working load 250 Ton/caisson
	331	Concrete Caisson#7	81		30	42	24.7	762	1067		working load 250 Ton/caisson
	332	Concrete Caisson#8	81		30	42	25.0	762	1067		working load 250 Ton/caisson
	333	Concrete Caisson#9	82		30	42	28.0	762	1067		working load 250 Ton/caisson
	334	Concrete Caisson#10	92		30	42	24.1	762	1067		working load 250 Ton/caisson
	335	Concrete Caisson#11	79		30	42	27.7	762	1067		working load 250 Ton/caisson
	336	Concrete Caisson#12	91		30	42	27.7	762	1067		working load 250 Ton/caisson
	337	Concrete Caisson#13	91		30	42	33.5	762	1067		working load 250 Ton/caisson
	338	Concrete Caisson#14	77		30	42	27.1	762	1067		working load 250 Ton/caisson
	339	Concrete Caisson#15	89		30	42	27.1	762	1067		working load 250 Ton/caisson
	340	Concrete Caisson#16	80		30	42	24.4	762	1067		working load 250 Ton/caisson
	341	Concrete Caisson#17	91		30	42	27.7	762	1067		working load 250 Ton/caisson
	342	Concrete Caisson#18	91		30	42	27.7	762	1067		working load 250 Ton/caisson
	343	12BP53 H-File	64	*			19.5			1 to 6	
	344	12BP53 H-File	64	*			19.5			1 to 6	
	345	12BP53 H-File	64	*			23.8			1 to 6	
	346	12BP53 H-File	78	*			23.8			1 to 6	

Table G-1
Foundation Elements - Highway 401 and Luslie Street Interchange
Project Number: TRANETO01245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated Base Diameter no as-builts	Diameter (in)	Base Diameter no as-builts based on table	Depth (in)	Diameter (mm)	Base Diameter (mm)	Battered	Notes
17-206/4	218	Not clear									
High 401 over pass at Leslie St.	219	12x25-3 H-Pile	Installed to vertical sand								
WBL Express	220	12x25-3 H-Pile	Installed to vertical till								
(16 Spans)	221	12x25-3 H-Pile	Installed to vertical till								
	222	12x25-3 H-Pile	Installed to vertical till								
	223	12x25-3 H-Pile	Installed to vertical till								
	224	12x25-3 H-Pile	Installed to vertical sand								
	225	Concrete Caisson#1	84		30	42	25.6	762	1067		Previously recognized as caisson. H pile design load 60 Ton
	226	Concrete Caisson#2	84		30	42	25.6	762	1067		working load 250 Ton/caisson
	227	Concrete Caisson#3	84		30	42	25.6	762	1067		working load 250 Ton/caisson
	228	Concrete Caisson#1	78		30	42	23.8	762	1067		working load 250 Ton/caisson
	229	Concrete Caisson#2	78		30	42	23.8	762	1067		working load 250 Ton/caisson
	230	Concrete Caisson#3	82		30	42	25.0	762	1067		working load 250 Ton/caisson
	231	Concrete Caisson#1	77		30	42	23.5	762	1067		working load 250 Ton/caisson
	232	Concrete Caisson#2	77		30	42	23.5	762	1067		working load 250 Ton/caisson
	233	Concrete Caisson#3	82		30	42	25.0	762	1067		working load 250 Ton/caisson
	234	Concrete Caisson#1	74		30	42	22.6	762	1067		working load 250 Ton/caisson
	235	Concrete Caisson#2	74		30	42	22.6	762	1067		working load 250 Ton/caisson
	236	Concrete Caisson#3	79		30	42	24.1	762	1067		working load 250 Ton/caisson
	237	Concrete Caisson#1	76		30	42	23.3	762	1067		working load 250 Ton/caisson
	238	Concrete Caisson#2	76		30	42	23.3	762	1067		working load 250 Ton/caisson
	239	Concrete Caisson#3	77		30	42	23.5	762	1067		working load 250 Ton/caisson
	240	Concrete Caisson#1	75		30	42	22.9	762	1067		working load 250 Ton/caisson
	241	Concrete Caisson#2	75		30	42	22.9	762	1067		working load 250 Ton/caisson
	242	Concrete Caisson#3	80		30	42	24.4	762	1067		working load 250 Ton/caisson
	243	12x25-3 H-Pile	64	*			19.5			1 to 6	
	244	12x25-3 H-Pile	64	*			19.5			1 to 6	
	245	12x25-3 H-Pile	64	*			19.5			1 to 6	
	246	12x25-3 H-Pile	78	*			23.8			1 to 3	

Table G-1
Project Number: Foundation Elements - Highway 401 and Leslie Street Interchange
TRANET0001245AA

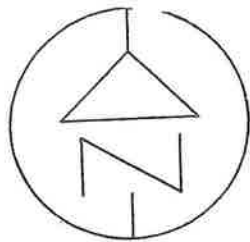
Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated (ft)	Diameter (in)	Base Diameter ** Estimated based on table no as-builts (in)	Depth (m)	Diameter (mm)	Base Diameter (mm)	Battered	Legend and Notes Signatures went back place Not Clear indicates data either not available or could not be read drawing. Here date based on interpretation of drawings and/or tables. No as-built construction records were located. One pile went to 45 m depth during the installation working load 250 Ton/caisson	Notes
37-206/5 Hwy 401 over pass at Leslie St. RAMP W-N/S (6 Spans)	619	128P53 H-Pile Concrete Caisson	50	*	30	42	27.4	762	1067	1 to 3		
	620	128P53 H-Pile Concrete Caisson	65	*	30	42	25.9	762	1067	1 to 6		
	621	128P53 H-Pile Concrete Caisson	60	*	30	42	18.3	762	1067	1 to 6		
	622	128P53 H-Pile Concrete Caisson	56	*	30	42	17.1	762	1067	1 to 6		
	623	128P53 H-Pile Concrete Caisson	77	*	30	42	23.5	762	1067	1 to 3		
37-206/6 Hwy 401 over pass at Leslie St. RAMP N-E (2 Spans)	624	128P53 H-Pile Concrete Caisson	88	*	30	42	25.8	762	1067	1 to 3		
	728	128P53 H-Pile Concrete Caisson	84	*	30	42	25.6	762	1067	1 to 3		
	729	128P53 H-Pile Concrete Caisson	75	*	30	42	22.9	762	1067	1 to 3		
	739	128P53 H-Pile Concrete Caisson	75	*	30	42	22.9	762	1067	1 to 3		
	739	128P53 H-Pile Concrete Caisson	75	*	30	42	22.9	762	1067	1 to 3		
37-206/7 Hwy 401 over pass at Leslie St. RAMP N-W (6 Spans)	518	128P53 H-Pile Concrete Caisson	74	*	30	36	22.6	762	914	1 to 6		
	519	128P53 H-Pile Concrete Caisson	72.5	*	30	36	22.1	762	914	1 to 6		
	520	128P53 H-Pile Concrete Caisson	72.5	*	30	36	22.1	762	914	1 to 6		
	521	128P53 H-Pile Concrete Caisson	77	*	30	36	23.5	762	914	1 to 6		
	523	128P53 H-Pile Concrete Caisson	88	*	30	36	26.8	762	914	1 to 6		

Appendix H

Construction Plan Drawing

North York
General Hospital

Don River



VIADUCT AND RAMP
REPLACED IN SEPARATE
CONTRACT

PROPOSED TWO SPAN
RIGID FRAME STRUCTURE

PROPOSED CELL
STRUCTURE FOR
GO COMMUTER
PARKING LOT
(438 CARS)

R-55
A-55

A-185

Ramp S-W
Ramp E-N/S

R-1500

01 WB Express

wy 401 EB Express

Hwy 401 EB Collector

R-6000

RETAINING WALL & NOISE BARRIER

PROPOSED NOISE BARRIER

PROPOSED RETAINING WALL & NOISE BARRIER

A-130

Woodsworth Road

R-50
A-55

R-1500

A-85

R-130

PROPOSED SLAB
ON STEEL GIRDER
STRUCTURES

PROPOSED RIGHT-OF-WAY

PROPOSED NOISE BARRIER

PROPOSED SINGLE SPAN
RIGID FRAME STRUCTURE

PROPOSED PEDESTRIAN CROSSING

Orléans Go Station

Appendix I

List of OPSDs, OPSSs and NSSPs

OPSD

OPSD 3000.100 Foundation Piles Steel H-pile Driving

OPSD3101.150 Walls, Abutment, Backfill Minimum Granular Requirement

OPSD3101.200 Walls, Abutment, Backfill Rock

OPSS

OPSS 539 - Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures

OPSS 903 - Construction Specification for Deep Foundations

NSSP

NSSP – Vibration Monitoring

NSSP-Caisson Piles

VIBRATION MONITORING - Item No.

Special Provision

The vibration monitoring equipment shall be placed on the existing and newly built 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 competent materials.

The results shall be certified by the Quality Verification Engineer 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 the seating on the competent materials. 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 and newly built structure are within acceptable levels. The above process must be repeated for each pile.

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 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.

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 and boulders are encountered and/or dewatering is required for the installation of the caisson piles, there will be no additional cost to the Contract.

Appendix J

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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.