

**PRELIMINARY FOUNDATION  
INVESTIGATION AND DESIGN REPORTS  
CN RAIL OVERPASS STRUCTURE  
HIGHWAY 401 REHABILITATION FROM  
LESLIE STREET TO WARDEN AVENUE  
MTO CENTRAL REGION, G.W.P. 2130-01-00  
GEOCRES 30M14-330**

Delcan Corporation  
Project: TRANETO01245AA-AB  
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  
CN Rail Overpass Structure, Highway 401 Rehabilitation from Leslie Street to  
Warden Avenue, MTO Central Region, G.W.P. 2130-01-00**

Please find attached our preliminary foundation investigation report relating to the above noted site.

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

For and on behalf of Coffey Geotechnics Inc.



**Ramon Miranda, P.Eng.**  
Principal Engineer

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**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
CN RAIL 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
- Viaduct (northwest quadrant of Highway 401 and Leslie Street interchange)
- New Leslie Street overpass structure (two span rigid frame structure)

Since 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.

Based on the information provided to us by Delcan, the existing CN Rail (C.N.R.) overpass structure which retains Highway 401 traffic over the existing rail track, will be replaced with a six segment rigid frame structure. Total length (perpendicular to centreline of Highway 401) of this proposed C.N.R. overpass structure will be about 195 m. Each single span rigid frame will be about 23.5 m wide and 8 m high (outer cross-sectional dimension), each segment with a variable length.

This preliminary foundation investigation report is prepared for the proposed new C.N.R. overpass structure. Only preliminary general arrangement drawing of the new structure was available at the time of

preparing this report. Details of the new Highway 401 structures over the CN track(s), and the applicable foundation details, will be developed during detail design.

It should be pointed out that foundation and construction details of the existing and proposed structure were not available at the time of preparing this report.

## **2 SITE DESCRIPTION AND GEOLOGY**

The site is located generally to the west of Leslie Street in the vicinity of the existing CN tracks, 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 Regions 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 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. The following are the available information / reports at the newly proposed C.N.R. overpass structure 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 St. 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 G4, G8, G9, G11 and G12 were advanced close to the newly proposed C.N.R. overpass structure.

❖ **Geocon Limited, Soil Conditions and Stability, Proposed Embankment, Leslie St. & Hwy. 401, S7002, April 8, 1960.**

The purpose of this study was to assess if there had been strength gain in the underlying clay soils as a result of embankment loading and comment on if a reduction of the previously recommended berm requirements could be altered. Three (3) boreholes were advanced in proximity to the previously drilled boreholes (Designated as Boreholes 1, 2 and 3 on Drawing 1) Boreholes 1 and 2 were advanced close to the newly proposed C.N.R. overpass structure.

❖ **Department of Highways Ontario, Foundation Investigation Report for Structures on Leslie St. & Hwy. 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 B4, B10, B11, B14 and B18 were advanced close to the newly proposed C.N.R. overpass structure.

Record of Borehole Sheets for all above mentioned boreholes which were advanced close to the newly proposed C.N.R. 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 newly proposed Highway 401 C.N.R. overpass structure, about 200 m west of 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 1 in Appendix C). The stratigraphy was based on the borehole logs and descriptions provided in the various reports.

Note 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 subsurface profiles along the West Bound, Core and East Bound Collector Lanes were 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 newly proposed Highway 401 C.N.R. overpass structure, based on a summary of the existing data. The following 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 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 C.N.R. Bridge area.

It should also pointed out that 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 investigation.

Table 3.2.1.1 is summary of the available boreholes at the newly proposed C.N.R. 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
GEOCON, 1960	1	144.9	123.1	No
	2	140.8	123.7	No
Department of Highways Ontario, 1964	B4	140.5	115.8	No
	B10	140.2	117.0	No
	B11	142.0	117.3	No
	B14	140.5	117.5	No
	B18	140.7	120.4	No

Company / Year of Investigation	Borehole No.	Existing Ground Surface Elevation (m)	Bottom Elevation of Boreholes (m)	Piezometer
The Foundation Companies of Canada, 1953	G4	140.5	121.8	No
	G8	141.2	122.5	No
	G9	140.0	118.5	No
	G11	138.2	120.3	No
	G12	140.0	112.9	No

Table 1 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 B10, B18 and G4) to depths ranging from approximately 0.1 m to 4.4 m below the ground surface, at the time of the explorations, or to elevations of approximately El. 141.9 m to 137.8 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 1.2 m, based on past explorations, extending to an average elevation of about El. 139.9 m.

The fill and/or clayey silt material at the site has been described as brown to grey sand, some gravel and clayey silt, some sand. At Borehole G8, a buried topsoil was encountered below the fill.

The fill for the most part was described as a basically granular (i.e. non-cohesive) soil. However, some fill material and the upper clayey silt soils were described as exhibiting some apparent cohesion, due to their clay content. The clayey silt was considered to be basically a cohesive material.

Standard Penetration Tests (SPT) 'N'-values of 3 to 43 blows/0.3 were recorded within the fill indicating a very loose to dense relative density in the basically granular fill. These values indicated that the fill, in some places had not received 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 B10, B18 and G4), a surficial granular soil, which ranges from sand, silt and sand to clayey silt and sand, was encountered approximate elevations of 142 m to 138 m and Table 1 in Appendix C. The thickness of the surficial granular soils encountered ranged between 3.5 and 5.4 m, with an average of approximately 4.6 m.

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

SPT 'N'-values of 5 to 83 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 clay with layers of silt (Geocon Report, 1960 and The Foundation Companies, Canada Report, 1953) to silty clay (Department of Highways Ontario, 1964) was encountered, typically below the surficial granular soils, at El. 137 to 134 m (average El. 135 m). Underlying the clay with layers of silt, an about 5 to 7 m thick clayey silt till was encountered at about El. 131 to 127 m (Geocon investigation, 1960 and The Foundation Companies investigation, 1953). This clayey silt till layer was not recognized as a separate unit in some past investigations at Highway 401 and Leslie interchange area 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 soil deposit from foundation engineering viewpoint based on the strength parameters measured by in-situ and laboratory tests. This cohesive deposit (combined upper and lower zones) was found to be approximately 12.3 to 14.5 m thick (average 13.3 m).

Atterberg Limits tests performed on samples from the deposit indicated the following index values (See Tables B-1, B-2 and B-3 and individual borehole log sheets in Appendices A and B):

Liquid Limit: 16 - 69 % (average 41% for upper clayey silt zone and 28% for lower clayey silt till zone)

Plastic Limit: 10 – 27 % (average 19% for upper clayey silt zone and 14% for lower clayey silt till zone)

Plasticity Index: 6 – 42 (average 22% for upper clayey silt zone and 14% for lower clayey silt till zone)

The above values are characteristic of a clayey soil of low to high plasticity. However, 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 only a few Standard Penetration Tests were performed within this deposit. SPT 'N'-values of 4 to 19 blows/0.3 m were recorded in this deposit. Field vane tests were also carried out within this cohesive soil deposit resulting in undrained, in-situ shear strengths of 10 kPa to 50 kPa with an average sensitivity of 2.7. Based on these test results and a tactile evaluation by others, the silty clay deposit was considered to have a consistency of soft to very stiff. Typically, the lower strength values were obtained in the upper zone and the higher values in the lower zone of glacial till origin.

Unconfined compression tests were also carried out in the laboratory on samples from this stratum resulting in unconfined compressive strength (UCS) of 11 kPa to 35 kPa. In general, the measured values were substantially lower than the vane results probably indicating disturbance of the collected samples.

Laboratory undrained triaxial tests were also conducted on samples from this stratum. In general, the measured values were substantially lower than the vane results probably indicating disturbance of the collected samples.

Figure B-1 in Appendix B presents results of past consolidation tests conducted on samples collected from this cohesive deposit.

Available in-situ and laboratory test results (Tables B-1, B-2 and B-3 and Figures B-1, B-2, B-3 and B-4) are included in Appendix B.

A 0.6 to 1.8 m thick very loose to compact granular soil (silty sand to sand) was encountered at the interface of the clay with layers of silt with clayey silt till at about El. 130 and 129 m in Boreholes 1 and G8.

Due to their mode of deposition, the presence of cobbles and boulders can be anticipated in the lower (clayey till) stratum.

### **3.2.5 Granular Glacial Till**

Underlying the cohesive soil deposit, a granular glacial till was encountered in the boreholes at depths ranging from about 17 m to 21 m below the existing ground surface at the time of the explorations or at elevations of approximately El. 124 m to 120 m (Average El. 122 m). The till deposit was described as a heterogeneous mixture of clayey silt, sand and trace of gravel (Department of Highways Ontario, 1964) to sandy silt containing cohesive layers/lenses and boulders (Geocon Report, 1960 and The Foundation Companies, Canada Report, 1953).

In general however, the till was 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.

SPT 'N'-values ranging from 32 to in excess of 100 blows/0.3m were recorded within the till indicating a dense to very dense condition. Some of the high 'N'-values may be recorded due to the presence of cobbles and boulders.

Due to their mode of deposition, the presence of cobbles and boulders should always be anticipated in the till stratum.

Some of the boreholes drilled elsewhere of the interchange indicate the presence of granular interglacial soils with significant upward groundwater gradient, underlying (or interbedded with) this glacial till deposit. This may be the case here too. Consideration may be given to drilling deeper boreholes to investigate this aspect in the detail design phase.

### 3.2.6 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 3.2.6.1: Groundwater Conditions**

Borehole	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Remarks
1	144.9	-	5.8/139.1*	Nov 2, 1959	
2	140.8	-	-	-	No information
B4	140.5	-	3.4/137.1*	1964	
B10	140.2	-	2.6/137.6*	1964	
B11	142.0	-	1.9/140.1*	1964	
B14	140.5	-	3.7/136.8*	1964	
B18	140.7	-	2.0/138.7*	1964	
G4	140.5	-	1.6/138.9*	Sept 8, 1953	
G8	141.2	-	0.0/141.2*	Sept 12, 1953	
G9	140.0	-	1.9/138.1*	Sept, 1953	
G11	138.2	-	0.3/137.9*	Sept, 1953	
G12	140.0	-	1.6/138.4*	Sept 22, 1953	

\* may not be stabilized

As can be seen from the table, the observed groundwater levels ranged between elevations of approximately 137 and 141 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 300 m east of the site, 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 16, 2009 through January 06, 2010. Of these boreholes three are located close to the newly proposed C.N.R. 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/C.N.R. Ramp W-N/S (# 37-206/5)					
W3	4847313.9	315652.0	144.8	26.3	No
Highway 401 Overpass at Leslie Street/C.N.R. Ramp N-W (# 37-206/7)					
N3	4847420.9	315653.1	142.4	27.9	No
N4	4847415.3	315597.1	144.2	24.7	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 soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

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 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 completion in the open boreholes. The boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

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

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, 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, three (3) boreholes (Boreholes N3, N4 and W3) were advanced adjacent to the newly proposed C.N.R. overpass structure.

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 three boreholes comprises fill materials and surficial granular soil deposits overlying typically firm to stiff silty clay, which is, in turn, underlain by cohesive and non-cohesive glacial till deposits. Boreholes N4 and W3, were terminated within the glacial till deposits, while the glacial till deposits are further underlain by a basal granular soil consisting of gravelly sand in Borehole N3. Borehole N3 was terminated within this basal granular soil deposit.

Details of the sub-surface conditions encountered in Boreholes W3, N3 and N4 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**

Boreholes W3, and N4 contacted a 0.1 m thick topsoil layer at the ground surface.

#### **3.4.2 Fill**

Boreholes N3, N4 and W3 contacted fill extending to depths of 3.8 to 5.5 m below the ground surface or to El. 139.7 to 138.6 m.

In Borehole N3, the top 1.4 m of the fill was found to consist of gravelly sand. Below this granular fill in Borehole N3, and the topsoil in Boreholes W3 and N4, the fill typically consists of fine grained granular materials of sandy silt to silty sand. The fill also contains zones of cohesive soil and trace to some gravel and cobbles. Some rootlets and organics have been encountered at the bottom portion of the fill in Boreholes N3 and W3. Borehole N4 also contacted a peaty topsoil layer at the interface of the fill with the native soil.

The grain-size distribution of a sample from the upper granular (i.e. gravelly sand) fill is given in Figure B-5, in Appendix B, with the following grain-size distribution, 28% gravel, 53% sand and 19 % silt and clay size particles.

The grain-size distribution of two samples from the cohesive zones within the fill is presented in Figure B-6, in Appendix B, with the following grain-size distribution, 0-4% gravel, 30-32% sand, 39-41% silt and 25-29% clay size particles.

The Atterberg limits test performed on two samples from the cohesive zones is given in Figure B-7 in Appendix B. The test yielded the following index values:

Liquid Limit:	18 – 24%
Plastic Limit:	12 – 14%
Plasticity Index:	6 – 10

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

Standard penetration tests performed in the basically granular fill (i.e. gravelly sand) encountered in the top 1.4 m in Borehole N3 yielded N-values of 31 and 39 blows/0.3 m, indicating a dense condition. In the fine grained granular fill (i.e. sandy silt to silty sand), which constitutes the bulk of the fill materials the recorded N-values range from 5 to 37 blows/0.3 m. These results indicate that the relative density of the fine grained granular fill can be described as very loose to dense but typically compact, while the general consistency of cohesive zones of the fill can be described as stiff to very stiff. These results indicate that 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.

### **3.4.3 Surficial Granular Soils**

Underlying the fill, Boreholes N3, N4 and W3 contacted surficial granular (non-cohesive) soils consisting of sand, silty sand to sandy silt and silt.

#### **3.4.3.1 Sand**

Below the fill, Boreholes N3 and N4 contacted a 1.3 to 2.3 m thick surficial sand deposit at depths of 3.8 and 4.5 m below the ground or at El. 139.7 and 138.6 m. The sand deposit was found to extend to depths of 5.1 and 6.8 m below the ground surface or to El. 137.4 and 137.3 m. The sand layer contains traces to some silt, some gravel.

The grain size distribution of two samples from the sand deposit was determined in laboratory which showed 0% gravel, 85-86% sand and 14-15% silt and clay size particles (see Figure B-8 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  $5 \times 10^{-3}$  cm/s. The sand is considered to be a granular (non-cohesive) soil type.



Standard Penetration tests performed in the deposit yielded N-values of 8 to 46 blows/0.3 m, indicating a loose to dense, but generally compact to dense relative density.

#### **3.4.3.2 Silty Sand to Sandy Silt**

Below the fill in Borehole W3, a relatively finer granular soil (i.e. silty sand to sandy silt) was encountered at a depth of 5.5 m or El. 139.3 m and was found to extend to a depth of 8.6 m or El. 136.2 m. The silty sand to sandy silt deposit contains traces of clay and the lower portion of this deposit exhibits a dilatant behaviour in the presence of water.

The grain size distribution of a sample from the silty sand to sandy silt deposit was determined in laboratory which showed 0% gravel, 23 % sand and 70 % silt and 7% clay size particles (see Figure B-9 in Appendix B). Based on the grain size distribution curves, the deposit is considered to be less pervious than the sand materials discussed in Section 3.4.3.1. N-values recorded range from 15 to 28 blows/0.3 m which indicates a compact condition.

#### **3.4.3.3 Silt**

Below the sand in Boreholes N3 and N4, a 1.6 and 2.7 m thick silt deposit was contacted at depths of 5.1 and 6.8 m or El. 137.4 and 137.3 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.

Two grain size analyses were carried out on representative samples of the silt deposit. The results are presented in Figure B-10 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:	8 – 23%
Silt:	66 -84 %
Clay:	8-11%

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.

Standard penetration test conducted in this surficial silt deposit gave N-values ranging from 3 to 17 blows/0.3 m which indicate very loose to compact relative density.

#### **3.4.4 Silty Clay**

Boreholes N3, N4 and W3 contacted a 7.9 to 13.1 m thick silty clay deposit below the surficial granular soils deposit, at depths ranging from 7.8 to 8.6 m or at El. 136.2 to 134.6 m. The deposit was found to extend to depths of 16.5 m to 20.9 m below the ground surface or to El. 128.3 to 121.5 m. This cohesive soil deposit frequently has a varved like or stratified structure and contains thin seams of silty sand to sandy silt and clayey silt. The presence of traces to some sand and gravel is also noted within the deposit.

The grain-size distribution of five samples from the silty clay deposit is given in Figure B-11 in Appendix B. This indicates the following grain-size distribution:

Gravel:	0 – 4%
Sand:	1 – 24%
Silt:	33 – 56%
Clay:	33 – 57%

The results of Atterberg limits test, performed on seven samples from the deposit is presented in Figure B-12 in Appendix B. The test results yielded the following index values:

Liquid Limit:	19 – 48%
Plastic Limit:	12 – 23%
Plasticity Index:	7 – 25
Natural Moisture Content:	15 – 38 %

These results are characteristic of clayey soils of low to medium plasticity and the fact that measured natural moisture contents are generally near the measured liquid limit indicates the likelihood of an only slightly over-consolidated soil deposit.

One oedometer (one-dimensional consolidation) test was performed in the laboratory on a thin-walled tube (TW) sample (from Borehole N3). The results are presented in Figure B-13 in Appendix B. These show a possible pre-consolidation pressure similar to the existing overburden pressure (i.e. normally consolidated). It should be pointed out that the presence of silty sand to sandy silt and clayey silt in the sample may have affected consolidation test results because the above mentioned soils are typically less compressible than the silty clay itself. The test indicates a compression index  $C_c$  of about 0.3 and rebound value  $C_r$  of 0.05, as well as a coefficient of consolidation  $C_v$  of about  $1 \times 10^{-3} \text{ cm}^2/\text{s}$ . The measured bulk unit weight of the sample is  $18.4 \text{ kN/m}^3$ .

Standard penetration test conducted in the silty clay deposit gave N-values which range from 1 to 9 blows/0.3 m. but typically 1 to 6 blows/0.3 m. Undrained-shear strengths as measured by MTO “N” type field were varied from 30 to 85 kPa indicating a firm to stiff consistency. The variation of the undrained shear strength as measured by vane tests with elevation is given in Figure B-14. It should however be pointed out that some higher undrained shear strengths may have been obtained from the silty sand to sandy silt and clayey silt interbeds within the silty clay deposit.

### **3.4.5 Glacial Till**

Underlying the silty clay, Boreholes N3, N4 and W3 contacted a glacial till deposit consisting of a heterogeneous mixture of silty sand to sandy silt and clayey silt. Traces to some gravel and occasional cobbles were also 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, especially in the sandy silt to silty sand till. As mentioned, the composition of the deposit was found to range primarily from a basically cohesive deposit consisting of clayey silt till to granular deposit consisting of sandy silt-silty sand till, as discussed in Sections 3.4.5.1 and 3.4.5.2, below.

### 3.4.5.1 Clayey Silt Till

Boreholes N4 and W3 contacted a glacial deposit consisting of clayey silt till underlying the silty clay at depths of 16.5 and 20.8 m or at El. 128.3 and 123.4 m. Borehole N4 was terminated within the clayey silt till at a depth of 24.7 m or at El. 119.5 m. In Borehole W3, the clayey silt till was found to be 5.3 m thick and extended to a depth of 21.8 m below the ground surface or to El. 123.0 m.

The grain-size distribution of two samples from the clayey silt till deposit is presented in Figure B-15 in Appendix B. This indicates the following grain-size distribution:

Gravel:	3 – 4%
Sand:	25 - 32%
Silt:	35 – 45%
Clay:	20 – 36%

The results of Atterberg limits tests, performed on the same two samples from the clayey silt till, are given in Figure B-16 in Appendix B. The tests yielded the following index;

Liquid Limit:	21 – 26%
Plastic Limit:	14 – 15%
Plasticity Index:	7 - 11

These results are characteristics of clayey soils of low plasticity and 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 1 to in excess of 100 blows/0.3 m were recorded, showing a wide variation. In Borehole W3, where relatively weak clayey silt till was encountered, field vane tests were performed and these yielded undrained shear strengths of 45 to in excess of 100 kPa. Based on these field test results the consistency of the deposit in this borehole can be described as firm to very stiff. In Borehole N4, the recorded N-values below about El. 123 m (i.e. about 0.4 m below the interface with the overlying silty clay) are all in excess of 100 blows/0.3 m. Based on this, the clayey silt till in this borehole can be described as having a hard consistency.

It is of intent to note that an about 0.5 m soil back up was encountered in Borehole W3 at about El. 125 m during the drilling.

### 3.4.5.2 Sandy Silt to Silty Sand Till

Underlying the clayey silt till in Borehole W3 and the silty clay in Borehole N3, a naturally coarser (i.e. basically granular) sandy silt to silty sand till was contacted, at depths of 20.9 and 21.8 m below the ground surface or at or El. 123.0 and 121.5 m. Borehole W3 was terminated within this glacial till deposit at a depth of 26.3 m or at El. 118.5 m, while Borehole N3 the deposit was fully penetrated and was found to be underlain at depths/Elevations of 26.6/115.8 m, by a basal granular deposit of gravelly sand.

The grain-size distribution of three samples from the sandy silt to silty sand till deposit is given in Figure B-17. This indicates the following grain-size distribution;

Gravel:	9 – 23%
Sand:	38 - 39%
Silt & Clay:	39 - 53% (average clay content 19%)

An Atterberg limits test was performed on a sample from the deposit recovered from Borehole N3 and it was found to be non-plastic (i.e. plastic limit could not be determined).

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.

#### **3.4.6 Basal Granular Soils**

Borehole N3 encountered a gravelly sand deposit underlying the silty sand till at a depth of 26.6 m or El. 115.8 m.

Borehole N3 was terminated after 1.3 m penetration into the gravelly sand deposit.

A standard penetration test N-value of in excess of 100 blows/0.3 m was recorded, which indicates that the gravelly sand is very dense.

It should be pointed out that the presence of lower interglacial granular soils was noted in some of the previously drilled (deeper) boreholes at the site. We recommend that this aspect be kept in mind when planning for the detail investigation.

#### **3.4.7 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 3.4.7.1 Groundwater Conditions**

Borehole	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Remark
N3	142.4	-	16.8/125.6*	Upon completion	Caved-in** @ 4.3 m
N4	144.2	-	19.8/124.4*	Upon completion	Caved-in** @ 10.7 m
W3	144.8	-	13.1/131.7*	Upon completion	Caved-in** @ 7.0 m 0.5 m soil back-up noted @ El. 124.6 m, while drilling.

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

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

It should however be pointed out that these values are unlikely to represent the stabilized groundwater levels at the time of the investigation and that reference can also be made to Section 3.2.6 of this report and to Table 1 in Appendix C. Since 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 piezometer was installed close to the newly proposed CNR overpass and only groundwater observations at the completion of subsurface investigation are available. 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 about 200 m east of the Leslie Street, known to be a branch of East Don River.

For and on behalf of Coffey Geotechnics Inc.



**Gwangha Roh, Ph.D.**



**Ramon Miranda, P.Eng.**

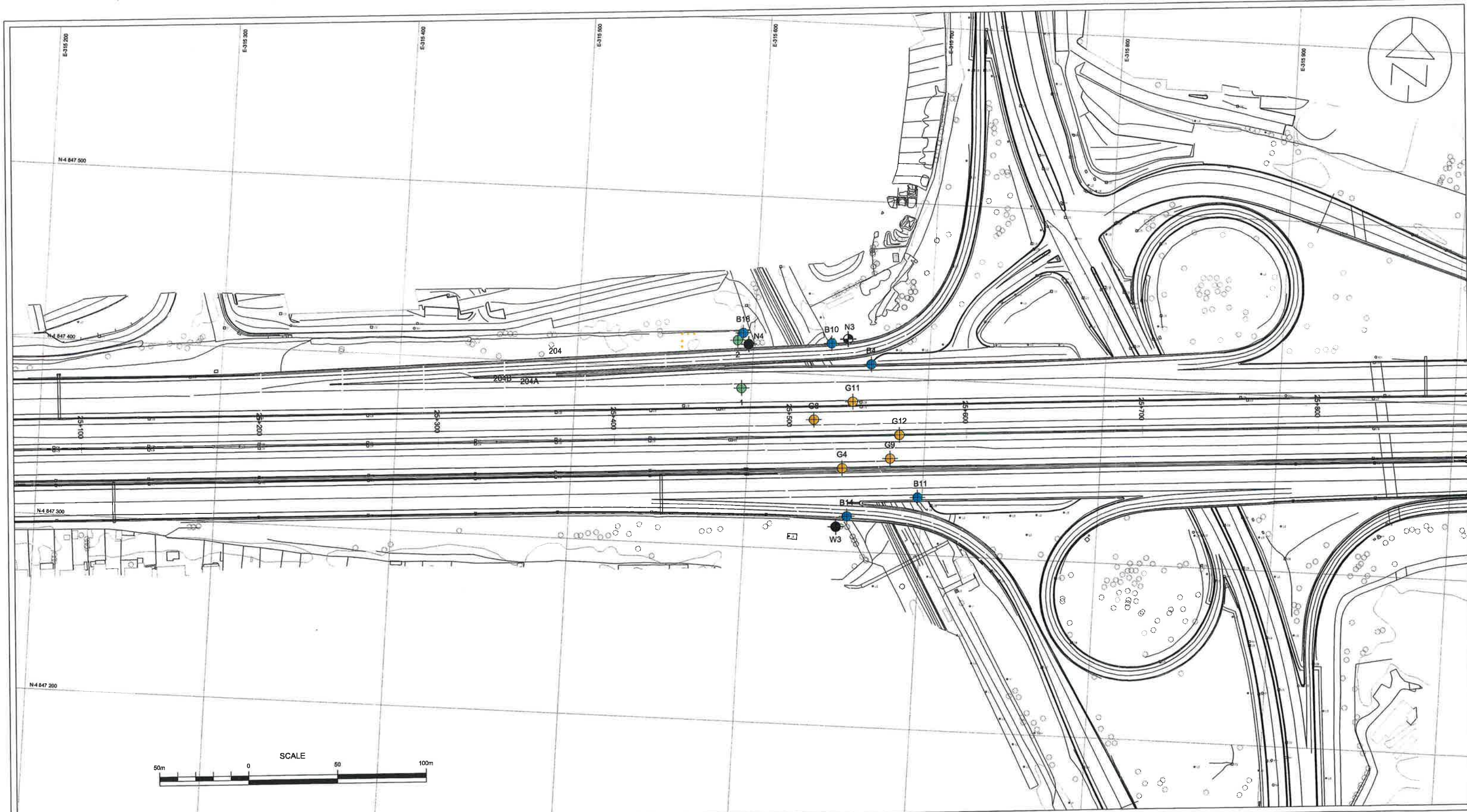


**Zuhtu Ozden, P.Eng.**



## Drawings





LEGEND	
	Borehole (Coffey, 2009)
	Borehole (Department of Highways Ontario, 1964)
	Borehole + DCPT (Coffey, 2009)
	Borehole (Geocan Ltd., 1960)
	Borehole (The Foundation Company of Canada/Geocan, 1953)

**NOTES**

1. Base plan provided by Delcan.

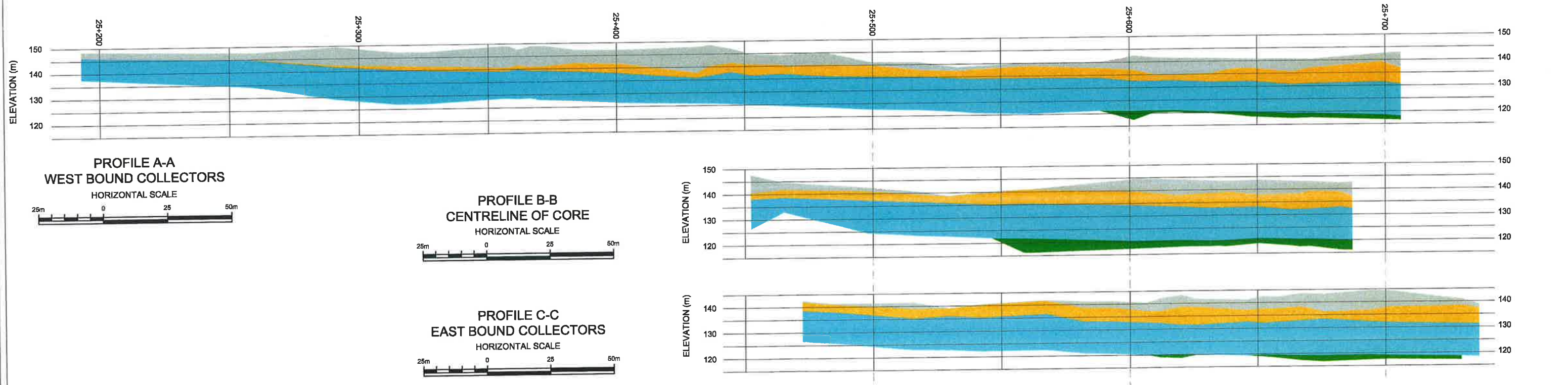
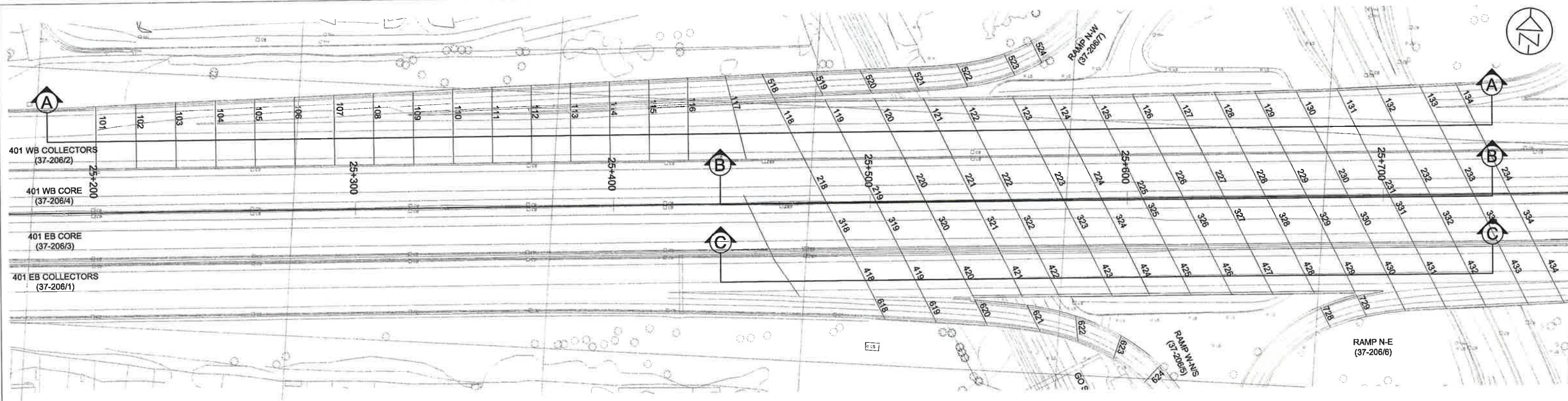
drawn	SH
approved	RM
date	Feb. 10, 2011
scale	As Shown
original size	Tabloid




**geotechnics**  
 SPECIALISTS MANAGING THE EARTH

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





**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	PHK
approved	RM
date	Mar. 10, 2010
scale	As Shown
original size	Tabloid



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

# Appendix A

## **Record of Borehole Sheets**



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 57002 BORING: ANK 1A DATUM GEODETIC CASING BX  
 BORING DATE OCT 27 - NOV 2, 1959 REPORT DATE MARCH 11, 1960 COMPILED BY M.W. CHECKED BY J.S.  
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

### SAMPLE CONDITION



A.S. - AUGER SAMPLE  
 S.T. - SLOTTED TUBE  
 W.S. - WASHED SAMPLE  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 C.S. - CHUNK SAMPLE

### SAMPLE TYPES

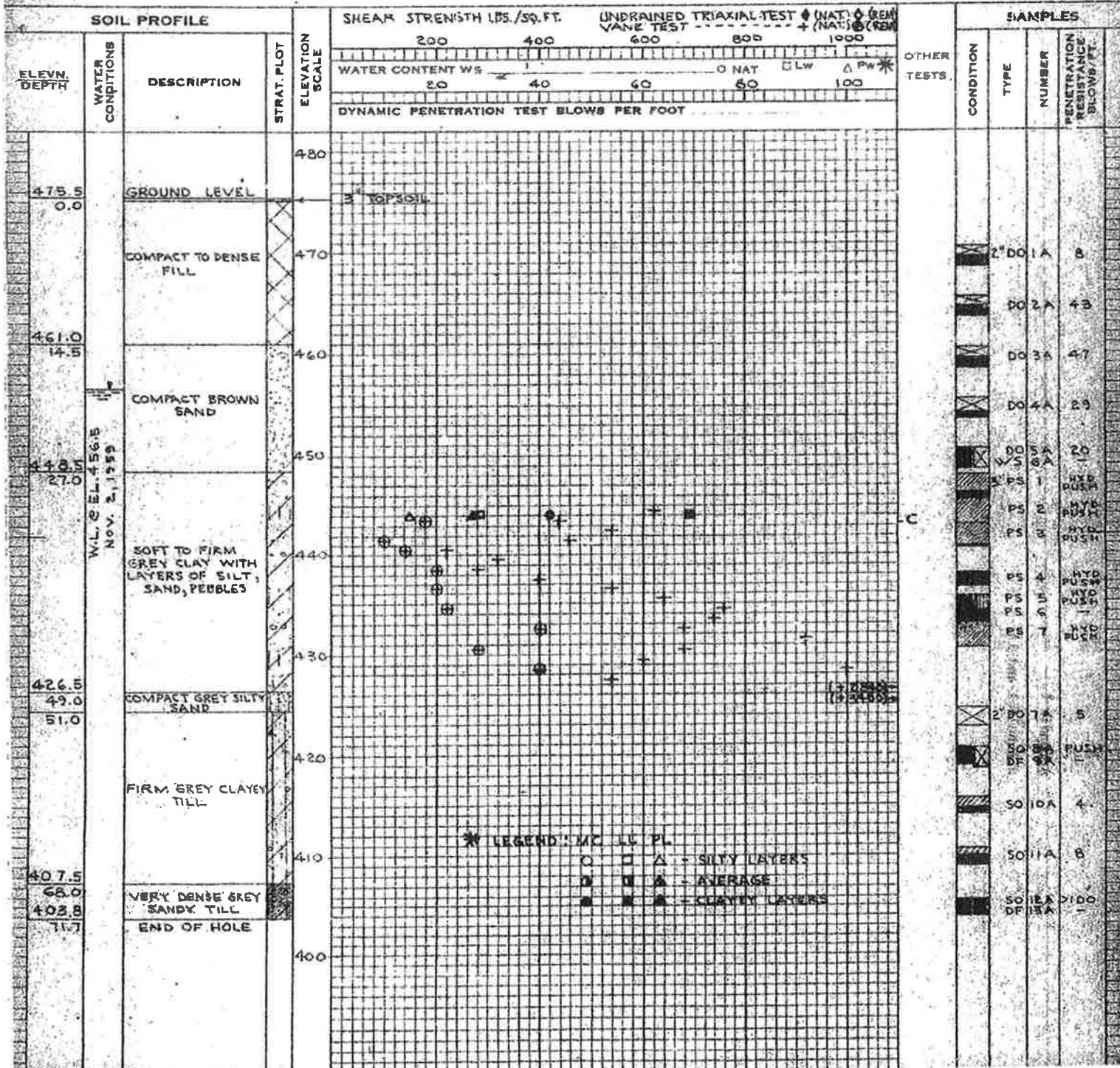
F.S. - FOIL SAMPLE  
 S.O. - SLEEVE-OPEN  
 S.F. - SLEEVE-FOOT VALVE  
 T.O. - THIN WALLED OPEN  
 R.C. - ROCK CORE  
 P.S. - PISTON SAMPLE

### ABBREVIATIONS

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

Y - WET UNIT WEIGHT  
 K - PERMEABILITY  
 C - CONSOLIDATION

WL - WATER LEVEL IN CASING  
 WT - WATER TABLE IN SOIL



# GEOCON

## OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 51002 BORING # 2 AND 2A DATUM GEODETIC CASING BA  
 BORING DATE NOV. 4, 1959 REPORT DATE MARCH 11, 1960 COMPILED BY MAN CHECKED BY MAN  
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

### SAMPLE CONDITION



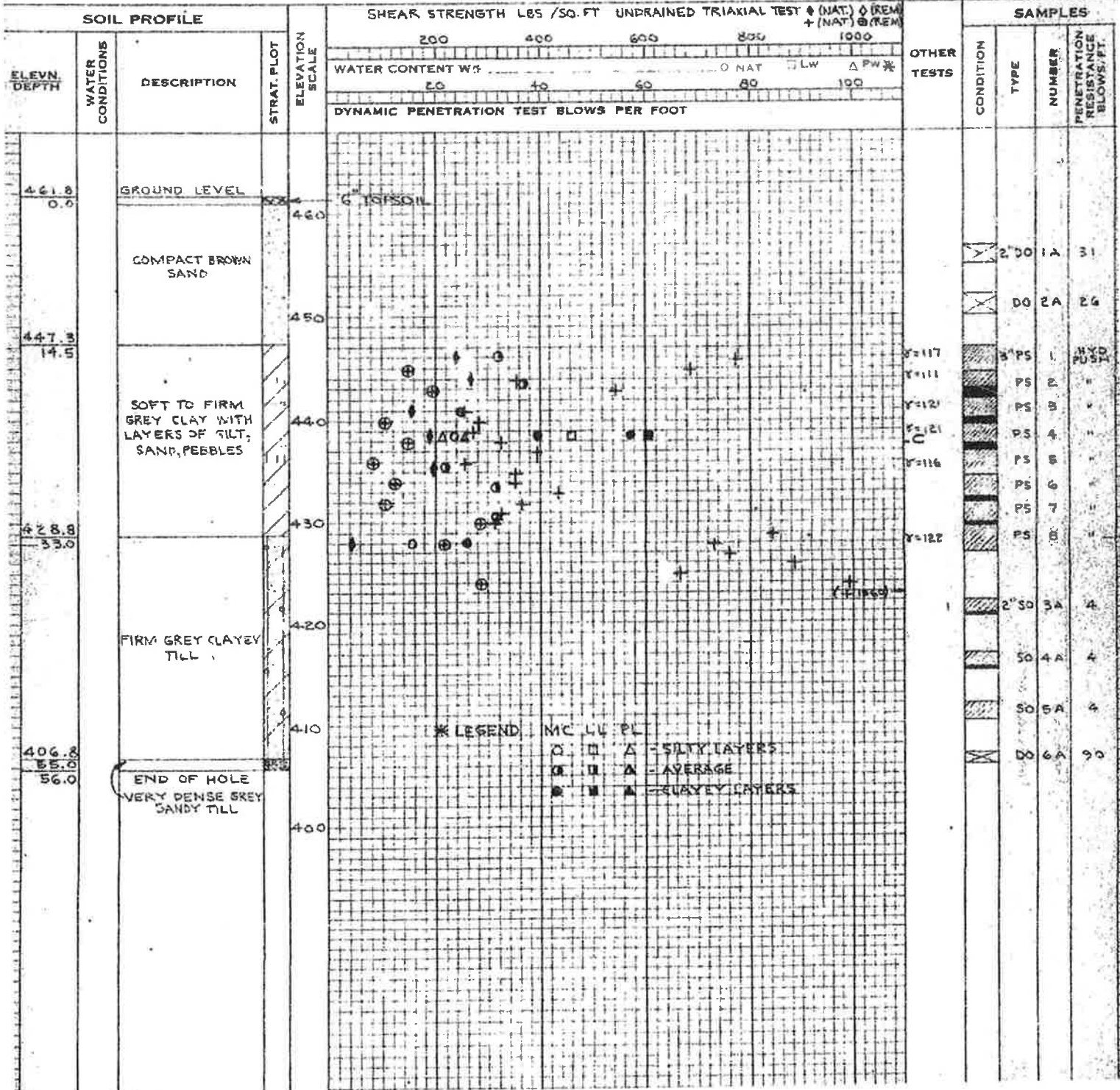
A.S. - AUGER SAMPLE  
 S.T. - SLOTTED TUBE  
 W.S. - WASHED SAMPLE  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 C.S. - CHUNK SAMPLE

### SAMPLE TYPES

F.S. - FOIL SAMPLE  
 S.O. - SLEEVE-OPEN  
 S.F. - SLEEVE-FOOT VALVE  
 T.O. - THIN WALLED OPEN  
 R.C. - ROCK CORE  
 P.S. - PISTON SAMPLE

### ABBREVIATIONS

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



B4

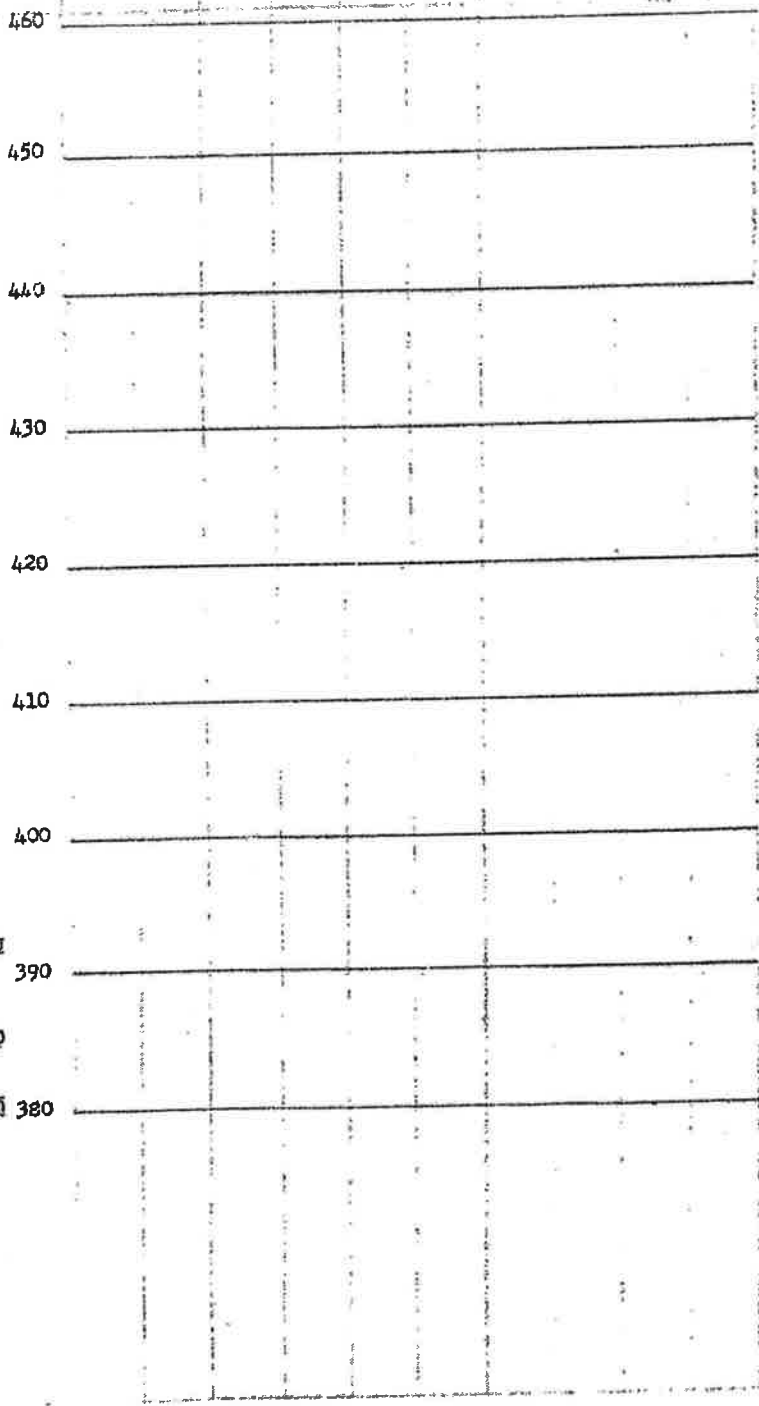
LOG 64-8-41  
W.D. 252-61-3  
DATE C.S.C.

RECORD OF BOREHOLE No. 4  
Stn. 127+75 and 133' Lt. of g. Hwy. 401  
June 3, 1964.  
Washboring using BK casing.

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

461	Groundlevel
0.0	Clay silt, sand and trace 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"
2	SS 100
	for 6"
3	SS 100
	for 6"



W.L. at  
SL. 449.8

B10

DEPARTMENT OF HIGHWAYS DIVISION MATERIALS & RESEARCH DIVISION				RECORD OF BOREHOLE NO. 10				FOUNDATION SECTION	
JOB 64-F-41		LOCATION Stn. 126+43 and 175' Lt. of E. Hwy. 401		ORIGINATED BY B.M.G.					
W.P. 252-61-3		BORING DATE June 10, 1964.		COMPILED BY B.M.G.					
DATE G.S.O.		BOREHOLE WASHBORING using BK casing.		CHECKED BY M.D.					
SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W <sub>c</sub>		BULK DENSITY P.C.F.	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLCT	NUMBER TYPE	BLOWS / FOOT ELEV SCALE	SHEAR STRENGTH P.S.F.	WATER CONTENT %			
460	Groundlevel			460					
	Clayey silt, sand and fine gravel. Brown.			450					
15.0	Silt. Br. grey			440					
17.6	Silty clay with some sand. Grey.			430					
				420					
				410					
400				400					
60.0	Heterogeneous mixt. of clayey silt, sand & gravel up to 2" (Glacial Till) V. dense Grey		1 SS 78						
			2 SS 100						
			for 4"						
			3 SS 100	390					
			for 4"						
384			4 SS 100						
76.0	End of borehole.		for 6"	380					

W.L. at  
El. 451.3



B11

DEPARTMENT OF HIGHWAYS - DIVISION OF MATERIALS & RESEARCH DIVISION		RECORD OF BOREHOLE NO. 11		FOUNDATION SECTION	
JOB <u>64-F-41</u>		LOCATION <u>Sta. 127+95 and 118' Rt. of E. Hwy. 401</u>		ORIGINATED BY <u>D.M.G.</u>	
W P <u>252-61-3</u>		BORING DATE <u>June 12, 1964.</u>		COMPILED BY <u>B.M.G.</u>	
DATUM <u>G.S.C.</u>		BOREHOLE <u>Washboring using BX casing.</u>		CHECKED BY <u>M.D.</u>	

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLCT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT - W <sub>L</sub>		BULK DENSITY P.C.F.	REMARKS
							BLOWS / FOOT	SHEAR STRENGTH P.S.F.	PLASTIC LIMIT - W <sub>P</sub>	WATER CONTENT - W		
466	Ground level											
0.6	Silty fine sand. Brown.					460						N.L. at El. 459.5
449						450						
17.0	Silty clay with some sand and gravel.  Grey.					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 Grey.		2	SS	100 for 6"							
			3	SS	12							
			4	SS	103							
			5	SS	100 for 10"	390						
385			6	SS	100 for 6"							
81.0	End of borehole.					380						

B14

DEPARTMENT OF HIGHWAYS MATERIALS & RESEARCH DIVISION				RECORD OF BOREHOLE NO. 14				FOUNDATION SECTION			
JOB 64-P-41		LOCATION Stn. 126+66 and 150' Rt. of E. Hwy. 401		ORIGINATED BY B.M.G.		COMPILED BY B.M.G.		CHECKED BY M.D.			
W.P. 252-61-3		BORING DATE June 16, 1964		BORING METHOD Washboring using BK casing							
DATUM G.S.C.											
ELEV DEPTH	DESCRIPTION	STRAT. PLCT	SAMPLE NUMBER	TYPE	BL. NO. / FOOT	ELEV. 5'	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	SHEAR STRENGTH P.S.F.	LIQUID LIMIT WL PLASTIC LIMIT WP WATER CONTENT W WATER CONTENT %	BULK DENSITY P.C.F.	REMARKS
461	Ground level										
0'-6"	Clayey silt and sand. Brown.					460					
						450					
446											
15.0	Silty clay with some sand. Grey					440					
						430					
						420					
						410					
401.7											
59.4	Heterogeneous mixt. of clayey silt, sand & trace of gravel up to 1"Ø. (Glacial till) V. dense Grey.		1	SS 100	for 6"	400					
			2	SS 100	for 4"						
			3	SS 100	for 4"	390					
385.5			4	SS 100	for 6"						
75.5	End of borehole.					380					

W.L. at  
El. 448.8



DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 18

FOUNDATION SECTION

JOB 64-F-41 LOCATION Stn. 124/80 and 194' It. of G Hwy. 401 ORIGINATED BY B.M.G.  
W P 252-61-3 BORING DATE June 22, 1964 COMPILED BY B.M.G.  
DATUM G.S.C. BOREHOLE TYPE Washboring using BK casing. CHECKED BY M.D.

SOIL PROFILE		STRAT PLOT	SAMPLES		ELEV. SCA.	DYNAMIC PENETRATION RESISTANCE		FLUID LIMIT		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION		NUMBER	TYPE		BLOWS / FOOT	BL	PL	W		
461.5	Groundlevel				460						
0.0	Clayey silt and sand. Brown.				450						
446.5					440						
15.0	Silt Grey				430						
17.0	Grey silty clay with some sand. Grey.				420						
403.0					410						
58.6	Heterogeneous mixt. of clayey silt, sand and gravel up to 2" Ø, (Glacial till) Boulder at 66'-6" V. dense. Grey.		1	SS-100	400						
395.0			2	SS-100							
66.6	End of Bore Hole										

W.L.  
at El. 455

## OFFICE REPORT ON SOIL EXPLORATION

G4

DRILL RIG: MACHINE JOB: C-1191 BORING: 1-A  
 CASING: 4" BX (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM: 100.00 DATE REPORT: AUG 24/53  
 SAMPLER HAMMER WT. 370 DROP: 18 1/2 INCHES COMPILED BY: J.C.D. CHECKED BY: G.W.E. BORING DATE: AUG 12/53

## SAMPLE CONDITION



DISTURBED  
 FAIR  
 GOOD  
 LOST

## SAMPLE TYPES

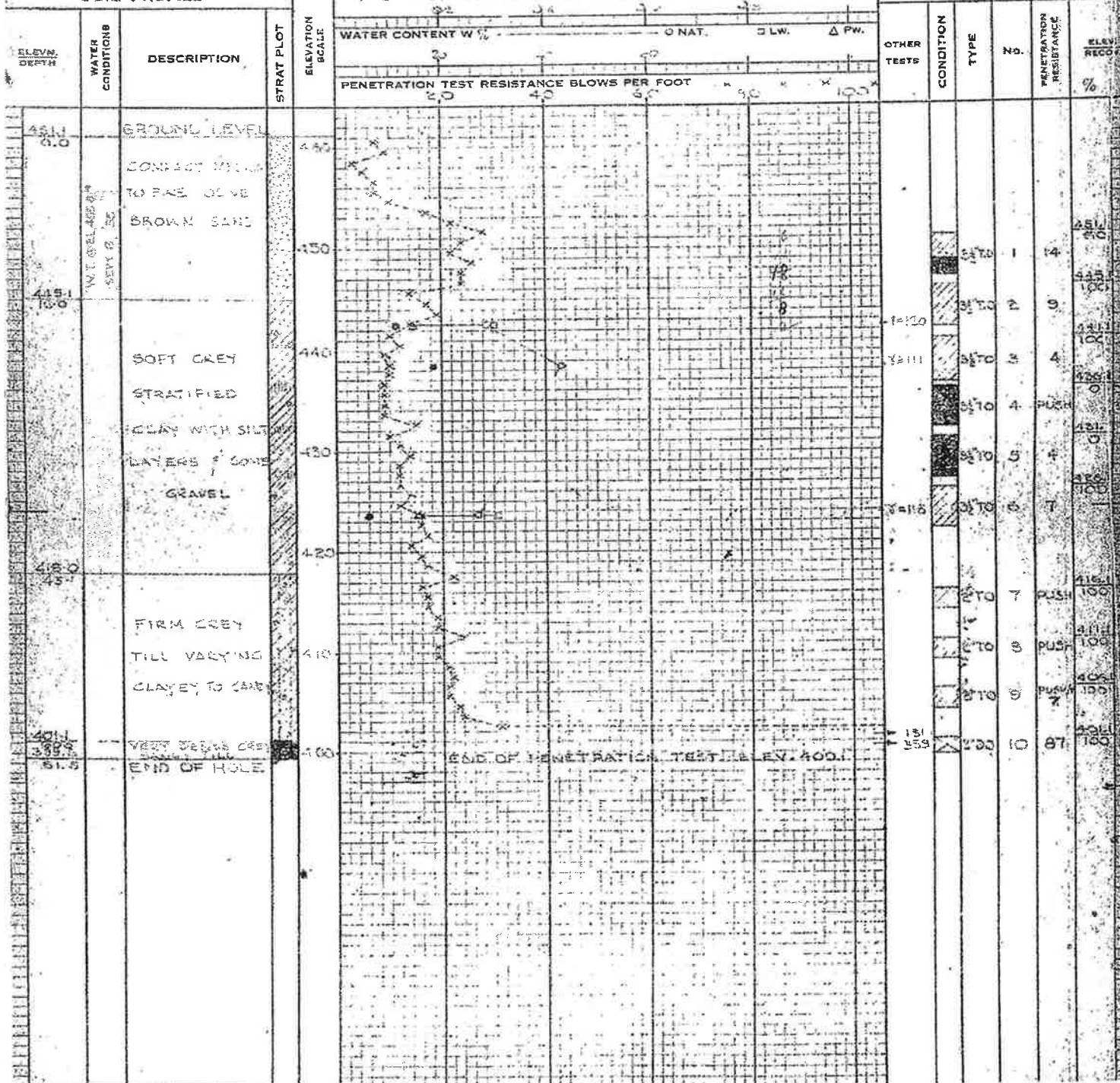
C.S. - CHUNK  
 D.O. - DRIVE-OPEN  
 D.F. - DRIVE-FOOT VALVE  
 D.P. - DRIVE PISTON  
 T.O. - THIN WALLED OPEN  
 T.P. - THIN WALLED PISTON  
 F.S. - FOIL SAMPLE  
 B.A. - BARREL AUGER  
 S.A. - SPIRAL AUGER  
 W.S. - WASHED SAMPLE  
 R.C. - ROCK CORE

## ABBREVIATIONS

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

## SOIL PROFILE

## SAMPLES



# OFFICE REPORT ON SOIL EXPLORATION

68

DRILL RIG. 10 AC-115 JOB 17147 BORING 1-B  
CASING 4 1/2 (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM 725.610 DATE REPORT SEP 10 1953  
SAMPLER HAMMER, WT. 34 DROP 3 INCHES COMPILED BY ALC CHECKED BY JWS BORING DATE SEP 10 1953

### 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

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

## SAMPLES

[illegible]





# OFFICE REPORT ON SOIL EXPLORATION

911

DRILL NO. 102-101 JOB 102-101 BORING # 101  
 CASING 3/4" (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM CRAIL DATE REPORT SEP 28 1953  
 SAMPLER HAMMER WT. 140 LBS DROP 18 INCHES COMPILED BY W. J. B. J. CHECKED BY W. J. B. J. BORING DATE 28 SEP 1953

**SAMPLE CONDITION**

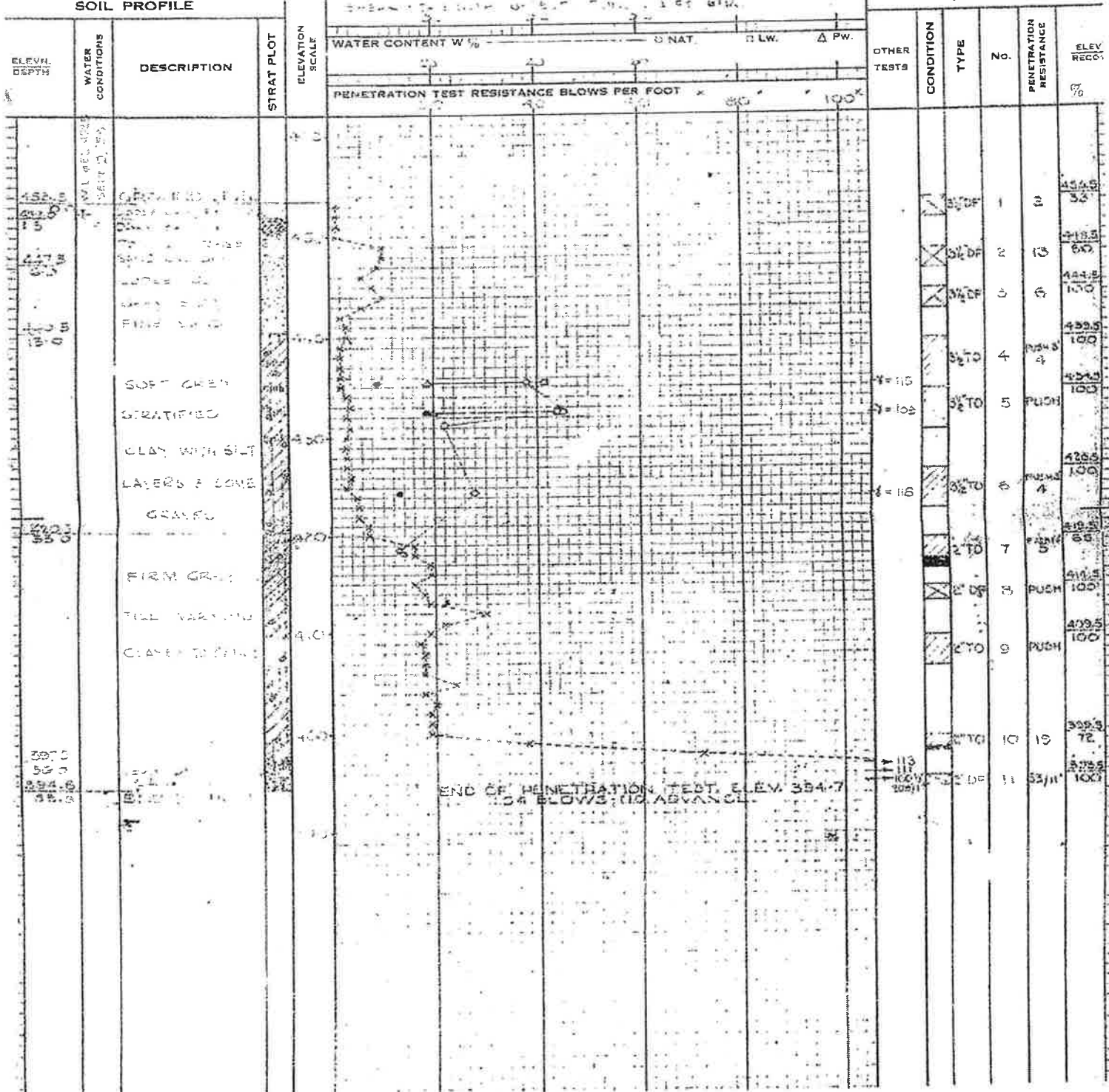

DISTURBED  
 FAIR  
 GOOD  
 LOST

**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  
 B.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  
 CC. - TRIAXIAL CONSOLIDATED QUICK  
 Q. - TRIAXIAL QUICK  
 S. - TRIAXIAL SLOW  
 γ. - UNIT WEIGHT  
 K. - PERMEABILITY  
 C. - CONSOLIDATION  
 CA. - CASING  
 WL. - WATER LEVEL IN CASING  
 WT. - WATER TABLE IN SOIL

**SOIL PROFILE**
**SAMPLES**




# RECORD OF BOREHOLE No W3

1 OF 2

METRIC

LOCATION W-S Ramp of 401 to Leslie Street (Northing 4847313.9 & Easting 315652.0)

ORIGINATED BY RK

DIST \_\_\_\_\_ HWY 401

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY SK

DATUM Geodetic

DATE 12/17/2009

CHECKED BY ZO

Continued Next Page

 $+^3, \times^3$ 

Numbers refer to  
Sensitivity

(%) STRAIN AT FAILURE



TRANETO01245AA: HWY401/Leslie St/Ramp W-S/N

# RECORD OF BOREHOLE No W3

2 OF 2

METRIC

GWP 2008-E-0012

LOCATION W-S Ramp of 401 to Leslie Street (Northing 4847313.9 & Easting 315852.0)

ORIGINATED BY RK

DIST HWY 401

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY SK

DATUM Geodetic

DATE 12/17/2009

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  Y  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
FLEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)		WATER CONTENT (%)				
							20 40 60 80 100	20 40 60 80 100		W <sub>P</sub> W W <sub>L</sub>				
							○ UNCONFINED + FIELD VANE	● POCKET PENETR X LAB VANE						
129.8 15.0	SILTY CLAY fr sand, fr gravel grey, firm, wet		15	SS	2									
128.3 16.5	some sand		16	SS	1									
	CLAYEY SILT TILL grey, firm to v stiff, moist		17	SS	2								spoon wet	
	freq. silty sand layers		18	SS	3								4 25 35 38 0.5 m soil back-up	
123.0 21.8	SANDY SILT TO SILTY SAND TILL some clay grey, v dense, moist		19	SS	29								spoon wet	
			20	SS	100 / 23 cm								9 38 35 18 Auger grinding @ 23.2 and 23.5 m	
			21	SS	100 / 10 cm								auger grinding @ 24.7 and 25.0 m	
118.5 26.3	End of Borehole Water level @ 13.1 m (not stabilized)* upon completion before auger pull out No water level was observed after auger pull out and borehole caved in @ 7.0 m.		22	SS	100 / 28 cm								spoon wet	

+ 3 x 3

Numbers refer to  
Sensitivity

20  
15 10 5

(%) STRAIN AT FAILURE

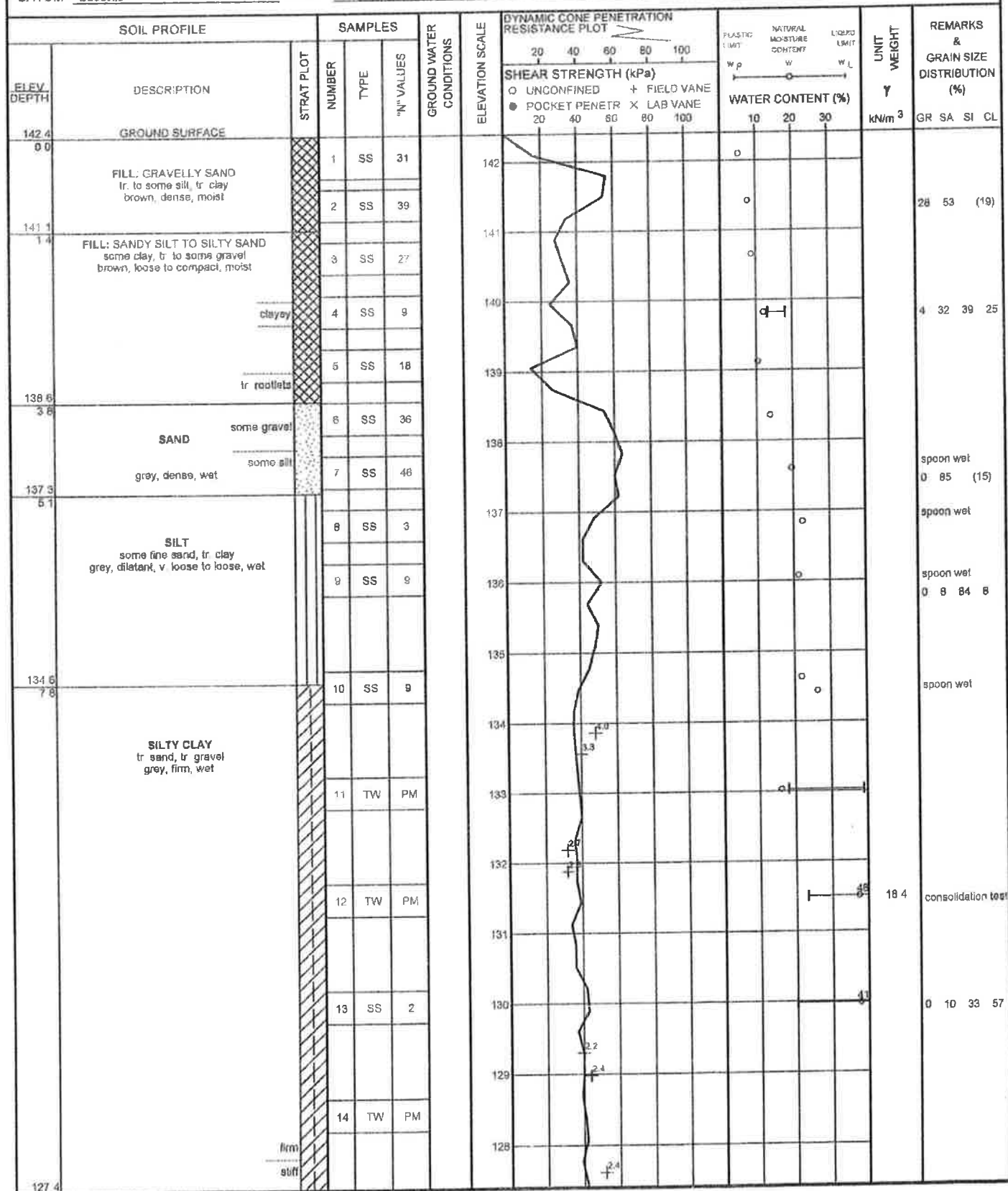
TRANETO01245AA: HWY401/Leslie SVRamp N/W

# RECORD OF BOREHOLE No N3

1 OF 2

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847420.9 & Easting 315553.1) ORIGINATED BY RK  
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK  
DATUM Geodetic DATE 11/19/2009 CHECKED BY ZO



Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

TRANETO01245AA: HWY401/Leslie St/Ramp N/W

# 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 w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
								○ UNCONFINED ● POCKET PENETR	+ FIELD VANE x LAB VANE						
127.4 15.0	SILTY CLAY tr. gravel, tr. sand grey, wet	firm stiff	15	SS	3	1	127								
			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	SS	100 / 25 cm		122								
			21	SS	100 / 13 cm		121								
			22	SS	100 / 20 cm		120								
			23	SS	100 / 29 cm		119								
115.8 26.6	GRAVELLY SAND grey, v. dense, wet						118								
							117								
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														

+ 3, x 3 Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



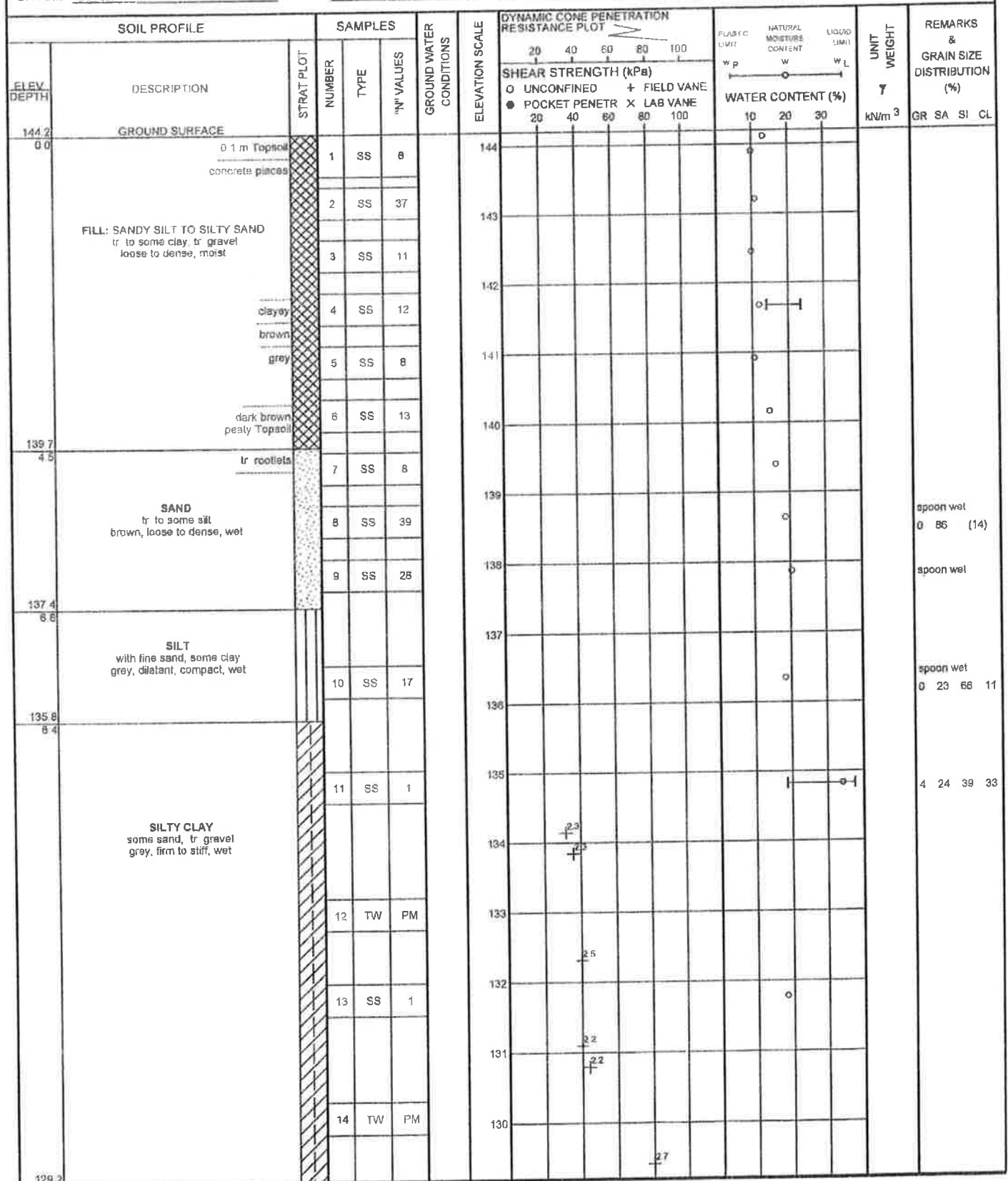
TRANETO801245AA: HWY401/Leslie St/Ramp N/W

# RECORD OF BOREHOLE No N4

1 OF 2

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847415.3 & Easting 315597.1) ORIGINATED BY RK  
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK  
DATUM Geodetic DATE 12/1/2009 CHECKED BY ZO



Continued Next Page

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

TRANETOBO1245AA HWY401/Leslie St/Ramp N/W

## RECORD OF BOREHOLE No N4

2 OF 2

**METRIC**

GWP	2008-E-0012	LOCATION	N-W Ramp of Leslie Street to 401 (Northing 4847415.3 & Easting 315597.1)	ORIGINATED BY	RK
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	SK
DATUM	Geodetic	DATE	12/1/2009	CHECKED BY	ZO

[illegible]

+ 3 X 3 Numbers refer to Sensitivity

# Appendix B

## Test Results

Table B-1 Borehole G4 (1953)

TABULATION OF RESULTS OF TESTS ON SAMPLES OF SOIL MARKED

C-7742 - BOREHOLE No. 4

Sample No.	Depth of Specimen ft.	Water content %	Consistency Limits		Unconfined Compression Strength as received ton/sq.ft.	Corresponding strain as received	Unit wet weight received lb/cu.ft.	Remarks
			Liquid Limit	Plastic Index				
1 (10'-14' (Recovery of 3')								From 10' to 10'6" fine sand with large pockets of silt; brownish grey; From 10'6" to 11'3" sand, coarse to fine, yellow. From 11'3" to 11'6" fine sand, some silt, brownish grey. From 11'6" to 11'8" sand, coarse to fine, yellow. From 11'8" to 13': very fine sand, silty, grey.
2 (15'-19')	13'-18'6"	28.7	29.2	14.3	0.23	0.20	120	From 15' to 17': very fine sand, silty, grey. From 17' to 19': clay, stratified, brownish grey, numerous thin layers and inclusions of silt, (some disturbance to sample).
3 (20'-24'6")	22'6"-23'	43.3			0.37	0.05	111	Stratified clay, grey, containing numerous inclusions and layers (up to 1/8") of silt.
6 (35'-39')	37'-37'6"	27.3	30.4	15.5	0.11	0.20	118	Clay, grey, containing few inclusions of silt. Sample very disturbed, faintly varved, 1/4" to 3/8" layers of clay with chunky structure and 1/4" to 1/2" layers of silty clay.



BENTONITE NO. 4

Sample No.	Depth of specimen ft.	Water content %	Consistency Limits		Unconfined Compression strength		Unit wt received lb/cu.ft.	Remarks
			Liquid Limit	Plasticity Index	as received	Corresponding strain		
7 (45'-47')	46'3"-46'9"	14.1			0.16	0.20	141	From 45' to 45'6": fine sand; from 45'6" to 47': mostly silt with some clay, few small pebbles and little sand.
8 (50'-52')	51'-51'6"	15.3	18.4	8.1	0.20	0.20	139	From 50' to 50'4": fine sand; from 50'4" to 52': mostly silt with some clay, few small pebbles and little sand.
9 (55'-59')	56'3"-56'9"	46.6			0.33	0.20	109	From 55' to 56': mostly silt, with fine sand and few small pebbles; from 56' to 57'6": clay, numerous thin layers of silt, 1" layer of silt at 57'4".

Table B-2 Borehole G9 (1953)

CONTINUATION OF TABLE OF TESTS ON SAMPLES OF SOIL BORED

C-7742 - CONTINUATION NO. 9

Sample No.	Depth of specimen ft.	Water content %	Consolidation Limit Liquid Plastic Shrink Limit	Unconsolidated Compressibility Test Initial Consolidation Index	Unconsolidated Compressibility Test Initial Consolidation Index	Unconsolidated Compressibility Test Initial Consolidation Index	Remarks
4 (19'-21')	21'-21'6"	36.8			0.39	0.05	118 Stratified clay, grey, 1/8" layer of clay alternating with 1/8" to 1/4" layers of silt and clay; also numerous inclusions of silt and few small pebbles; lower amount of silt from 19' to 19'6" 20'6" to 20'10", 21'6" to 22'2"; soft when remoulded.
6 (24'-29')	26'-26'6"	35.8	37.5	18.9	18.6	0.22	0.16 Clay, grey, containing numerous inclusions and thin layers of silt also few subangular gravel-size particles (up to 3/4"); soft when remoulded.
11 (34'-37'8") (Recovery of 2'4")	34'10"-35'4"	17.1	21.6	12.2	9.4	0.16	0.20 From 34' to 34'9": varved clay (1/7" to 1/4" layers of silt and 1/4" layers of clay); From 34'9" to 35'12": Silty clay containing some fine sand and few small pebbles; from 35'18" to 36'4": varved clay.
14 (49'-52'8")	50'6"-51'	45.8	50.0	22.6	27.4	0.29	0.08 From 49' to 51'5": varved clay (1/2" to 1-1/2" layers of silt and 1/4" to 3/4" layers of clay); From 51'5" to 52'8": Silty till.
16 (59'-61'9")	59'6"-60"	35.6 13.2					 From 59' to 59'9": varved clay (3/4" to 2" layers of silt and 1/4" to 3/4" layers of clay; from 59'9" to 61'9": Silty till

TABULATION OF RESULTS OF TESTS ON SAMPLES OF SOIL MARKED

C-742 - BOREHOLE NO. 11

Sample No.	Depth of specimen ft.	Water content %	Consistency Limits Liquid Plastic Limit	Plasticity Index	Unconfined Compressive Strength as rec'd.	Corresponding strain	Unit weight received	Remarks
					ton/sq.ft.		lb/cu.ft.	
4 (14'-19')	18'-18'6"	33.0	40.7	19.9	20.8	0.18	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 17'6" 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 19'6" 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 29'6" to 31': subangular gravel-size particles, sand and silt with 4" layer of silt at 30'

BOREHOLE NO. 11

Sample No.	Depth of specimen ft.	Water content %	Consistency Limits		Unconfined Compression		Unit wet weight as received lb/cu.ft.	Remarks
			Liquid Limit	Plastic Index	Strength as received	Corresponding strain		
9	45'6"-46'	14.1	15.9	9.6	6.3	0.29	0.29	142
(44'-47') (Recovery of 2'6")								
								From 44' to 45': subangular grave size particles, sand and silt, 50 clay, with 2" layer of clay; From 45' to 46'6": silty clay with numerous inclusions of silt, few small pebbles.

Figure B-1 Consolidation Tests Results (Boreholes 1 and 2, 1960)

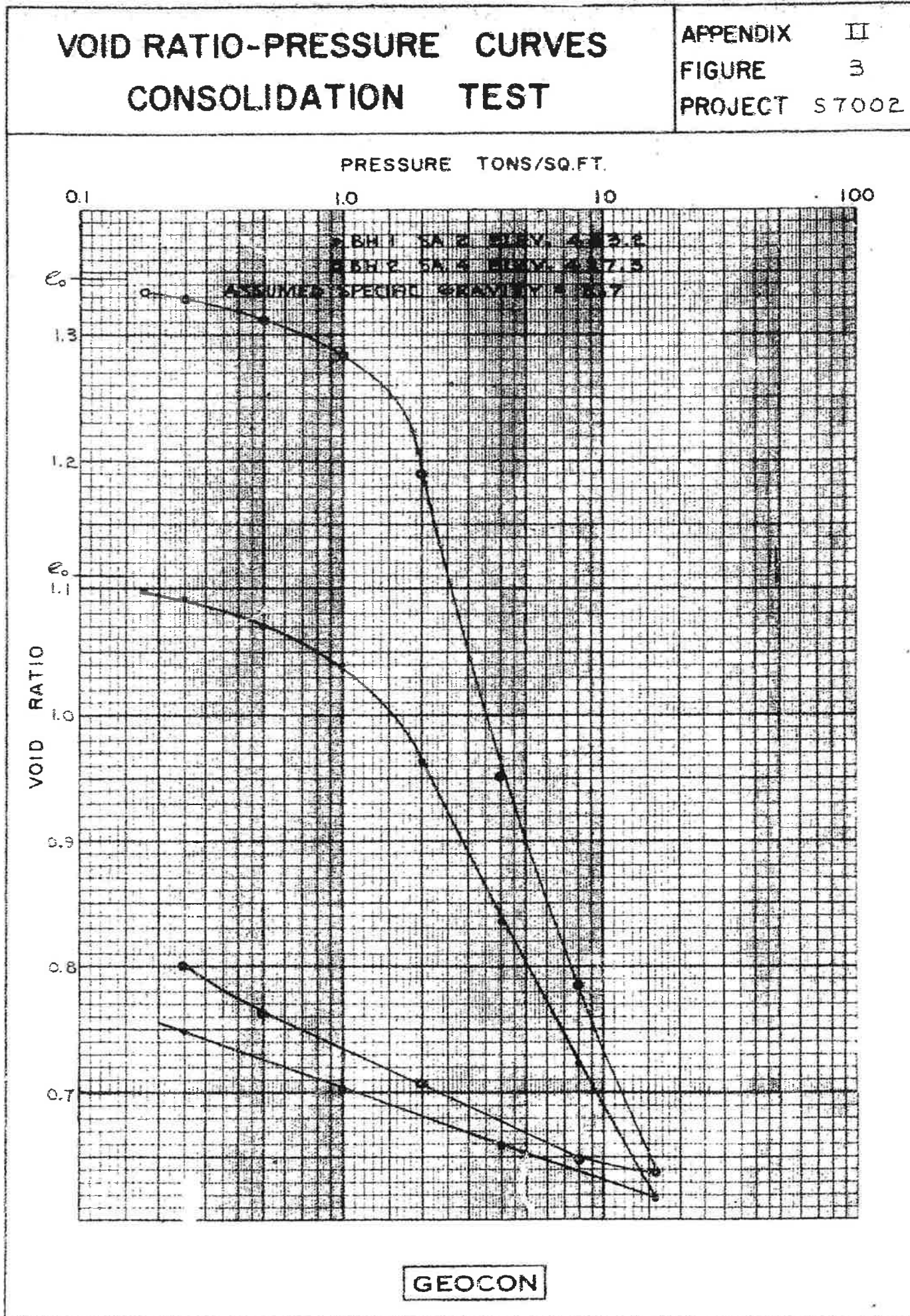
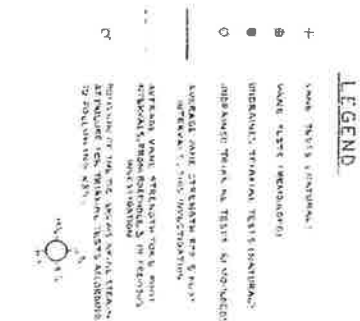
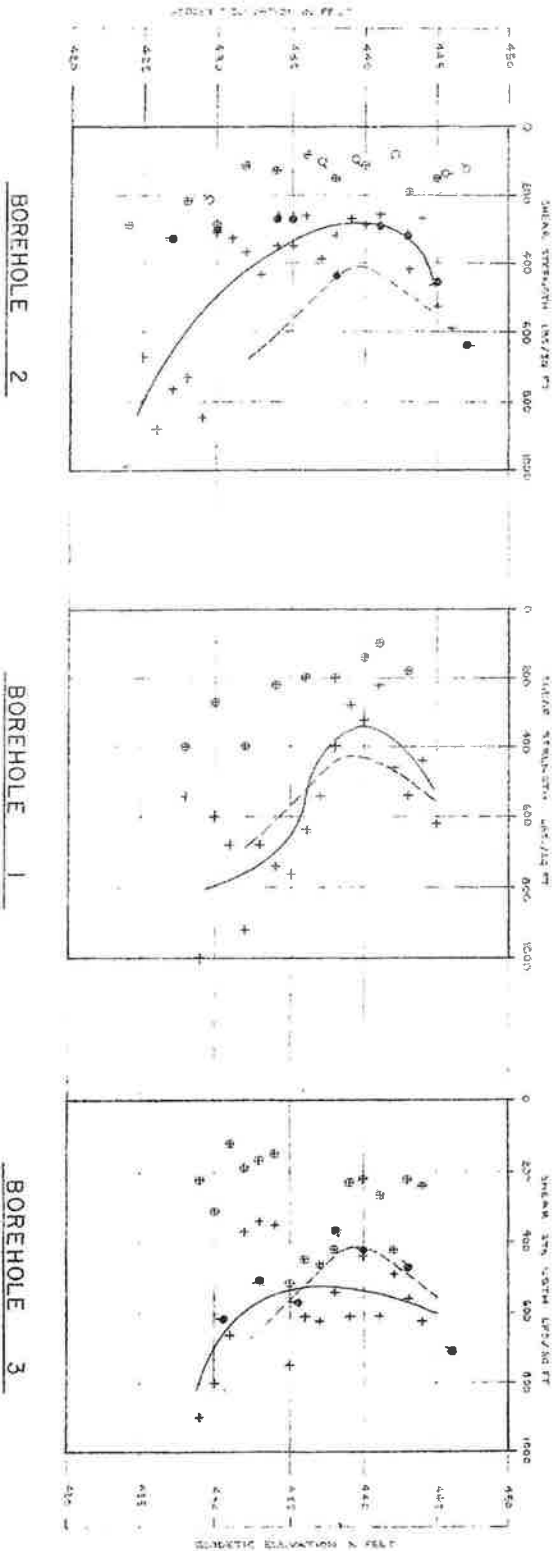


Figure B-2 Shear Strength Plot (Boreholes 1 and 2, 1960)





IN-SITU VANE SHEAR TESTS  
TYPICAL STRESS-STRAIN CURVES  
FOR THE SOFT CLAY

APPENDIX II  
FIGURE 2  
PROJECT C7142

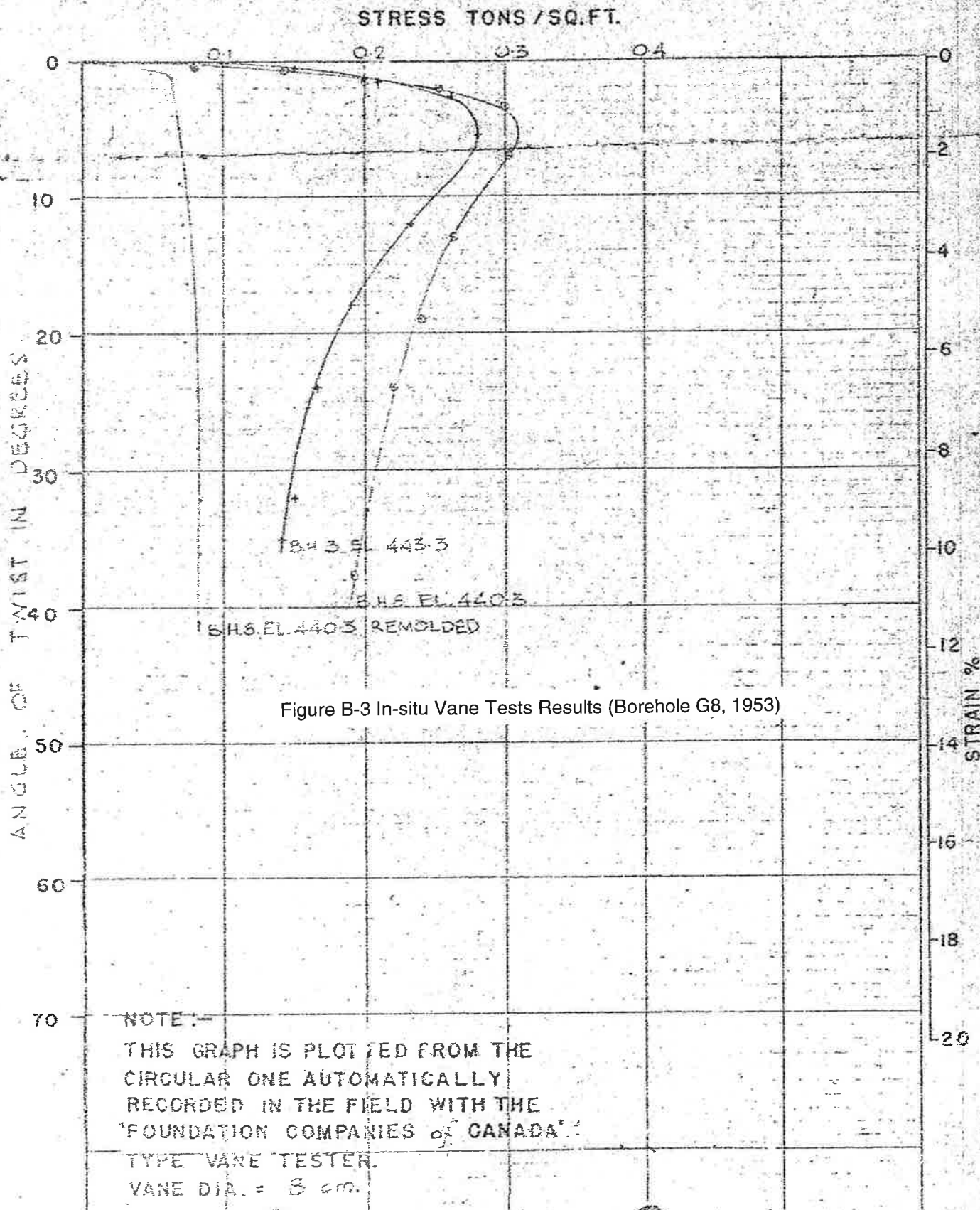


Figure B-3 In-situ Vane Tests Results (Borehole G8, 1953)



# UNCONFINED COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES

APPENDIX II  
FIGURE 3  
PROJECT C7142

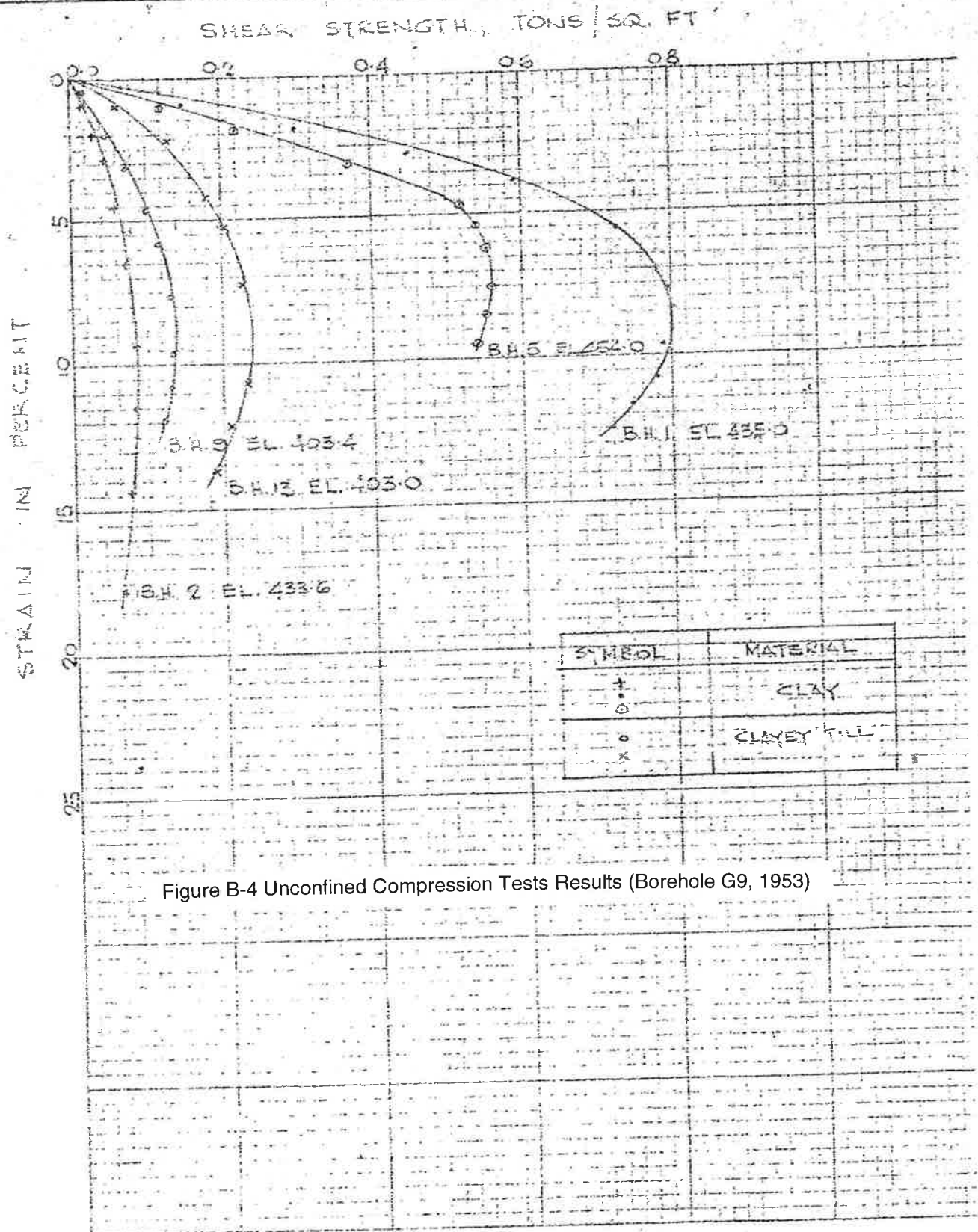


Figure B-4 Unconfined Compression Tests Results (Borehole G9, 1953)

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT

SAND

GRAVEL

Coarse

Fine

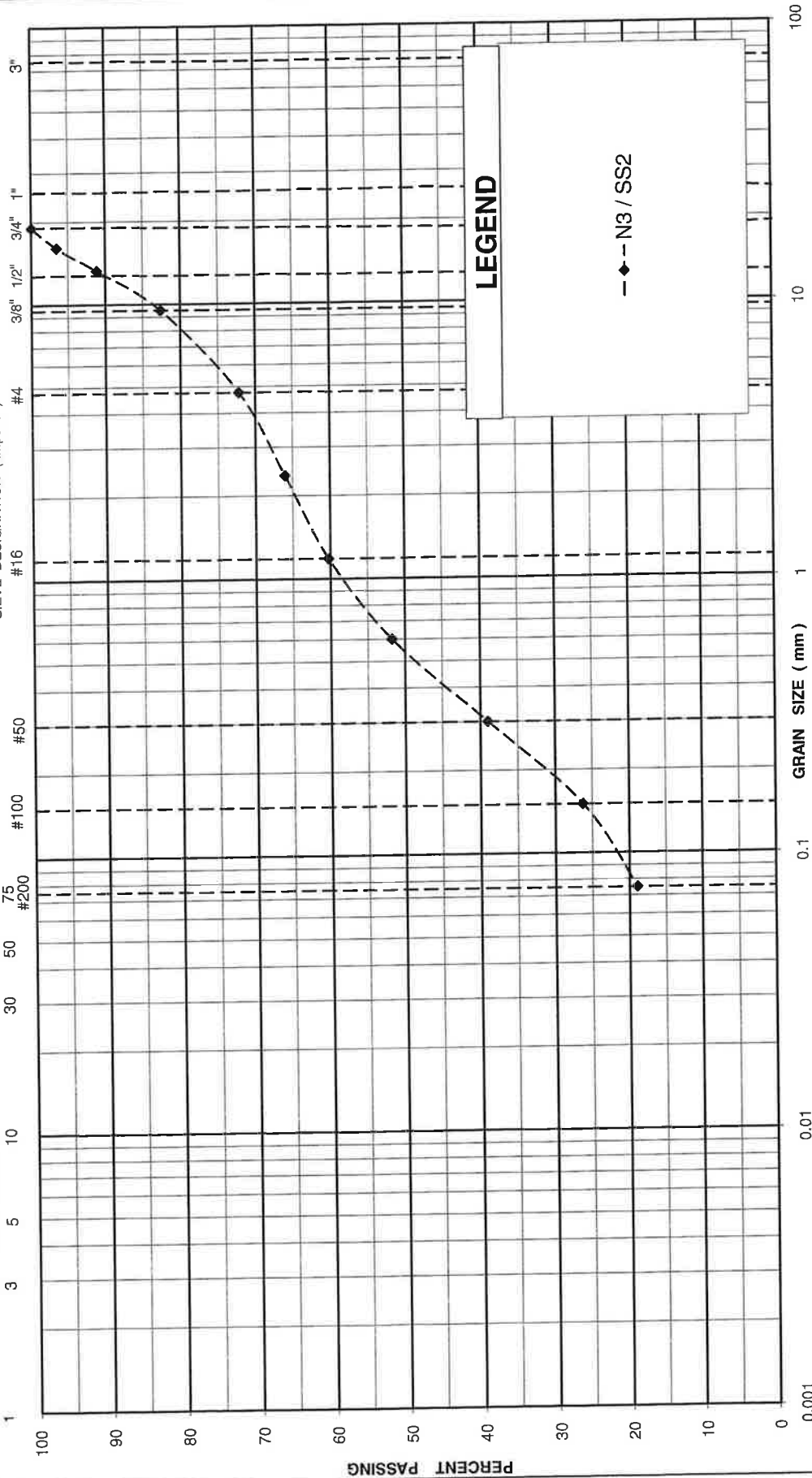
Coarse

Medium

Fine

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



LEGEND

--♦-- N3 / SS2

GRAIN SIZE DISTRIBUTION  
FILL: Gravelly Sand

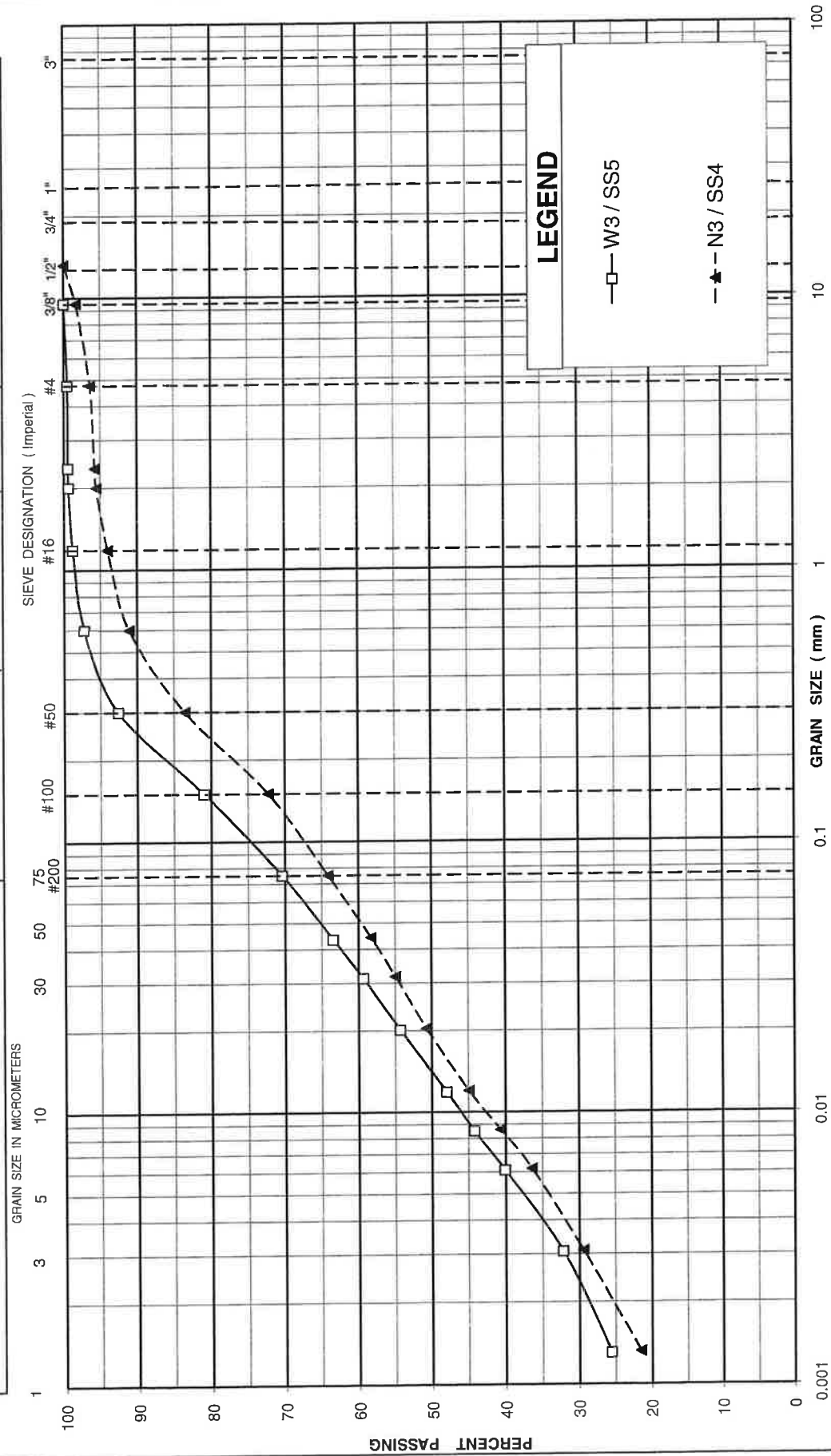
FIGURE NO.: B-5

PROJECT NO: TRANETOB01245AA

DATE: MAR. 2010

# UNIFIED SOIL CLASSIFICATION SYSTEM

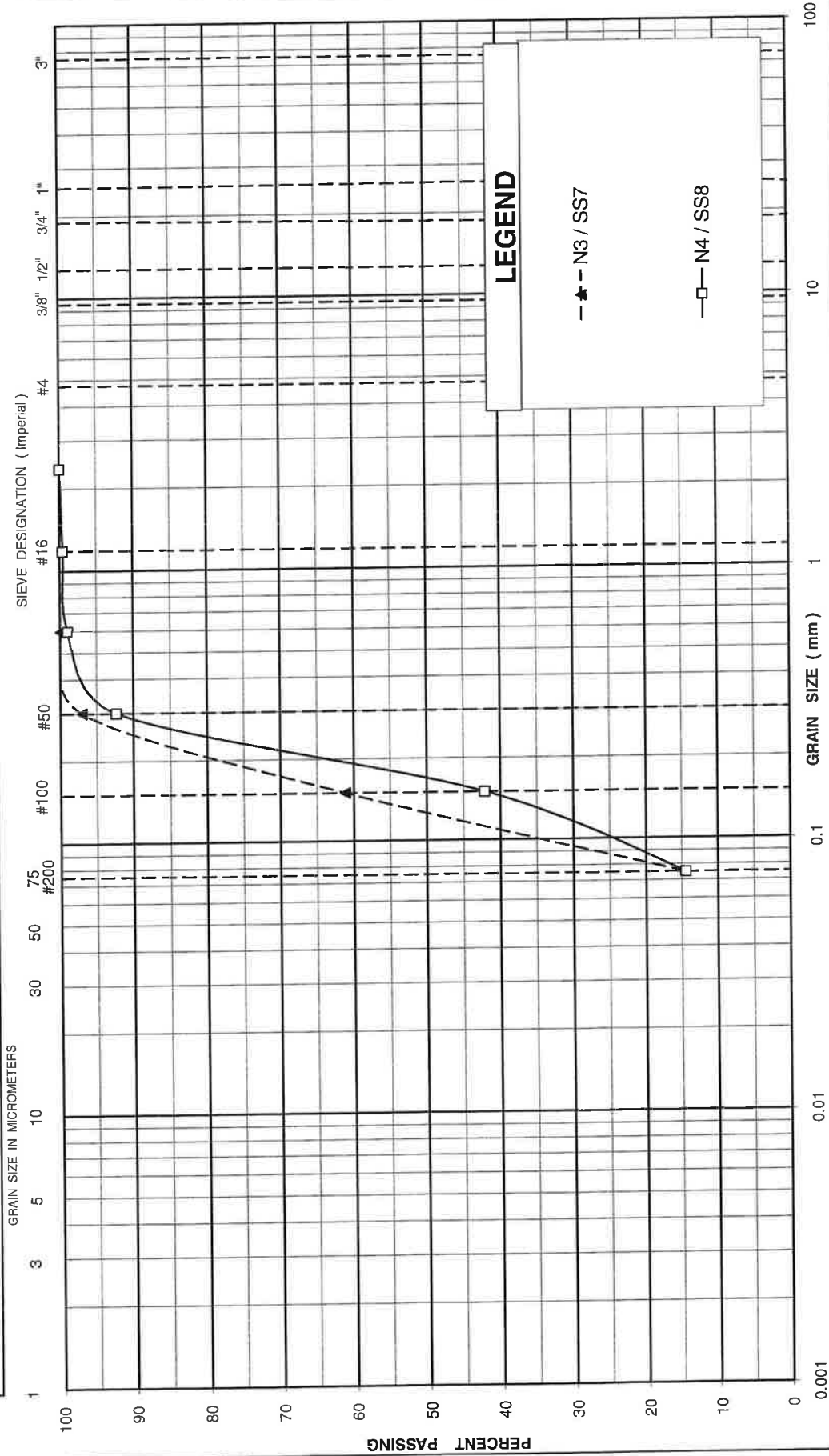
CLAY AND SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	





# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



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## GRAIN SIZE DISTRIBUTION

SAND

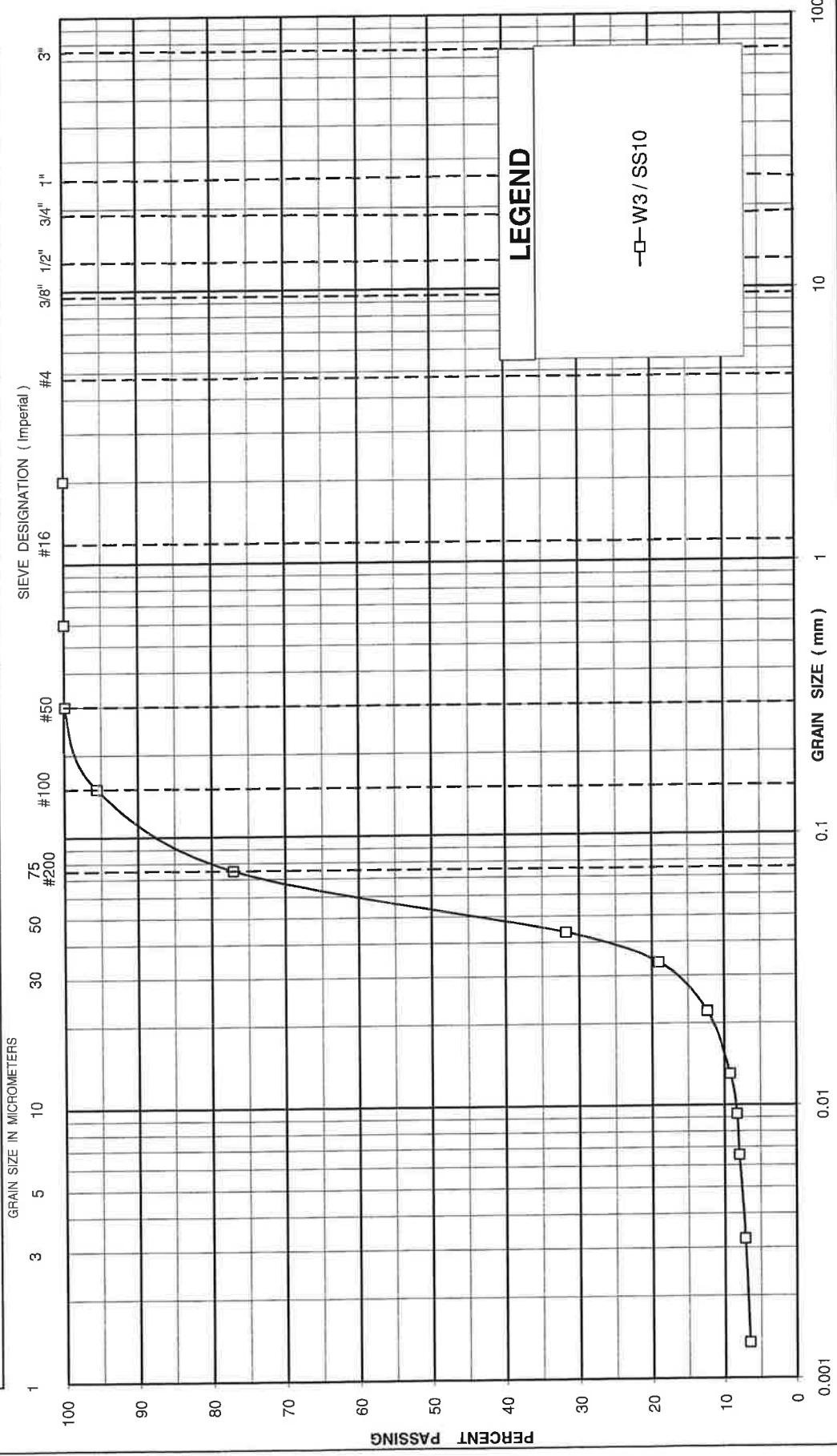
FIGURE NO.: B-8

PROJECT NO: TRANETO801245AA

DATE: MAR. 2010

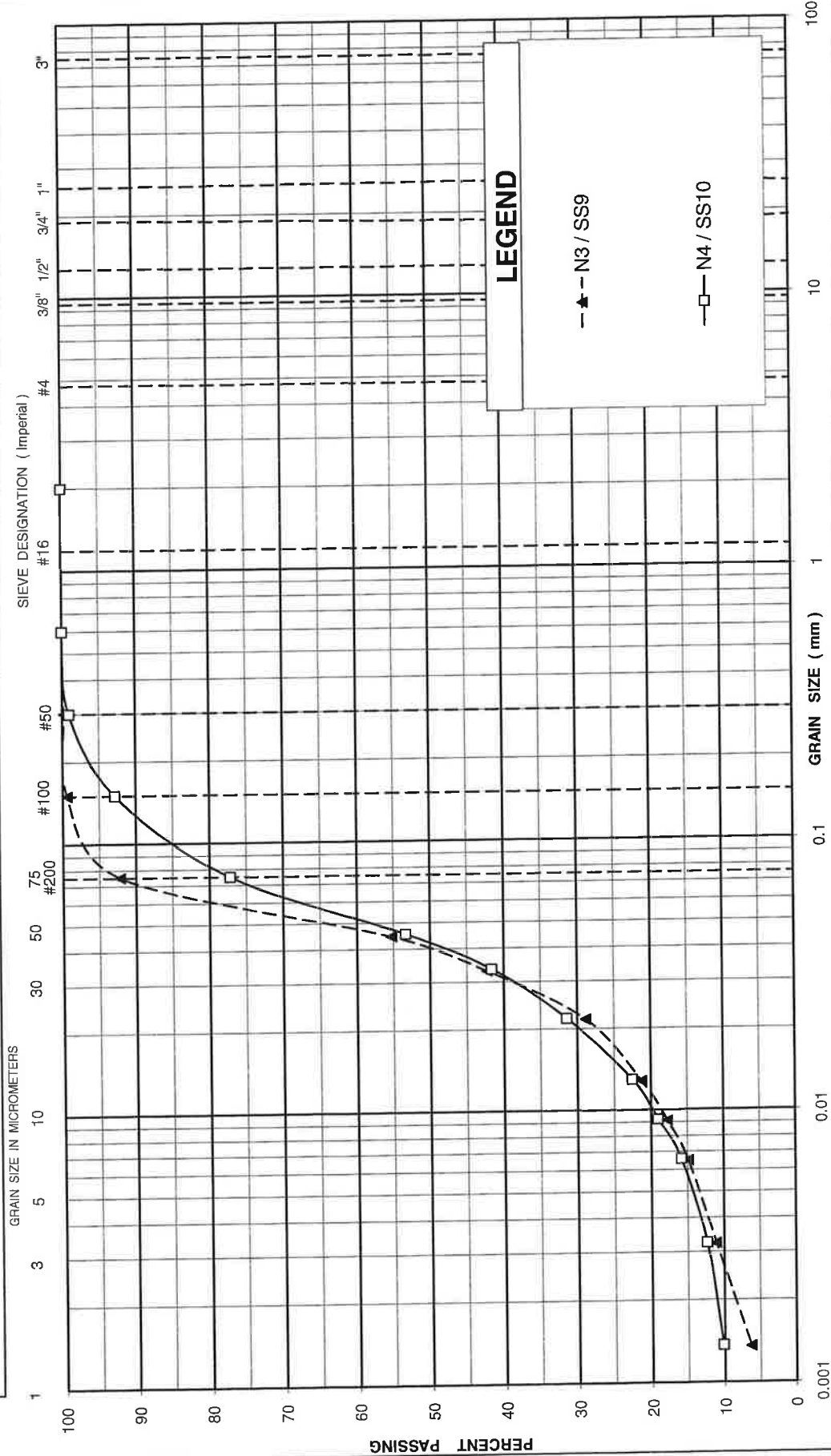
# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



# UNIFIED SOIL CLASSIFICATION SYSTEM

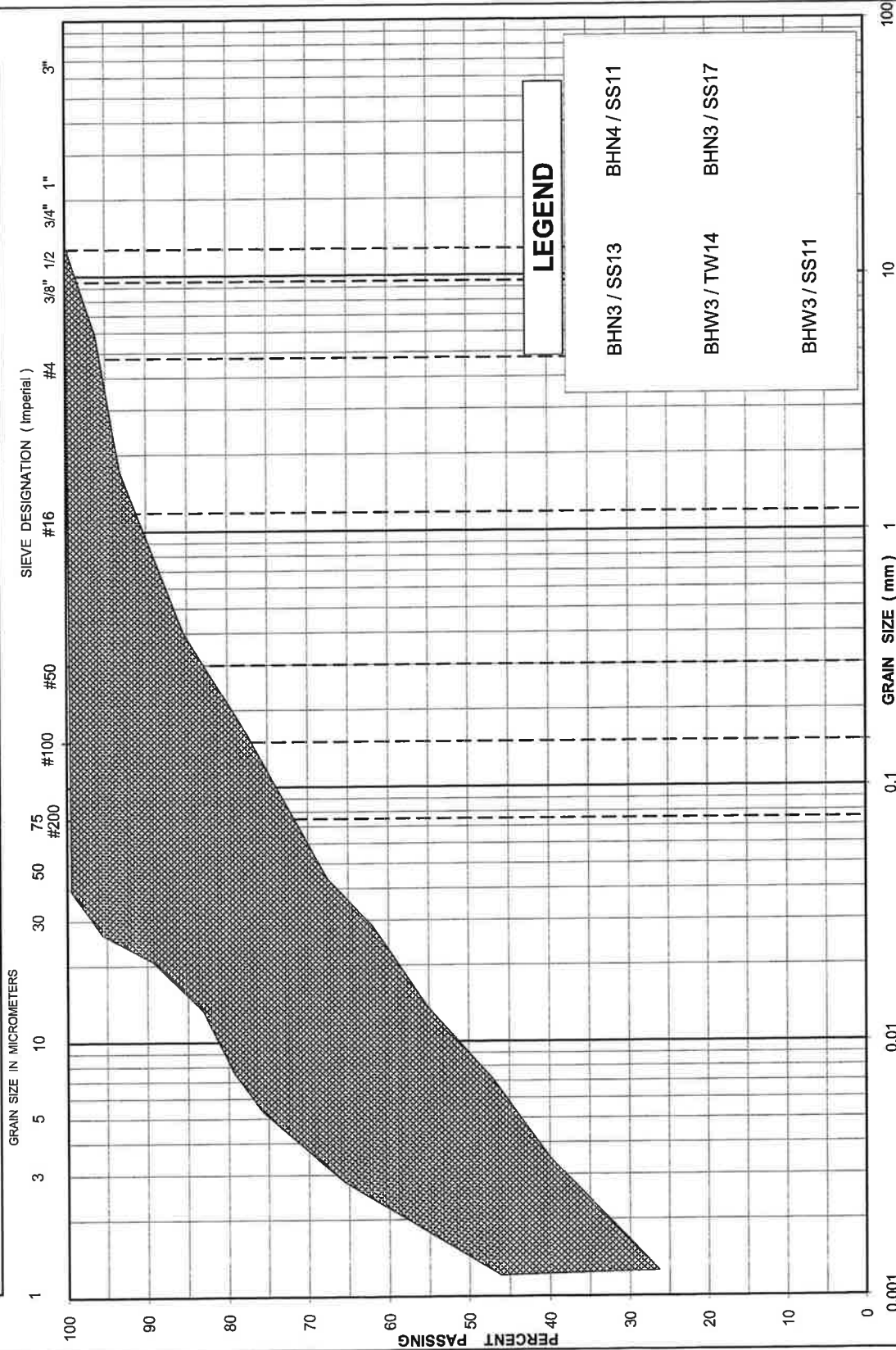
CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	





# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



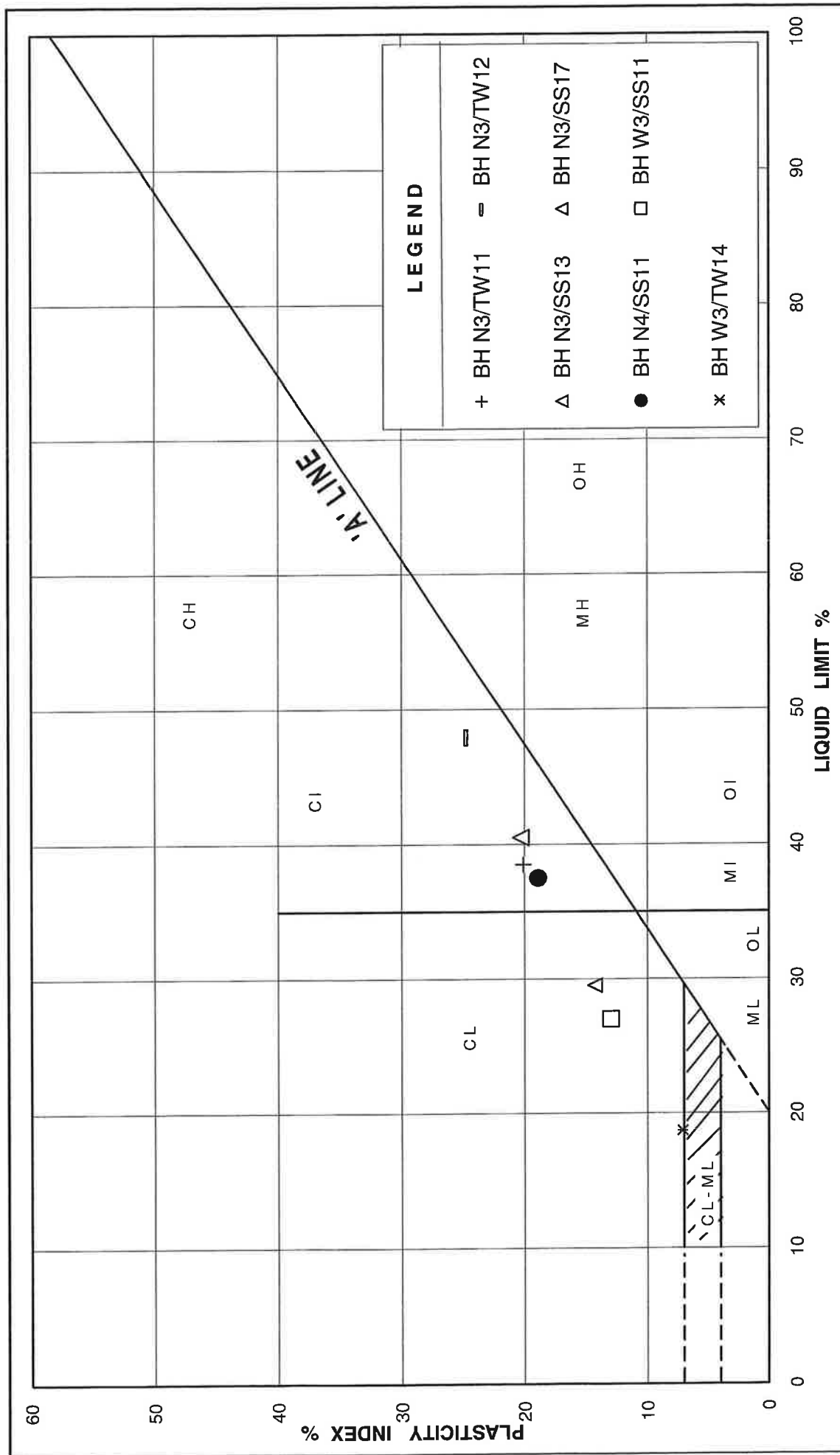


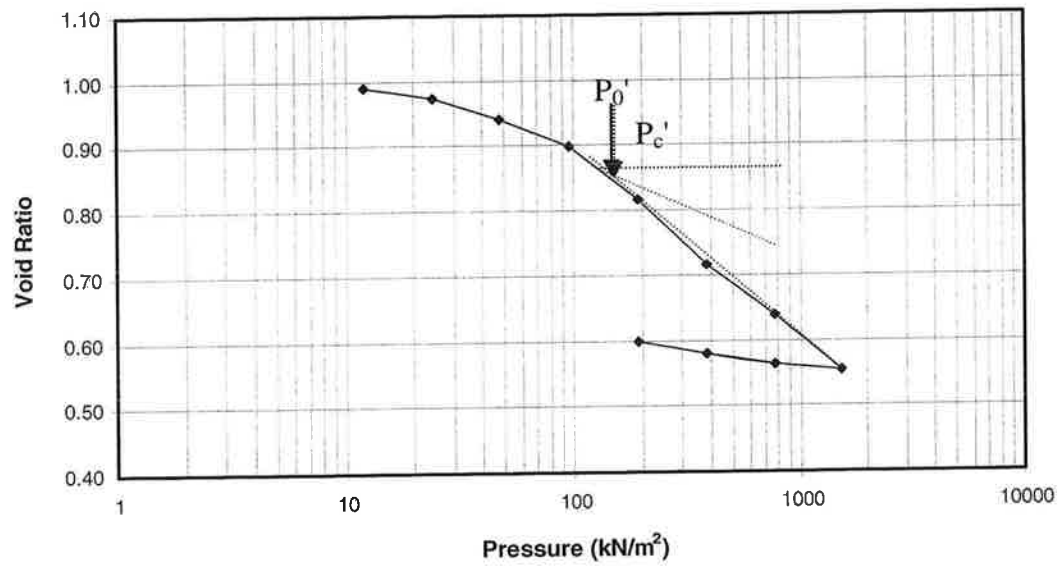
FIGURE No. B-12

REF. No. TRANETOBO1245AA

DATE MARCH, 2010

# Consolidation test- Borehole N3 TW 12

## Void Ratio versus Pressure



## Coefficient of Consolidation vs. Pressure

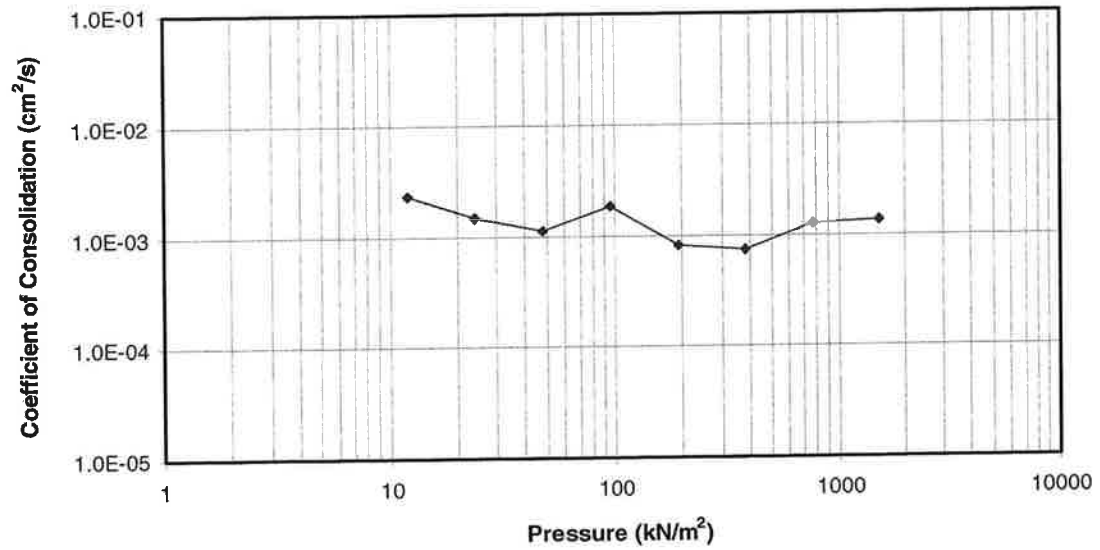


Figure B-13

### Field Vane Test Results

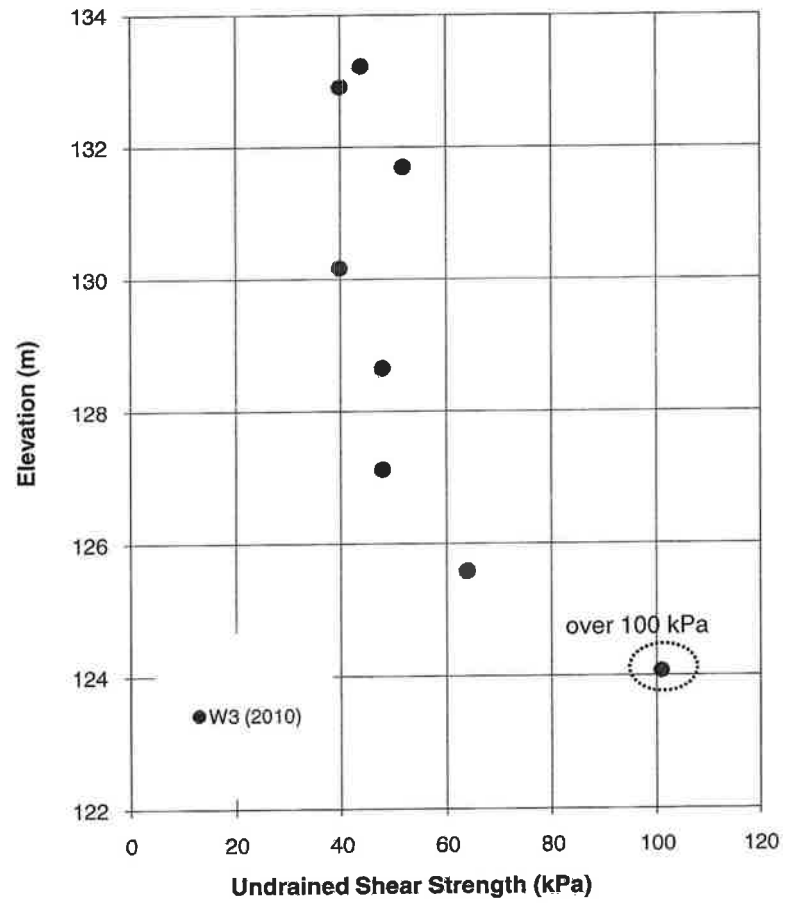
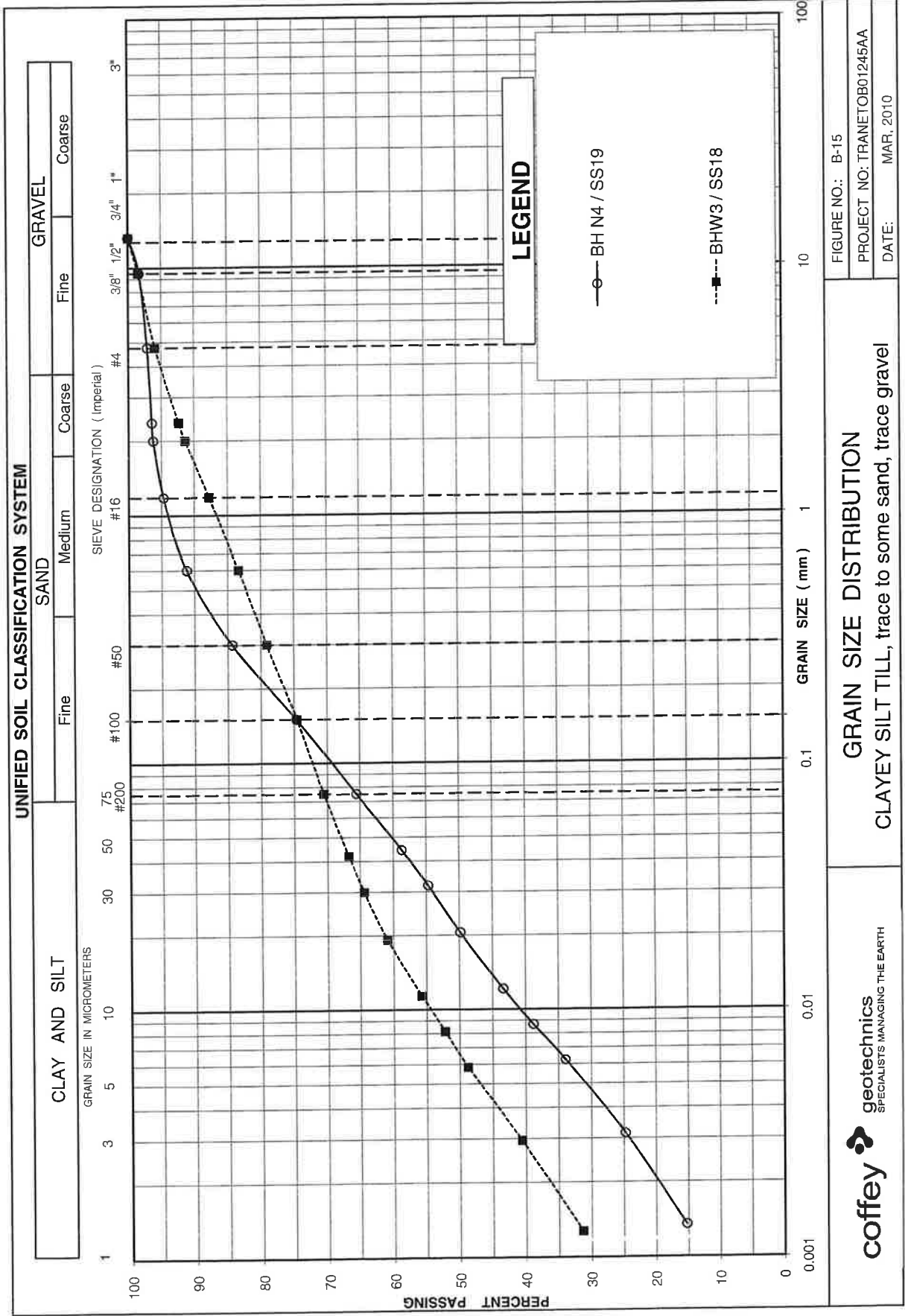
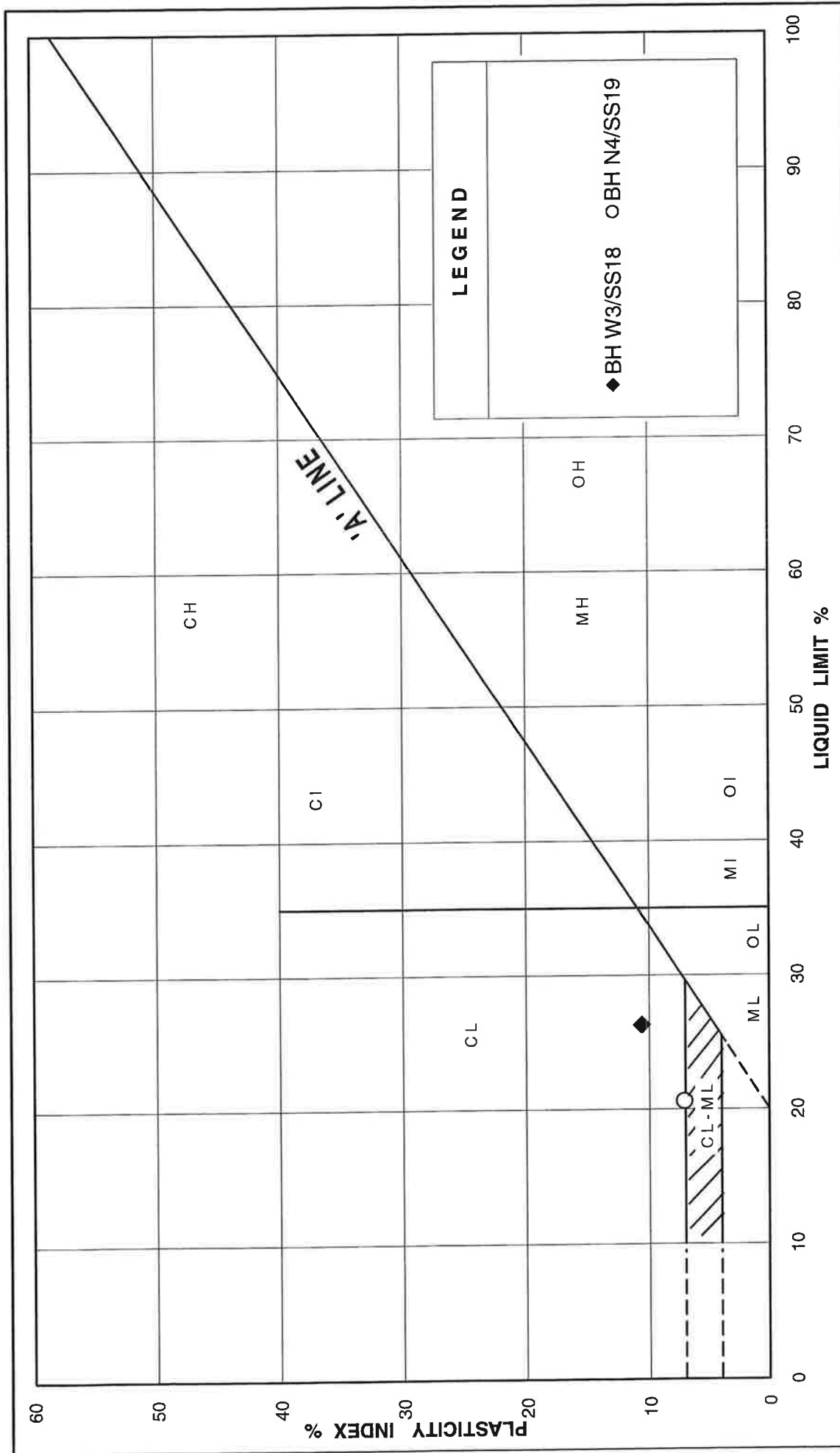



Figure B-14 Undrained Shear Strength Plot





<p><b>coffey</b>  <b>geotechnics</b> SPECIALISTS MANAGING THE EARTH</p>	<p><b>PLASTICITY CHART</b></p> <p><b>CLAYEY SILT TILL</b></p>			FIGURE No. B-16
				REF. No. TRANETOB01245AA
				DATE MARCH, 2010



UNIFIED SOIL CLASSIFICATION SYSTEM				
CLAY AND SILT	SAND			GRAVEL
	Fine	Medium	Coarse	Fine Coarse



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SPECIALISTS MANAGING THE EARTH

# Appendix C

**Stratigraphic Contacts - Highway 401 and C.N.R. Overpass Site**

TABLE 1.

TABLE 1.																	
References	Borehole Designations	Ground Surface EL.	Top of Sand	Top of Cohesive Soil	Top of Till	Water Table	Ground Surface EL.	Top of Sand	Top of Cohesive Soil	Top of Till	Water Table	Depth to Till	Thickness				
													Fill or Clayey Silt, Organics	Sand	Cohesive Soil		
		(ft)	(ft)	(ft)	(ft)	(ft)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(ft)	(ft)	(ft)
	Minimum	453.5	445.0	440.0	393.0	448.8	138.2	137.8	134.1	119.8	136.8	16.8	0.1	3.5	12.3	0.0	11.5
	Maximum	475.5	449.0	449.0	407.5	463.3	144.9	141.9	136.9	124.2	141.2	20.7	4.4	5.4	14.5	14.5	17.6
	Averages	462.0	444.2	444.2	400.6	454.5	140.8	139.9	135.4	122.1	138.5	18.7	1.2	4.6	13.3	7.9	15.0
	1	475.5	448.5	448.5	407.5	456.5	144.9	140.5	136.7	124.2	139.1	20.7	4.4	3.8	12.5	14.5	12.5
	2	461.8	447.3	447.3	406.8	449.8	140.8	140.6	136.3	124.0	137.1	16.8	0.2	4.3	12.3	0.5	14.0
	B4	461.0	452.0	440.0	393.0	449.8	140.5	137.8	134.1	119.8	137.1	20.7	2.7	3.7	14.3	9.0	12.0
	B10	460.0	460.0	442.4	400.0	451.3	140.2	140.2	134.8	121.9	137.6	18.3		5.4	12.9	0.0	17.6
	B11	466.0	465.5	449.0	401.5	459.5	142.0	141.9	136.9	122.4	140.1	19.7	0.2	5.0	14.5	0.5	16.5
	B14	461.0	460.5	446.0	401.7	448.8	140.5	140.4	135.9	122.4	136.8	18.1	0.2	4.4	13.5	0.5	14.5
	B18	461.5	461.5	444.5	403.0	455.0	140.7	140.7	135.5	122.8	138.7	17.8		5.2	12.6	0.0	17.0
	G4	461.1	461.1	445.1	401.1	455.8	140.5	140.5	135.7	122.3	138.9	18.3		4.9	13.4	0.0	16.0
	G8	463.3	457.3	444.3	402.3	463.3	141.2	139.4	135.4	122.6	141.2	18.6	1.8	4.0	12.8	6.0	13.0
	G9	459.3	458	440.8	396.7	453	140.0	139.6	134.4	120.9	138.1	19.1	0.4	5.2	13.4	1.3	17.2
	G11	453.5	452	440.5	397.3	452.5	138.2	137.8	134.3	121.1	137.9	17.1	0.5	3.5	13.2	1.5	11.5
	G12	459.4	459	441.4	396.4	454.2	140.0	139.9	134.5	120.8	138.4	19.2	0.1	5.4	13.7	0.4	17.6

# Appendix D

## Site Photographs



**Ramp N-W (Looking Towards West)**



**Ramp W-N/S (Looking Towards East)**

# 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 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS  $\bar{N}$ .

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

ROCKS ARE DESCRIBED BY THEIR 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:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$c_c$	1	COMPRESSION INDEX
$c_s$	1	SWELLING INDEX
$c_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	$^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	$^\circ$	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_c$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

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



**PRELIMINARY FOUNDATION DESIGN  
REPORT, CN RAIL OVERPASS STRUCTURE  
HIGHWAY 401 REHABILITATION FROM  
LESLIE STREET TO WARDEN AVENUE  
MTO CENTRAL REGION, G.W.P. 2130-01-00  
GEOCRES 30M14-330**

Delcan Corporation  
Project: TRANETOB01245AA-AB  
September 30, 2011

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## Appendices

Appendix F: General Arrangement Drawing

Appendix G: Foundation Elements - Highway 401 and Leslie Street Interchange

Appendix H: Evaluation of Foundation Alternatives

Appendix I: List of OPSSs, OPSDs and NSSPs

Appendix J: Limitations of Report

**PRELIMINARY FOUNDATION DESIGN REPORT  
CN RAIL 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 was proposed and 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 changed and the new proposed structures for the project are as follows:

- C.N.R. overpass structure (single span rigid frame structure)
- Structure(s) over the existing Oriole GO parking
- 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 our foundation investigation (in 2009 and 2010) and 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 is not consistent with normal MTO procedures due to this fact. For example, boreholes at the south and north ends of the proposed structures were found not close enough to the proposed structures. This report provides preliminary foundation recommendations for the proposed C.N.R. overpass structure. Foundation investigation recommendations for detail design are included in Section 5.

Based on the Drawing (See General Arrangement Drawing in Appendix F) provided to us by Delcan, the existing C.N.R. overpass structure, which carries the Highway 401 traffic over the existing CN rail, will be replaced with a single span rigid frame structure in six segments across the width of Highway 401. The total length of the structure will be about 195 m, each segment will be 23.5 m wide and 8 m high (outer cross-sectional dimension with variable lengths) as shown on the Drawing in Appendix F.

It is our understanding that the proposed replacement will be carried out in stages to accommodate the Highway 401 traffic. As well, the details of construction staging are not available at the time of preparing this report.

In general, the sub-surface stratigraphy comprises fill materials and surficial non-cohesive to cohesive (typically non-cohesive) soil deposits overlying silty clay, which is in turn underlain by cohesive and non-cohesive glacial till deposits. The glacial deposits are further underlain in general 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. The original grade at the site was found to be typically between El. 138 and 141 m.

## 4.1 Foundations

We understand that the newly proposed overpass structure will be constructed almost at the same location as the existing overpass structures but the existing rail track will be shifted to the west by about 4.3 m, based on the Drawing provided to us by Delcan, to accommodate future track positioning. . Interference between existing foundations and new foundations is expected. This aspect should be taken into consideration in the design and during the construction. It is our understanding that the proposed rigid frame structure will have similar minimum clearance height 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 ramp structure from the south and the remaining Highway 401 structures (lanes) will be retained for the Highway 401 traffic during the W-N/S Ramp C.N.R. overpass structure construction. After the construction of the southernmost segment, construction will subsequently move to the other segments (towards the north) and the newly built segment(s) and the remaining Highway 401 structures will support the Highway 401 traffic.

Based on the available MTO Geocres information, the existing C.N.R. overpass structure foundations under the Highway 401 are not clearly documented but the most southerly and the most northerly ramp structures over CN Rail (i.e. Ramp W-N-S and Ramp N-W) are supported on pile (12 BP 53 equivalent to HP 310X79) foundations. Battered piles from the existing structure (typically 6V:1H) may interfere with the new piles and vice versa. This aspect should be taken into consideration in the design and during the construction. Details of the existing foundations are summarized in Table G-1 in Appendix G.

Based on the results of our investigation, 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*
- *CFA*

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

The following paragraphs present a discussion on these options. All options discussed below should be further studied and confirmed during the detail design phase.

#### 4.1.1 Spread Footing Foundations

Based on the prevailing subsurface conditions, the use of spread footings is not considered feasible for this project. 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 proposed C.N.R. overpass structure. It is noted that some of the existing Highway 401 structures in the vicinity of the proposed C.N.R. overpass structure are supported on drilled caisson foundations. Since the site is located in the Greater Toronto Area (i.e. close to residential areas and hospitals), drilled caisson is typically considered to be a favourable deep foundation option because of less noise and vibration generated during the construction in comparison with driven piles. The existing structure need 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 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. For this project however we recommend that the total penetration be limited to no more than 2.0 m.

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	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)	2000	3000	75	120	1730(SLS) 2650 (ULS)
1.07 m (42-inch)	2000	3000	75	120	2300 (SLS) 3500 (ULS)
1.22 m (48-inch)	2000	3000	75	120	2900 (SLS) 4400 (ULS)

\*assume a penetration of 2.0 m into the very dense / hard till ( $N > 50$  blows/0.3 m)

These caisson sizes are recommended for efficiency in installation in consideration of the prevailing subsurface conditions. As well as was mentioned before it is assumed that the caissons will be socketed at least 2.0 m into the very dense / hard till (i.e.  $N > 50$  blows/0.3 m). Higher caisson resistances would be

available for greater penetration into the competent till but this is not recommended with due consideration of the excess hydrostatic pressures which prevail at the site.

A sample calculation in determining the caisson resistances is as follows. A 1.07m (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 distance between any two adjacent caissons is not less than 2.5 diameters (centre to centre).

Table 4.1.2.2 presents the anticipated caisson depths/elevations at Boreholes W3, N3 and N4.

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
W3	144.8	24.3	120.5	V. dense Sandy Silt to Silty Sand Till
N3	142.4	23.9	118.5	V. dense Silty Sand Till
N4	144.2	23.0	121.2	Hard Clayey Silt Till

The minimum caisson diameter would be 0.76 m to enable the cleaning and inspection of the base of the caisson. For this project, if this size caisson is to be used we will be pleased to present recommended resistances.

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 equals to 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.

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 and anticipated cobbles and boulders in the transition zone of silty clay to glacial till deposit as well as within the glacial till deposits. This can be discussed with a specialist contractor in relation to cost vs. caisson diameter. Dewatering will 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 cobbles and boulders 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, above mentioned geotechnical resistances need to be revised. To prevent the disturbance of the base of the caisson, the concrete must be poured without delay after cleaning 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 also be 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.

Removal of the existing structure may be required for installation of 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 risks in caisson installation, including the definition of the interface between the till deposits and the underlying water bearing granular deposits.

#### **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 C.N.R. 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 bridge 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. 120 to 117 m) can be designed for MTO's standard values of 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, by increasing steel area. Normally, somewhat higher resistances are available for the pile sizes recommended. However, in view of the possible upward gradients, artesian pressures and known past problems experienced at this interchange, the use of higher capacities is not recommended. The anticipated pile tip (refusal) elevations at Boreholes W3 and N3 (i.e. refusal in the granular till) and the basically cohesive till in Borehole N4 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)
W3	144.8	24.1	120.7
N3	142.4	24.0	118.4
N4	144.2	22.7	121.5

According to the GA Drawing provided to us by Delcan, the top of pile elevations are planned at about 2 m below the existing grade (existing grade is assumed at El. 140 m) and therefore the anticipated pile lengths would typically be between 16 m and 20 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.

The driving of the piles will need to be conducted in accordance with OPSS903. If the piles encounter refusal before sufficiently penetrating into the competent materials, then pile capacities may need to be revisited and alternative measures sought. 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 however that 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 and/or excess upward hydrostatic pressures 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 we found batter steeper than 4V:1H is difficult to install in practice. As well, this is particular site is rather congested and battered piles present a greater chance of causing interference with existing piles. For these reasons, we recommend that the batter be no steeper than about 4V:1H.

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

Removal of the existing structure may be required prior to driving piles where they interfere with the existing foundations. However, for this aspect, the use of H-Pile may have slight advantage compare to caisson foundations due to a smaller area (cross-section) of H-Pile (less change to hit the existing foundation) and flexibility of pile layout within the proposed pile cap. This aspect needs to be carefully investigated at the detail design phase.

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 vibration 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 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 rigid frame structure.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in most soil and rock types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing 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 observed subsurface conditions at the site, geotechnical resistance of micropile primarily depends on the bond length within the competent glacial till deposit and the type of micropile/installation method. For a preliminary design purposes, axial resistances of up to about 500 kN/micropile are available at ULS (for 260 mm diameter micropile) with a penetration not less than 5 m into the competent glacial till. 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 may 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 (removal of the existing structure may only be mandatory for micropiles at a later stage where they interfere with the existing foundations)

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

#### **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 continuous flight augers. 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 to 36 inches), but to our knowledge locally available diameters are 0.5 m to 0.6 m (20 to 24 inches) and installed 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 steel reinforcement is often limited 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 continuous augers 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 reduced. 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 in low headroom conditions or in confined spaces 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 further pursue this option.

#### 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). 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**

Considering the environmental implications of the driven pile option (e.g. vibration and noise), the caisson option is the preferred alternative at this preliminary design stage. However, this should be confirmed by the TPM Consultant once all the other aspects such as cost, staging and other 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 =  $22 \text{ kN/m}^3$

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 =  $21 \text{ kN/m}^3$

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. 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 4.4.1: Recommended Unfactored Parameters for Temporary Shoring Design**

Soil Type	$K_a$	$K_o$	$K_p$	$\gamma$ (kN/m <sup>3</sup> )
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
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 SCOPE OF WORK REQUIRED FOR DETAILED DESIGN

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.

The 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 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. The choice of the new foundation type including battering, if any, will be influenced by the type, positioning and other details (e.g. batter angles) of the existing foundations and this aspect should be studied in more detail in the detail design stage.

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

## **General Arrangement Drawing**

# METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN

DIST. No.  
CONT No 2010-  
WP No 630-

HIGHWAY 401  
CNR OVERHEAD  
GENERAL ARRANGEMENT

**Delcan**



SHEET

## GENERAL NOTES :

### CLASS OF CONCRETE

ALL CONCRETE 30 MPa

### CLEAR COVER TO REINFORCING STEEL

FOOTINGS 100 ± 25

DECK TOP 70 ± 20  
BOTTOM 40 ± 10

REMAINDER, UNLESS OTHERWISE NOTED. 70 ± 20

### REINFORCING STEEL:

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. STAINLESS REINFORCING STEEL BARS WITH THE PREFIX 'S' SHALL CONFORM TO TYPE 316LN OR DUPLEX 2205 WITH A MINIMUM YIELD STRENGTH OF 420 MPa. BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS 'B'.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1 AND SS12-2, UNLESS OTHERWISE INDICATED.

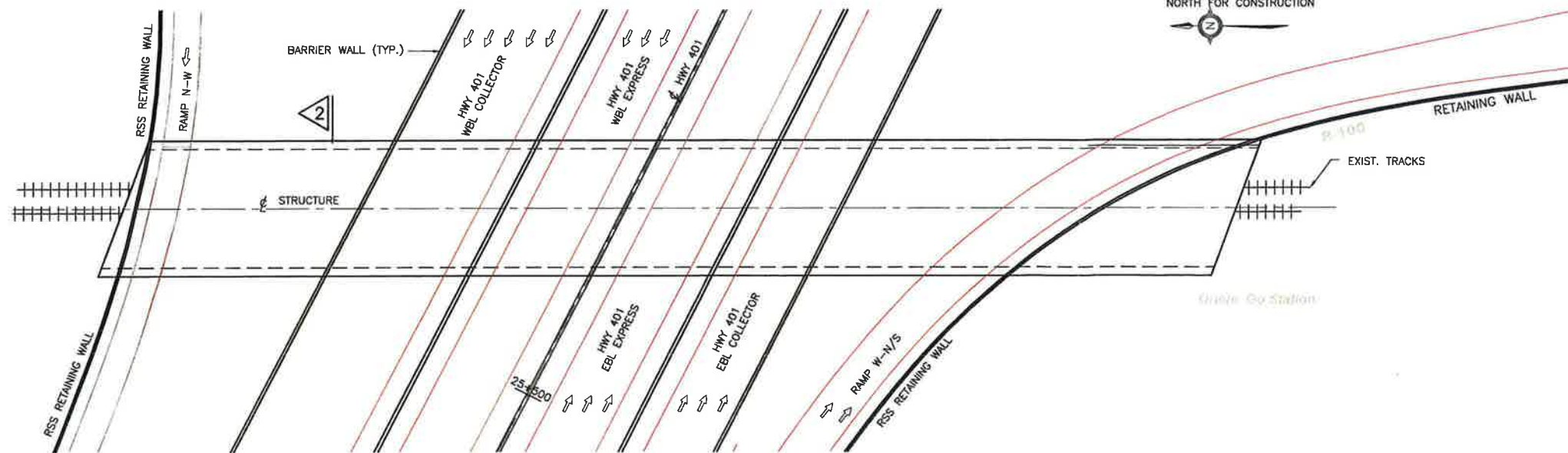
### CONSTRUCTION NOTES

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

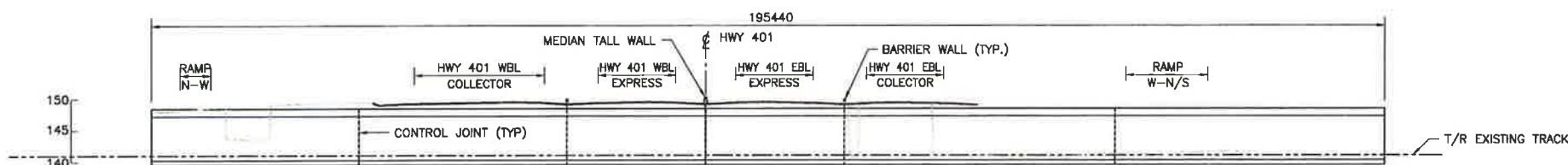
BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS, KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500mm.

### LIST OF ABBREVIATIONS:

T/P - DENOTES TOP OF PAVEMENT.  
T/R - DENOTES TOP OF RAIL.  
T/F - DENOTES TOP OF FOOTING.  
E/P - DENOTES EDGE OF PAVEMENT.  
WP - DENOTES WORKING POINT.



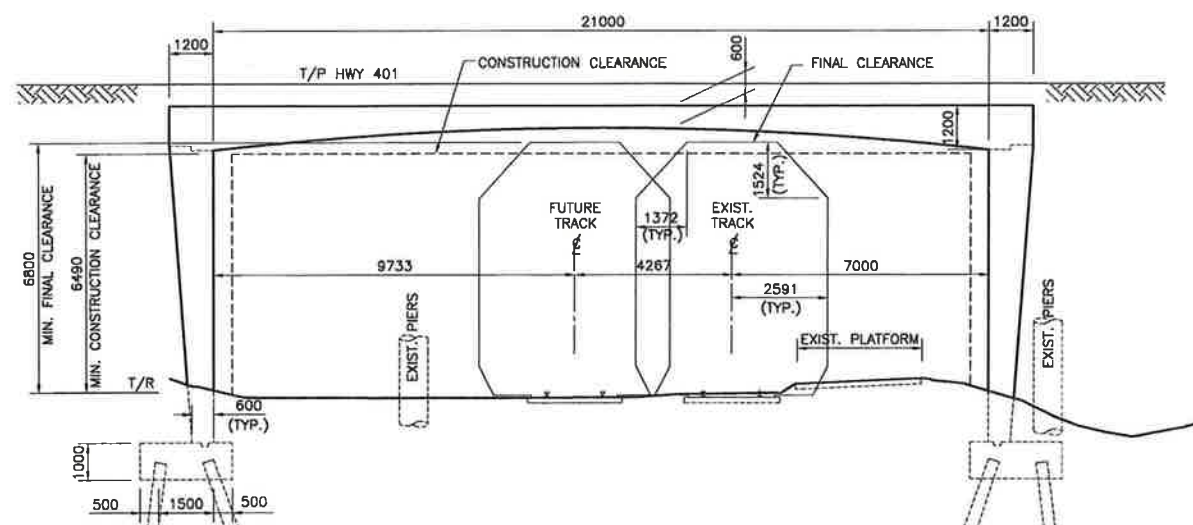
PLAN  
1:500



1 ELEVATION  
1:500

EAST

WEST



1  
1:100

REVISIONS	DESCRIPTION
DESIGN N.W.	CHK F.O. CODE CHBDC-06 LOADCL-625-0MT DATE SEPT 2010
DRAWN A.K.L.	CHK N.W. SITE STRUCT SCHEME DWG 1



# Appendix G

**Foundation Elements - Highway 401 and Leslie Street Interchange**

Table G-1  
Project Number: Foundation Elements - Highway 401 and Leslie Street Interchange  
TRANETO801245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-bults	Depth	Diameter	Base Diameter	Battered	Legend and Notes Signifies extent load 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.	Notes
				(ft)	(in)	(in)	(m)	(mm)	(mm)			
37-206/1 Hwy 401 over pass at Leslie EBL Collectors (16 spans)	418	12BP53 H-Pile	35.0	*			45.7			1 to 3	working load 250 Ton/Caisson	
	418	12BP53 H-Pile	68	*			26.8				design load 60 Ton/pile	
	419	Concrete Caisson	n/a	n/a	35		n/a			1 to 6	working load 250 Ton/Caisson	
	420	12BP53 H-Pile	n/a	n/a			n/a			1 to 6	design load 60 Ton/pile	
	421	12BP53 H-Pile	n/a	n/a	35		n/a				working load 250 Ton/Caisson	
	422	Concrete Caisson	n/a	n/a	35						working load 250 Ton/Caisson	
	423	Concrete Caisson	n/a	n/a	35						working load 250 Ton/Caisson	
	424	Concrete Caisson#1	81		30	42	24.7	762	1067		working load 250 Ton/Caisson	
	424	Concrete Caisson#2	78		30	42	23.8	762	1067		working load 250 Ton/Caisson	
	424-3	12BP53 H-Pile			30						Caisson blow-out/Replaced by H-Pile, 60 Ton/Pile	
	425	Concrete Caisson#1	82		30	42	25.0	762	1067		working load 250 Ton/Caisson	
	425	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	425	Concrete Caisson#3	78		30	42	23.8	762	1067		working load 250 Ton/Caisson	
	426	Concrete Caisson#1	81		30	42	24.7	762	1067		working load 250 Ton/Caisson	
	426	Concrete Caisson#2	78		30	42	23.8	762	1067		working load 250 Ton/Caisson	
426-3	427	12BP53 H-Pile			30						Caisson blow-out/Replaced by H-Pile, 60 Ton/Pile	
	427	Concrete Caisson#1	77		30	42	23.5	762	1067		working load 250 Ton/Caisson	
	427	Concrete Caisson#2	77		30	42	23.5	762	1067		working load 250 Ton/Caisson	
	427	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	428	Concrete Caisson#1	79		30	42	24.1	762	1067		working load 250 Ton/Caisson	
	428	Concrete Caisson#2	78		30	42	23.8	762	1067		working load 250 Ton/Caisson	
	428	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	429	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	429	Concrete Caisson#2	77		30	42	23.5	762	1067		working load 250 Ton/Caisson	
	429	Concrete Caisson#3	70		30	42	21.5	762	1067		working load 250 Ton/Caisson	
	430	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	430	Concrete Caisson#2	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	430	Concrete Caisson#3	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	430	Concrete Caisson#4	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	431	12BP53 H-Pile	65	*			19.8			1 to 6		
432	432	12BP53 H-Pile	65	*			19.8			1 to 6		
	433	12BP53 H-Pile	70	*			21.3			1 to 6		
	434	12BP53 H-Pile	80	*			24.4			1 to 3		

Table G-1  
Project Number:  
Foundation Elements - Highway 401 and Leslie Street Interchange  
TRANETO801245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated (ft)	Diameter (in)	Base Diameter ** Estimated based on table no as-built	Depth (m)	Diameter (mm)	Base Diameter (mm)	Battered	Legend and Notes Signifies event took 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.	Notes
Hwy 401 overpass at Leslie St. WHL Connectors [33 spans]	101	12BP53 H-Pile	54	*						1 to 3		
	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	53.5	*	30	36	15.0	762	914		working load 250 Ton/caisson	
	106	Concrete Caisson	63	*	30	36	15.2	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	24.5	762	914		working load 250 Ton/caisson	
	112	Concrete Caisson	80.5	*	30	36	21.5	762	914		working load 250 Ton/caisson	embankment failure north of the bent/working load 200 Ton/caisson
	113	Concrete Caisson	70.5	*	30	36	21.5	762	914		working load 250 Ton/caisson	embankment failure north of the bent/working load 200 Ton/caisson
	114	Concrete Caisson	72	*	30	36	21.9	762	914		working load 250 Ton/caisson	embankment failure north of the bent/working load 200 Ton/caisson
	115	Concrete Caisson	72	*	30	36	21.9	762	914		working load 250 Ton/caisson	embankment failure north of the bent/working load 200 Ton/caisson
	116	Concrete Caisson	74	*	30	36	22.6	762	914		working load 250 Ton/caisson	embankment failure north of the bent/working load 200 Ton/caisson
	117	Concrete Caisson	80	*	30	36	24.4	762	914		working load 250 Ton/caisson	embankment failure north of the bent/working load 200 Ton/caisson
	118	Concrete Caisson									design load 60 Ton/pile	
	119	12BP53 H-Pile	n/a	n/a							working load 250 Ton/caisson	
	120	12BP53 H-Pile	n/a	n/a							working load 250 Ton/caisson	
	121	Concrete Caisson	n/a	n/a	30	36	23.5	762	914		settling problem, caisson replaced with 6 new tube piles of 220 mm diameter, also replaced the Pier.	
	122	Concrete Caisson	77	*	30	36	23.2	762	914		working load 250 Ton/caisson	
	123	Concrete Caisson	76	*	30	36	23.5	762	914		working load 250 Ton/caisson	
	124	Concrete Caisson	77	*	30	36	23.8	762	914		working load 250 Ton/caisson	
	125	Concrete Caisson	78	*	30	36	23.5	762	914		working load 250 Ton/caisson	
	126	Concrete Caisson	77	*	30	36	23.5	762	914		working load 250 Ton/caisson	settling problem, caisson replaced with 6 new tube piles of 220 mm diameter, also replaced the Pier
	127	Concrete Caisson	77	*	30	36	23.5	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	78	*	30	36	23.8	762	914		working load 250 Ton/caisson	
	131	12BP53 H-Pile	64	*			19.5			1 to 6		
	132	12BP53 H-Pile	64	*			19.5			1 to 6		
	133	12BP53 H-Pile	64	*			19.5			1 to 6		
	134	12BP53 H-Pile	78	*			23.8			1 to 3		

Table G-1  
Project Number: Foundation Elements - Highway 401 and Leslie Street Interchange  
TRANETO801245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no es-bults	Depth	Diameter	Base Diameter	Battered	Legend and Notes Signifies event took 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.	Notes
			(ft)	(ft)	(in)	(in)	(m)	(mm)	(mm)			
37-206/3	318	12BPS3 H-Pile										
Hwy 401 overpass at Leslie St.	319	Not Clear										
EBL Express	320	Not Clear										
(16 Spans)	321	Not Clear										
	322	Not Clear										
	323	Not Clear										
	324	12BPS3 H-Pile	78	*	30	42	23.8	762	1067	not clear		
	325	Concrete Caisson#1	85		30	42	25.9	762	1067		Previously recognized at caisson, H pile design load 60 Ton	
	326	Concrete Caisson#2	98		30	42	25.9	762	1067		working load 250 Ton/caisson	
	327	Concrete Caisson#3	98		30	42	25.9	762	1067		working load 250 Ton/caisson	
	328	Concrete Caisson#1	82		30	42	25.0	762	1067		working load 250 Ton/caisson	
	329	Concrete Caisson#2	83		30	42	24.7	762	1067		working load 250 Ton/caisson	
	330	Concrete Caisson#3	83		30	42	24.7	762	1067		working load 250 Ton/caisson	
	331	Concrete Caisson#1	82		30	42	25.0	762	1067		working load 250 Ton/caisson	
	332	Concrete Caisson#2	92		30	42	25.0	762	1067		working load 250 Ton/caisson	
	333	Concrete Caisson#3	92		30	42	25.0	762	1067		working load 250 Ton/caisson	
	334	Concrete Caisson#1	79		30	42	24.0	762	1067		working load 250 Ton/caisson	
	335	Concrete Caisson#2	91		30	42	27.7	762	1067		working load 250 Ton/caisson	
	336	Concrete Caisson#3	91		30	42	27.7	762	1067		working load 250 Ton/caisson	
	337	Concrete Caisson#1	77		30	42	25.4	762	1067		working load 250 Ton/caisson	
	338	Concrete Caisson#2	89		30	42	27.1	762	1067		working load 250 Ton/caisson	
	339	Concrete Caisson#3	89		30	42	27.1	762	1067		working load 250 Ton/caisson	
	340	Concrete Caisson#1	80		30	42	24.4	762	1067		working load 250 Ton/caisson	
	341	Concrete Caisson#2	81		30	42	27.7	762	1067		working load 250 Ton/caisson	
	342	Concrete Caisson#3	81		30	42	27.7	762	1067		working load 250 Ton/caisson	
	343	Concrete Caisson#1	64	*			19.4			1 to 6		
	344	Concrete Caisson#2	64	*			19.5			1 to 6		
	345	Concrete Caisson#3	64	*			19.5			1 to 6		
	346	Concrete Caisson#1	76	*			23.8			1 to 3		

Table G-1  
Project Number: Foundation Elements - Highway 401 and Leslie Street Interchange  
TRANET0801245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated no as-builts	Diameter (in)	Base Diameter ** Estimated no as-builts	Depth (m)	Diameter (mm)	Base Diameter (mm)	Battered	Legend and Notes Signifies exact bore place Non Clear Indicator 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. Encasement failure north of the bent, pile tip at surface sand deposit	Notes
37-209/4	218	Not clear	intended to surface sand									
Hwy 401 over 209s at Leslie St.	219	12BP53 H-Pile	intended to surface sand									
WIL Easement	220	12BP53 H-Pile	intended to surface sand									
(1.5 Spans)	221	12BP53 H-Pile	intended to surface sand									
	222	12BP53 H-Pile	intended to surface sand									
	223	12BP53 H-Pile	intended to surface sand									
	224	12BP53 H-Pile	intended to surface sand									
	225	Concrete Caisson#1	84		30	42	25.6	762	1067	not clear	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#4	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	229	Concrete Caisson#5	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	230	Concrete Caisson#6	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	231	Concrete Caisson#7	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	232	Concrete Caisson#8	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	233	Concrete Caisson#9	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	234	Concrete Caisson#10	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	235	Concrete Caisson#11	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	236	Concrete Caisson#12	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	237	Concrete Caisson#13	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	238	Concrete Caisson#14	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	239	Concrete Caisson#15	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	240	Concrete Caisson#16	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	241	Concrete Caisson#17	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	242	Concrete Caisson#18	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	243	Concrete Caisson#19	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	244	Concrete Caisson#20	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	245	Concrete Caisson#21	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	246	Concrete Caisson#22	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	247	Concrete Caisson#23	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	248	Concrete Caisson#24	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	249	Concrete Caisson#25	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	250	Concrete Caisson#26	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	251	Concrete Caisson#27	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	252	Concrete Caisson#28	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	253	Concrete Caisson#29	84		30	42	25.6	762	1067		working load 250 Ton/caisson	
	254	Concrete Caisson#30	84		30	42	25.6	762	1067		working load 250 Ton/caisson	

Table G-1  
Project Number: Foundation Elements - Highway 401 and Leslie Street Interchange  
TRANETO801245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated		Diameter	Base Diameter ** Estimated based on table no as-builts	Depth	Diameter	Base Diameter	Battered	Legend and Notes		Notes
				(ft)	(in)							(m)	(mm)	
37-206/5 Hwy 401 over pass at Leslie St. RAMP W-N/S (6 Spans)	618	12BP53 H-Pile	50	*		30	42	27.4	762	1067	1 to 3			Note data 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
	619	Concrete Caisson	85	*		30	42	25.9	762	1067	1 to 6			
	620	12BP53 H-Pile	60	*		30	42	18.3	762	1067	1 to 6			
	621	12BP53 H-Pile	56	*		30	42	17.1	762	1067	1 to 6			
	622	Concrete Caisson	77	*		30	42	23.5	762	1067	1 to 6			
	623	Concrete Caisson	77	*		30	42	23.5	762	1067	1 to 6			
37-206/6 Hwy 401 over pass at Leslie St. RAMP E-W (2 Spans)	624	12BP53 H-Pile	88	*		30	42	26.6	762	1067	1 to 3			Note data 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
	728	12BP53 H-Pile	84	*		30	42	25.6	762	1067	1 to 3			
	729	Concrete Caisson#1	75	*		30	42	22.9	762	1067	1 to 3			
	729	Concrete Caisson#2	75	*		30	42	22.9	762	1067	1 to 3			
37-206/7 Hwy 401 over pass at Leslie St. RAMP W-W (6 Spans)	518	Concrete Caisson	74	*		30	36	22.6	762	914	1 to 6			Note data 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
	519	Concrete Caisson	72.5	*		30	36	22.1	762	914	1 to 6			
	520	12BP53 H-Pile		*		30	36				1 to 6			
	521	12BP53 H-Pile				30	36				1 to 6			
	522	Concrete Caisson	77	*		30	36	23.5	762	914	1 to 6			
	523	Concrete Caisson	88	*		30	36	26.6	762	914	1 to 6			

# Appendix H

## **Evaluation of Foundation Alternatives**



**Table H-1. Foundation Options for Single-Span Rigid Frame C.N.R. Overpass Structure**

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations	-	-	Low cost	-not feasible due to the prevailing subsurface conditions
Driven steel H-piles foundations	<ul style="list-style-type: none"> <li>-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles</li> <li>-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)</li> <li>-Cannot be installed prior to removal of existing superstructures</li> </ul>	<ul style="list-style-type: none"> <li>-Some Interference of the existing structure foundation is expected depend on the proposed pile foundation layout</li> <li>-Cobbles, boulders may be encountered during the driving, which may present problems</li> <li>-Vibration and noise</li> <li>-Possible problems with slope instability to north-west</li> </ul>	Moderate cost	<ul style="list-style-type: none"> <li>-Feasible but may be unacceptable due to environmental considerations (i.e. noise and vibration generated by construction)</li> <li>-The use of driven piles will need further study due vibrations and noise generated</li> </ul>
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	<ul style="list-style-type: none"> <li>-Less vibrations and noise created than driven piles</li> <li>-Cannot be installed prior to removal of existing superstructure</li> </ul>	<ul style="list-style-type: none"> <li>-Some Interference of the existing structure foundation is expected depend on the proposed pile foundation layout</li> <li>-The presence of cobbles and boulders may present problems during the installation of drilled caisson foundations.</li> <li>-Basal heave possibility if extended deeper and/or close to the base generally sand sand layer. May require dewatering</li> </ul>	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 steel H-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"> <li>-Minimizes vibrations and dewatering.</li> <li>-Can be installed in low overhead conditions</li> <li>- Expensive due to special equipment / material and specialist contractor</li> </ul>	<ul style="list-style-type: none"> <li>-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</li> <li>-Less geotechnical resistance than caissons or piles</li> </ul>	Expensive	-Feasible but more expensive than driven H-piles and drilled caissons.
CFA (continuous flight auger pile)	<ul style="list-style-type: none"> <li>- Rapid installation accelerates foundation construction, which reduces project schedules</li> <li>-Hydrostatic uplift can be counter-balanced by concrete or a sand cement mix</li> <li>- Suitable for low headrooms or confined spaces if segmental augers are available</li> <li>-Limited installation noise and vibration for sensitive urban environments</li> <li>-Disposal of spoils will be problem if soil at the site is contaminated</li> <li>-Local installers may not have powerful enough equipment to reach desired depths below ground surface</li> </ul>	<ul style="list-style-type: none"> <li>- Problems may arise during the installation due to cobbles, boulders due to the less torque of CFA machine than normal caisson installation machine.</li> <li>-Depth may be an issue</li> <li>-Quality control issue (automated monitoring equipment provides real-time quality control can overcome this issue)</li> </ul>	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 I

**List of OPSSs, OPSDs 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
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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
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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.