

**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORTS
LESLIE STREET OVERPASS STRUCTURE
HIGHWAY 401 REHABILITATION FROM
LESLIE STREET TO WARDEN AVENUE
MTO CENTRAL REGION, G.W.P. 2130-01-00
GEOCRES 30M14-332**

Delcan Corporation
Project: TRANETOB01245AA-AD
September 30, 2011

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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
Leslie Street Overpass Structure, Highway 401 Rehabilitation from Leslie Street to
Warden Avenue, MTO Central Region, G.W.P. 2130-01-00, GEOCRE 30M14-332**

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

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

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Principal Engineer

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MTO CENTRAL REGION, G.W.P. 2130-01-00**

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by Delcan Corporation (Delcan) to carry out a preliminary foundation engineering investigation at the Site of the Highway 401 and Leslie Street interchange in Toronto, Ontario.

The existing interchange consists of seven (7) structures, as described below and shown on Drawing 1.

Structure No.	Description of Structure	No. of Spans
37-206/1	East Bound Lanes – Collectors	16 spans
37-206/2	West Bound Lanes – Collectors	33 spans
37-206/3	East Bound Lanes – Core	16 spans
37-206/4	West Bound Lanes – Core	16 spans
37-206/5	Ramp W-N/S	6 spans
37-206/6	Ramp N-E	2 spans
37-206/7	Ramp N-W	6 spans

The construction of the interchange started in the mid 1950's and modifications were made in the mid 1960's and early 1990's. The site has therefore been the subject of various geotechnical assessments over the past six decades. We understand that the existing structures, which span over Leslie Street and the CNR tracks, are nearing the end of their serviceable life expectancy and that a number of rehabilitation alternatives is currently being considered.

In 2010, Coffey reviewed the existing geotechnical data and submitted a report describing the overall subsurface conditions (see 'Foundation Engineering Assessment Report, Highway 401 and Leslie Street Interchange, Toronto, Ontario, G.W.P. 2130-01-00, Dated March 22, 2010).

As part of the rehabilitation, three highway ramp structures were planned to be rehabilitated and Coffey was asked to conduct a preliminary investigation for the following 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)

As Coffey's foundation investigation was carried out for the previous rehabilitation plan (involving the ramp structures), boreholes drilled by Coffey were generally located outside the footprint of the more recently proposed new structures. No additional boreholes were advanced for the proposed new structures and Coffey was asked to prepare this preliminary foundation investigation report based on the available subsurface information only, including Coffey's recent boreholes, wherever applicable.

2 SITE DESCRIPTION AND GEOLOGY

The site is located at the Highway 401 interchange with Leslie Street, in North York (Toronto), Ontario. At this location, Highway 401 crosses over both Leslie Street and the CNR track(s), adjacent and parallel to Leslie Street. In general, the grade along this section of 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 CNR track is about 141 m while Leslie Street below Highway 401 is at approximately El. 136 m. The grade at the site also falls from North to South.

Within this general area, the overburden consists of Pleistocene or glacial deposits, which were laid down under a vast thickness of ice or else within the glacial rivers and lakes associated with them. Soils which were deposited by the ice itself form the glacial till deposits which are mainly unsorted, heterogeneous materials, while those formed by melt waters are typically stratified deposits. Near and within the present river valleys, these deposits were modified by rivers emptying into the glacial Lake Iroquois, the fore runner of the present Lake Ontario, whereby sand, gravel, silt and minor amounts of clay were deposited. Finally, further modifications took place in the more recent era by the West Don River and the East Don River.

In summary, below some man-made fill and/or modern post-glacial deposits, the stretch of land along Highway 401 near Leslie Street is underlain by silty sands (shallow lake deposits – Peel Pond), silty clay (deeper lake deposits – Peel Pond), glacial tills and intermittent silt and sand layers.

The depth of the overburden in the general area can be expected to be more than 40 m, with the surface of the shale bedrock anticipated at about El. 75 m to 90 m. The uppermost grey shale, known as the Georgian Bay Formation, belongs to the upper Ordovician Period of the Palaeozoic era and is approximately 440 million years old.

3 SUBSURFACE CONDITIONS

3.1 Past Reports

As previously mentioned, a number of geotechnical investigations has been conducted for the Highway 401 and Leslie Street interchange, including the viaduct site, between 1953 and 1990. A complete overview of the studies are presented in Coffey report TRANETOB01245AA-AB, dated March 22, 2010. The following section provides a brief overview of the purposes and results of these studies conducted at the interchange.

Copies of the borehole logs, from these reports, as well as laboratory results plans and sections, where available, are presented in Coffey report TRANETOB01245AA-AB, dated March 22, 2010, and not copied herein.

The Foundation Company of Canada Limited, Toronto Bypass Highway #401, Soil Conditions – C.N.R. & Leslie St. Overpass, C7142, September 30, 1953.

The purpose of this study was to assess the embankment failure which took place during its construction of the west approach of the core lanes and to provide remedial measure recommendations for the proposed embankments. Nineteen (19) boreholes were advanced for this study, designated as G-Series of boreholes (i.e. G1, etc.). This report was also presented under the cover of Geocon Limited with the same date.

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 to comment on if a reduction of the previously recommended berm requirements could be altered. Three (3) boreholes, designated as Boreholes 1, 2 and 3, were advanced in proximity of the previously drilled boreholes.

Department of Highways Ontario (now Ministry of Transportation, Ontario – MTO), 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 (2) dynamic cone penetration tests were performed, designated as B-Series boreholes (i.e. B1, B2, etc.).

Department of Highways Ontario, Foundations Section, Materials and Testing Division, Report on Vertical and Lateral Load Tests on 30" ϕ Concrete Caisson and Steel H-Pile at Leslie Street and Hwy. 401 Interchange, W.P. 266-61, April 9, 1965.

This memorandum provides the results of load tests conducted at the site, one borehole log and a cross section indicating Boreholes 201 to 211 which were advanced by H. Q. Golder and Associates Ltd., Report No. 6205, dated October 1962. As noted above, only one borehole log was available for purposes of our data compilation. However, subsurface data were obtained by scaling from the cross section provided. Boreholes were designated 201, etc.

Department of Highways Ontario, Materials and Testing Division, Structures on Leslie St. & Hwy. 401 Interchange, W.P. 252-61-3, April 5, 1966.

This memorandum indicated twelve (12) borings were advanced following the blow-out of caissons 424-3 and 426-3. The twelve (12) borings were advanced in proximity to the proposed caissons. Borehole logs were however not provided.

Department of Highways Ontario, Foundations Section, Materials and Testing Division, Caisson Installation, Structure on Leslie St. & Hwy. 401 Interchange, W.P. 252-61-3, January 26, 1967.

This memorandum provides a discussion of the subsurface conditions encountered by Dominion Soil Investigation Limited (Soil Investigation, Hwy. 401 and Leslie Street, 6-12-1, W.P. 266-61, February 9, 1967) in proximity to the caissons to be used to support the core lanes and recommendations for the installation of future caissons. Eighteen (18) borings were advanced, designated 1A, 2A, etc.

Ministry of Transportation, Engineering Material Office, Foundation Design Section Foundation Investigation Report, Structure Widening Leslie Street & C.N.R. Overpass Hwy 401, W.B. Collector Lanes, W.P. 260-86-01/A, February 21, 1990.

This report presents a historic summary of work conducted related to the embankment, west of the railway tracks, and the results of a recent foundation investigation conducted for the proposed widening of the West Bound Collector Lanes. For this work, an additional five boreholes was advanced (Designated 1-1, etc.).

Additional Studies

Some additional subsurface exploration work was also conducted in the area but not incorporated into this work due to the shallow depths of the boreholes or distance from the site.

3.2 Compiled Subsurface Conditions Based on Previous Data

3.2.1 Background

To study the subsurface conditions at the newly proposed Highway 401 Leslie Street overpass structure, 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 investigations, estimated subsurface profiles along the West Bound, Core and East Bound Collector Lanes are presented in Drawing 2.

The following assumptions were made for this project:

- Elevations were assumed to be based on the geodetic datum.
- The surface elevation was based on those indicated on the borehole logs and no correction was made; existing topography was unavailable.
- Depths noted below, for the various units, were based on measurements below the existing ground surface, at the time the explorations were completed, no correction was made.
- Imperial elevations were directly converted to metric and no correction factor was used.
- Locations of boreholes were approximated based on those indicated on the drawings provided in the referenced reports.

The following provides a compiled overview of the subsurface conditions encountered at the surrounding area of the newly proposed Highway 401 / Leslie Street 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 Leslie Street 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 Leslie Street 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
Department of Highways Ontario, 1964	B7	139.6	114.9	No
	B8	143.6	113.1	No
	B15	143.3	115.5	No
	B16	137.5	112.6	No
Dominion Soil Investigation Limited, 1967	1A	141.4	114.8	No
	2A	141.6	114.0	No
	3A	145.0	114.0	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

An about 0.15 m thick topsoil layer was encountered at the ground surface (at the time of investigation) at Boreholes B7, B8 and B16.

Boreholes 2A and 3A contacted a granular fill consisting of sand and silt in Borehole 2A. The granular fill was found to contain some gravel. The presence of traces of organic matter and traces to some clay was also noted within the fill. The granular fill was encountered from the ground surface (at the time of investigation) to depths ranging from approximately 4.3 m to 5.2 m below the ground surface, at the time of the explorations, or to approximate elevations 140.7 to 136.4 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 fill is considered to be a granular material. One Standard Penetration test performed at the bottom of the fill layer in Borehole 2A yielded an N-value of 69 blows/0.3 m, which indicated a very dense condition at that location.

3.2.3 Surficial Granular Soils

Underlying the topsoil (Boreholes B7, B8 and B16), fill materials (Borehole 2A) and from the ground surface (Boreholes 1A and B15), surficial fine granular soils were encountered at approximate elevations of 143 m to 136 m. The thickness of the surficial granular soils encountered ranged between 4.7 and 13.6 m, with an average of approximately 9.2 m.

This stratum was typically described as brown to grey, silty fine sand, silt and fine sand, silt to clayey silt with sand and gravel. The presence of organics was noted at one borehole location. Based on the available data, this stratum is considered a granular (i.e. non-cohesive) material.

In the B-series boreholes (i.e. Boreholes B7, B8, B15 and B16), no field testing was conducted, while in the A-series boreholes, SPT 'N'-values of 12 to in excess of 100 blows/0.3 m were recorded within these surficial granular soils, indicating a compact to very dense relative density.

3.2.4 Cohesive Soils

Grey silty clay and silty clay to clayey silt were encountered, below the surficial granular soils, at El. 132.9 to 129.2 m (average El. 131.1 m). Lower portion of this clayey soil deposit in Boreholes 1A and 2A was identified as a more sandy material below El. 125 m and could be of glacial till origin, similar to the underlying soils. This till like material and the underlying clayey silt till were not recognized as separate units 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 from foundation engineering viewpoint based on the strength parameters measured nearby the site by in-situ and laboratory tests. This cohesive deposit (combined upper and lower zones) was found to be approximately 10.1 to 13.1 m thick (average 12.5 m).

SPT 'N'-values of 4 to 13 blows/0.3 m were recorded within this deposit, indicating a soft to stiff consistency.

The presence of cobbles and boulders may be anticipated in the lower (i.e. sandy soil below El. 125 m) zones, due to the mode of deposition.

3.2.5 Glacial Till

Underlying the cohesive soil deposit, a glacial till was encountered in the boreholes at depths ranging from about 19.8 m to 26.2 m below the existing ground surface at the time of the explorations or at elevations of approximately El. 119.8 m to 117.3 m (Average El. 118.6 m). The till deposit was described as a heterogeneous mixture of clayey silt, sand and trace of gravel (B-series boreholes by the Department of Highways Ontario, 1964) to sandy silt till, silt till and clayey silt till (A-series boreholes by Dominion Soil Investigation Limited, 1967).

SPT 'N'-values ranging from 63 to in excess of 100 blows/0.3m were recorded for this heterogeneous mixture of clayey silt, sand and trace of gravel in B-series boreholes (Boreholes B7, B8, B15 and B16), indicating a very dense condition. Some of the high 'N'-values may be recorded due to the presence of cobbles and boulders.

SPT 'N'-values ranging from 63 to in excess of 100 blows/0.3 m were recorded for above mentioned various soils in A-series boreholes (Boreholes 1A, 2A and 3A), indicating a very dense relative density.

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

Boreholes B7, B8, B15 and B16 were terminated within the glacial till deposit at depths of 24.7 to 30.5 m or El. 115.5 to 112.6 m.

It should be pointed out that in Borehole A-3, a 0.7 m thick fine to coarse sand layer, was encountered at El.118.9 m, sandwiched between the silty clay deposit and the underlying glacial till deposit. This water bearing sand layer is a granular (i.e. non-cohesive) soil type and based on the recorded N-values of 29 and 72 blows/0.3 m its relative density is described as dense.

3.2.6 Basal Granular Soils

Underlying the till (Boreholes 1A, 2A and 3A, Dominion Soil Investigation Limited, 1967), a basal sand deposit was encountered at depths ranging from about 25.5 to 29.4 m (Average 27.2 m) below the existing ground surface at the time of the explorations or at elevations of 115.9 to 114.8 m (Average El. 115.4 m). This basal sand was not fully explored and its lateral and vertical extent is unknown (i.e. even the deeper boreholes were all terminated within the lower sand after 0.9 to 1.5 m penetration into the layer).

The sand deposit was described as a fine to coarse sand. It is therefore considered to be a granular (non-cohesive) soil type.

Within this deposit, a slight artesian condition was observed at or below El. 115.5 to 114.6 m.

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

3.2.7 Groundwater Conditions

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

Table 3.2.7.1: Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Remarks
1A	141.4	-	-	"Slight artesian pressure below El. 379 ft" or El. 115.5 m, while drilling.
2A	141.6	-	-	"Slight artesian pressure below El. 376 ft" or El. 115.0 m, while drilling.
3A	145.0	-	-	"Slight artesian pressure observed below El. 379.1 ft." or El. 115.5 m, while drilling.
B7	139.6	-	2.8/136.8	-
B8	143.6	-	7.9/135.7	-
B15	143.3	-	12.3/130.9	-
B16	137.5	-	8.9/128.5	-

* may not be stabilized

As can be seen from the table, the observed groundwater levels in Boreholes B7, B8, B15 and B16, upon their completion, ranged between elevations of approximately 137 and 129 m. These boreholes were terminated in the glacial till deposits, while in Boreholes A1, A2 and A3, which were extended below the till into the underlying sand deposit, a slight artesian condition was observed while drilling. This artesian condition is believed to be emanating from the sand deposit underlying the till. 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 less permeable cohesive soil deposit, especially during rainy periods/spring thaw.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The groundwater in the upper units may also be controlled by the water level in the watercourse located about 200 m east of the site, known to be a branch of the 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 one borehole is located close to the newly proposed Leslie Street overpass structure, as follows:

Table 3.3.1: Borehole Location and Drilling Depth

Borehole No.	Location (Coordinates)		Existing Ground Surface Elevation (m)	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
	Northing	Easting			
Highway 401 Overpass at Leslie Street/CNR Ramp N-E (# 37-206/6)					
E2	315835.3	4847329.4	140.4	28.8	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 borehole was put down using a track mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. The borehole was advanced using continuous flight hollow-stem augers.

Samples in the borehole 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 bottom of borehole. 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 borehole were observed during drilling and upon its completion. The borehole was grouted upon its completion using a cement/bentonite mixture as per MTO procedures.

The borehole location was established in the field by Coffey engineering staff, in relation to the existing features. The location was then tied in and the geodetic elevation of the ground at the borehole location was 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 and Atterberg Limits tests was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

3.4 Subsurface Conditions Encountered During Recent Investigation

As mentioned before, one (1) borehole (Borehole E2) was advanced adjacent to the newly proposed Leslie Street overpass structure.

The plan location of Borehole E2 is presented on Drawing No. 1. Details of subsurface conditions encountered at borehole location for the investigation, including results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheet in Appendix A. Detailed laboratory test results are enclosed in Appendix B.

In general, the sub-surface stratigraphy encountered in the borehole comprises fill material and a surficial fine grained granular soil deposit overlying typically firm to stiff silty clay, which is, in turn, underlain by a cohesive glacial till deposit. The glacial till deposit is further underlain by a basal granular soil consisting of sand. Borehole E2 was terminated within this basal granular soil deposit.

Details of the sub-surface conditions encountered in Borehole E2 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 Fill

Borehole E2 contacted silty sand fill extending to a depth of 5.1 m below the ground surface or to El. 135.3 m. The fill consists of silty sand and contains traces of clay and gravel.

The grain-size distribution of a sample from the fill is given in Figure B-1, in Appendix B, with the following grain-size distribution, 1% gravel, 53% sand and 46 % silt and clay size particles.

Standard penetration tests performed in the basically granular fill (i.e. silty sand) yielded N-values of 4 to 56 blows/0.3 m, indicating a very loose to dense relative density. These N-values indicate that some portions of the fill have received a reasonable degree of compaction when the fill was first placed, while others did not (i.e. the middle zone does not appear to have received a systematic compaction).

3.4.2 Surficial Granular Soil

Underlying the fill, Borehole E2 contacted an about 3.1 m thick surficial granular (non-cohesive) soil consisting of silt with some fine sand, at a depth of 5.1 m or El. 135.3 m. The grain-size distribution of a sample from the surficial granular soil is given in Figure B-2, in Appendix B, with the following grain-size distribution, 0% gravel, 12% sand and 79% silt and 9% clay size particles. The material was noted to exhibit dilatant behaviour in the presence of water.

Standard Penetration tests performed in the deposit yielded N-values of 8 to 14 blows/0.3 m, indicating a loose to compact relative density.

3.4.3 Silty Clay

Borehole E2 contacted a 12.3 m thick silty clay deposit below the surficial granular soils deposit, at a depth of 8.2 m or at El. 132.2 m. The deposit was found to extend to a depth of 20.5 m below the ground surface or to El. 119.9 m. This cohesive soil deposit frequently has a varved like or stratified structure and contains thin seams of silty sand to sandy silt and clayey silt. The presence of traces of sand and gravel is also noted within the deposit.

The grain-size distribution of a sample from the silty clay layer is given in Figure B-3, in Appendix B. The following grain-size distribution is indicated: 0% gravel, 6% sand, 35 % silt and 59 % clay size particles.

The results of Atterberg limits test, performed on two samples from the deposit is presented in Figure B-4 in Appendix B. The test results yielded the following index values:

Liquid Limit:	37-40 %
Plastic Limit:	18-20 %
Plasticity Index:	19-20
Natural Moisture Content:	31-37 %

These results are characteristic of clayey soils of medium plasticity and the fact that measured natural moisture contents are generally closer to the measured liquid limits, rather than the measured plastic limits indicates the likelihood of an only slightly over-consolidated soil deposit.

Standard penetration test conducted in the silty clay deposit gave N-values which range from 2 to 8 blows/0.3 m. Undrained-shear strengths, as measured by MTO "N" type field vane, varied from 28 to 52 kPa, indicating a firm to stiff consistency. It should however be pointed out that some of the higher undrained shear strengths may have been obtained from the observed sandy zones within the silty clay deposit.

3.4.4 Glacial Till

Borehole E2 contacted a glacial deposit, consisting of clayey silt till, underlying the silty clay at a depth of 20.5 m or at El. 119.9 m. The clayey silt till was found to be 4.5 m thick and extended to a depth of 25.0 m below the ground surface or to El. 115.4 m.

The grain-size distribution of a sample from the clayey silt till layer is given in Figure B-5, in Appendix B, with the following grain-size distribution, 3% gravel, 29% sand, 50 % silt and 18 % clay size particles.

The results of Atterberg limits test, performed on a sample from the deposit is presented in Figure B-6 in Appendix B. The test results yielded the following index values:

Liquid Limit:	16 %
Plastic Limit:	11 %
Plasticity Index:	5
Natural Moisture Content:	11 %

These results are characteristic of clayey soils of low plasticity and the fact that measured natural moisture content is near to the measured plastic limit indicates the likelihood of a heavily over-consolidated soil deposit.

Standard penetration test N-values of 27 to 84 blows/0.3 m were recorded, showing a wide variation. A field vane test was also performed and this yielded an undrained shear strength of 64 kPa. Based on these field test results, the consistency of the deposit in this borehole can be described as firm to hard.

Due to the mode of deposition, presence of cobbles and boulders should be anticipated within the till deposits.

3.4.5 Basal Granular Soil

Below the clayey silt till, Borehole E2 contacted a basal sand deposit at a depth of 25.0 or El. 115.4 m. Borehole E2 was terminated within the basal sand at a depth of 28.8 or El. 111.6 m. This deposit contains traces of gravel and silt. While drilling in the deposit, an about 3.7 m and 2.5 m soil back up was observed at El. 114.5 m and 111.6 m. Dynamic cone penetration tests (DCPT) were also performed from the bottom of the Borehole E2 (where soil back ups were observed) until refusal on the dynamic cone penetration was encountered.

The grain-size distribution of a sample from the basal sand was determined in our laboratory, which showed 3% gravel, 90% sand and 7% silt and clay size particles (see Figure B-7 in Appendix B).

A standard penetration test (SPT) yielded an N-value of 12 blows/0.3 m at the top portion of this layer. Due to the observed soil back up, no further SPT test was performed in the borehole, below this upper zone.

3.4.6 Groundwater Conditions

Groundwater conditions were observed in the open borehole while drilling and upon completion of Borehole E2. The observations made in the borehole are shown on the Record of Borehole Sheet in Appendix A,

Table 3.4.6.1 Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Remark
E2	140.4	-	19.8/120.6*	Upon completion	Caved-in @ 8.5 m ** Excess soil back-up noted in the basal sand at El. 115 m and below, while drilling

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

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

From the observations it appears that an excess upward groundwater gradient exists in the basal granular soils underlying the till. A similar observation was made in the previously drilled boreholes at the site in 1966.

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 0.2 km east of the Leslie Street, known to be a branch of the East Don River.

For and on behalf of Coffey Geotechnics Inc.


Gwangha Roh, Ph.D.

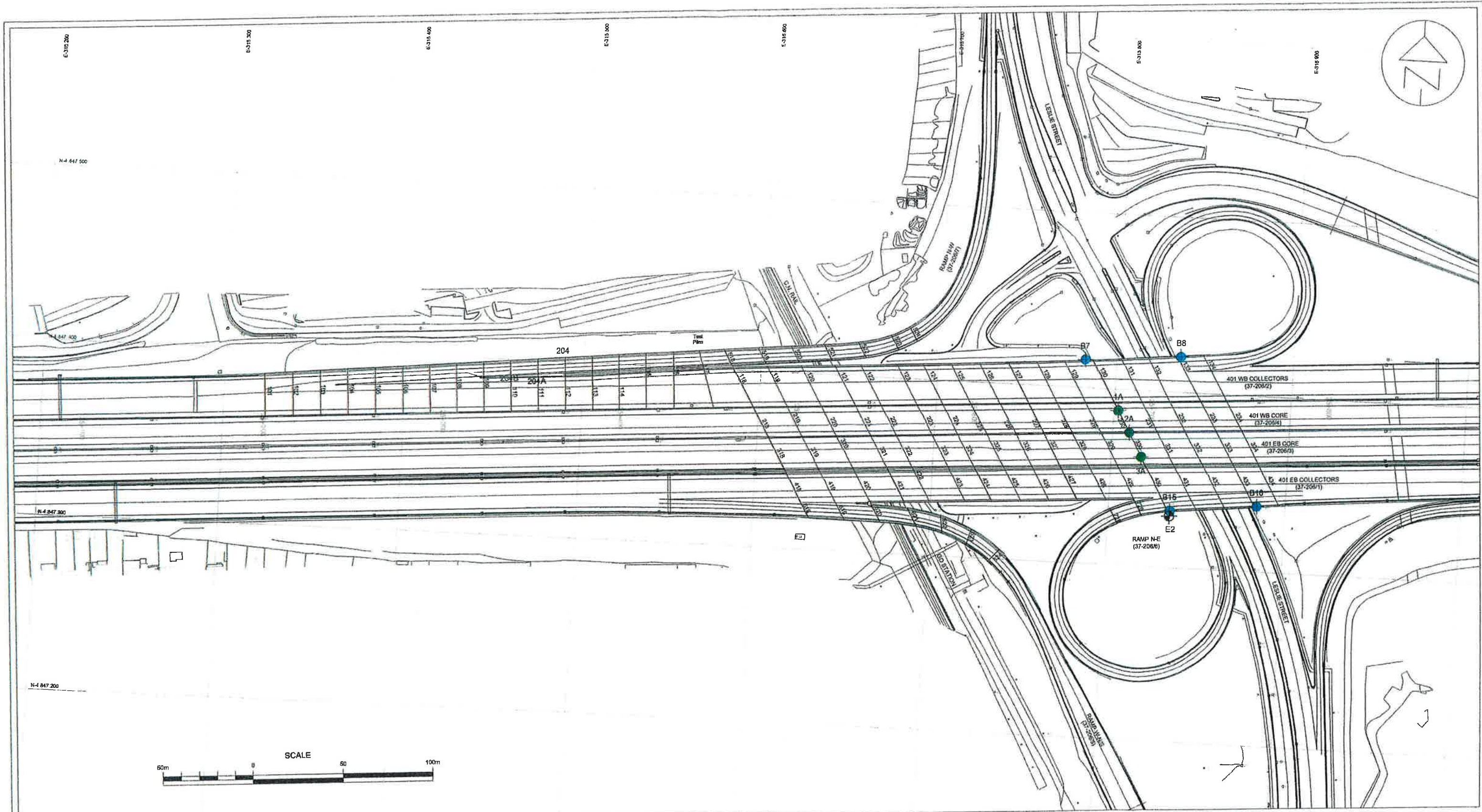

Ramon Miranda, P.Eng.




Zuhtu Ozden, P.Eng.



Drawings



- LEGEND**
- ⊕ Borehole + DCPT (Coffey, 2009)
 - Borehole (Department of Highways Ontario, 1984)
 - Borehole (Dominion Soil Investigation Limited/Department of Highways Ontario, 1987)
 - 1/3 Bore Number (TYP)

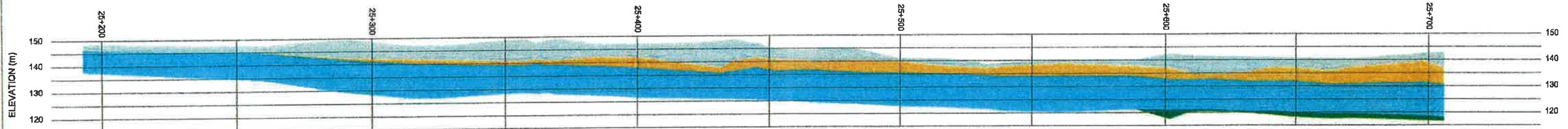
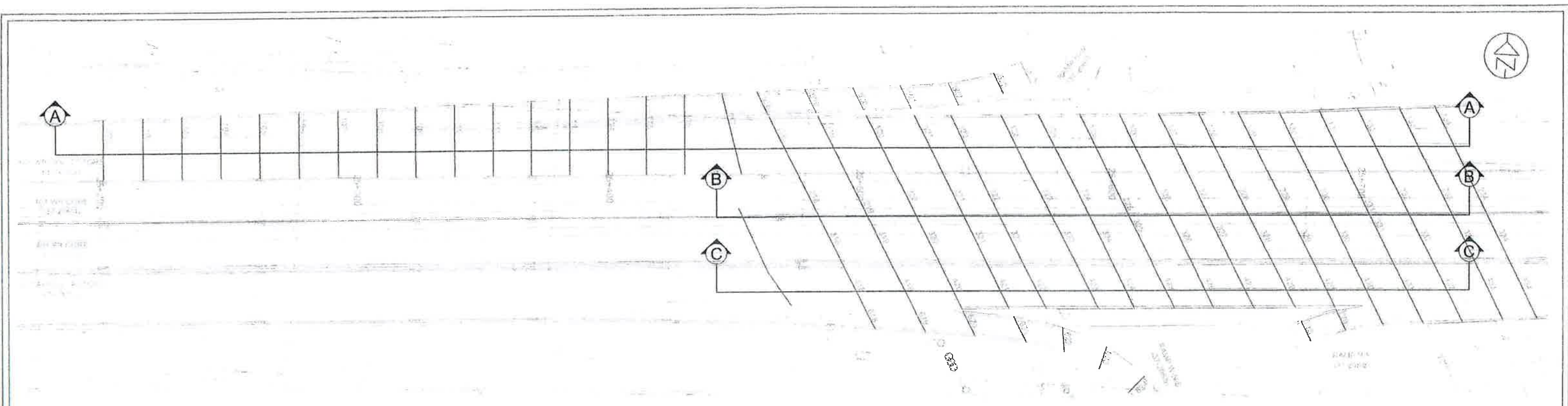
NOTES

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

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

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SPECIALISTS MANAGING THE EARTH

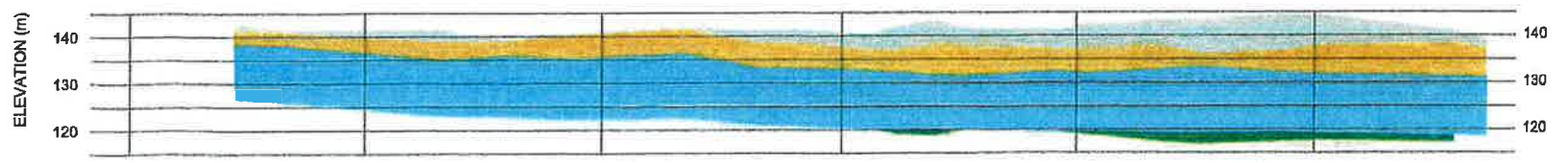
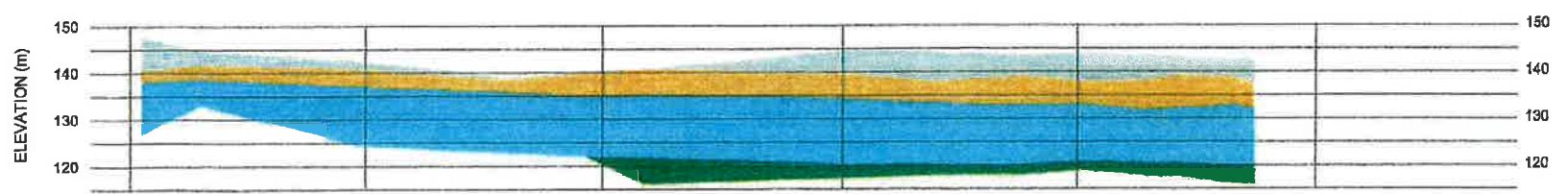
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project:	PROPOSED LESLIE STREET OVERPASS STRUCTURE HIGHWAY 401 AND LESLIE STREET INTERCHANGE TORONTO, ONTARIO		
title:	SITE AND BOREHOLE LOCATION PLAN		
project no:	TRANFTOR01245AA-AD	drawing no:	1



PROFILE A-A
WEST BOUND COLLECTORS
HORIZONTAL SCALE

PROFILE B-B
CENTRELINE OF CORE
HORIZONTAL SCALE

PROFILE C-C
EAST BOUND COLLECTORS
HORIZONTAL SCALE



- LEGEND**
- Fill / Clayey Silt
 - Silty Sand
 - Silty Clay
 - Glacial Till

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

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



client:	DELCAN CORPORATION		
project:	PROPOSED LESLIE STREET OVERPASS STRUCTURE HIGHWAY 401 AND LESLIE STREET INTERCHANGE TORONTO, ONTARIO		
title:	ESTIMATED SUBSURFACE PROFILES		
project no:	TRANETO01245AA-AD	drawing no:	2

Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE NO 7 (G3)

SECTION

DEPARTMENT OF AGRICULTURE
NATIONAL BUREAU OF RESEARCH

NO. 7 (G3) LOCATION Sta. 131400 and 141' Lt. of E. Hwy. 401

DATE OF BORING June 2, 1954

BY B.M.G.

DAY G.S.C.

WATERBORING USING 8 1/2" CASING

BY B.M.G.

H.D.

		RESISTANCE			SOUND DENSITY	REMARKS
458 Groundlevel Topsoil	450					
0.3 Clayey silt with sand and gravel. Brown.	450					
	440					
437 21.0 Silt and fine sand with organic wood material around El. 433.	430					
424 34.0 Silty clay with some sand. Grey.	420					
	410					
	400					
391 67.0 Heterogeneous mixt. of clayey silt, sand and gravel up to 1"Ø. (Glacial Till) V. dense Grey	380	SS 100 for 6" SS 100 for 9"				
377 81.0 End of borehole.	370	SS 100 for 9"				

W.L. at
El. 448.9

DEPARTMENT OF AGRICULTURE
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 3 (38)

FOUNDATION SECTION

JOB 54-F-41

LOCATION Stn. 112+75 and 112+ Lt. of E. Hwy. 401

ORIGINATED BY B.H.G.

W.F. 252-61-3

BORING DATE June 8, 1964.

COMPILED BY B.H.G.

DATUM G.S.C.

BOREHOLE TYPE Washboring using BX casing.

CHECKED BY H.D.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT. PLOT	SAMPLES NO.	ELEV. SCALE FOOT	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	WATER CONTENT %	BULK DENSITY PCF	REMARKS
471.0	Ground level			470				
0.6								
	Silty sand, trace of clayey silt in places.			460				
	Brown changing to grey at El. 434			450				
				440				
				430				
426				420				
45.0	Silty clay with some sand.			410				
	Grey.			400				
				390				
385				380				
86.0	Heterogeneous mixt. of clayey silt, sand & trace of gravel up to 1/2" Ø. V. dense Grey		1 53 101					
			2 31 80					
			3 53 100					
			for 9"					
			4 33 100					
			for 10"					
100.10	End of borehole.							

W.L. at
El. 445.2

RECORD OF BOREHOLE NO 13 (B15)

FOUNDED ON SECTION

64-P-41

LOCATION ON Sta. 132+50 to 148' Rte. of E. Hwy. 401

OPERATED BY B.H.G.

232-01-3

BORING DATE June 18, 1964

PROJECT BY B.H.G.

64-P-41

BORING METHOD Washboring using BX casing.

NO. OF TESTS 1

EL. (FEET)	STRAT. PLAN	DEPTH (FEET)	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	WATER CONTENT	BULK DENSITY	REMARKS
429.0	Groundlevel	470				
0.0		460				Silt & fine sand. Traces of clay and fine gravel. Brown changing to gray at El. 434
431.5		450				
38.6		440				
431.5		430				Silty clay with some sand. Gray.
		420				
		410				
		400				
389		390				
81.0			1 SS 106			Heterogeneous mixt. of clayey silt, trace of sand & gravel up to 1"Ø. (Glacial till) V. dense.
			2 SS 100			Gray.
379			3 SS 100			
91.0			for 11"			End of borehole.
			for 6"			

U.L. at
El. 429.5

DEPARTMENT OF HIGHWAYS DIVISION
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 16 (B16)

FOUNDATION SECTION

LOG 64-F-41 LOCATION Stn. 134+15 and 143' E. of E Hwy, 401 ORIGINATED BY B.H.G.
 W.P. 252-61-1 BORING DATE June 18, 1964 COMPILED BY B.H.G.
 DATA G.S.C. BORING TYPE Washboring using RZ casing. CHECKED BY M.D.

ELEV DEPTH	DESCRIPTION	STATION PLOT	NUMBER TYPE	REMARKS	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	WATER CONTENT %	WATER CONTENT %	BULK DENSITY P.C.F.	REMARKS
4.91	Groundlevel								
9'-6"	Silty fine sand, trace of clay and fine gravel. Brown.								
17.0'	Silt and trace of fine sand. Grey								
22.0'	Silty clay with some sand. Grey.								
386.2									
64.10	Heterogeneous mixt. of clayey silt, silt, sand & trace of gravel up to 2". (Glacial till) V. dense Grey		1 SS 63						
			2 SS 108						
			3 SS 100						
			for 11"						
			4 SS 64						
369.5			5 SS 81						
82.6	End of borehole.								

W.L. at
El. 421.7

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

OUR REFERENCE NO. 6-12-1
YOUR REF. NO. W.P. 255-81

CLIENT D.H.O.

PROJECT HWY. NO. 401 B. LESLIE ST.

LOCATION BETWEEN CAISSONS 230-1 & 230-2

DATUM ELEVATION G.S.C.

METHOD ST. 304-40 WASHERBORING
QUANTITY OF BOREHOLE 2 3/8"
DATE JAN. 3-5, 1967

ENCLOSURE NO.

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	LOCATION	1	2	3	4	P _L	W	L _L	
461.9	0	GROUND SURFACE												
460														
450	10	Compact to Dense Brown SILTY FINE SAND		1	SS	32								
440	20													
430.0	28.0	Soft to Firm Grey CLAYEY SILT to SILTY CLAY with some sand		2	SS	4								
420	40													
410	50													
400	60	Sandy below el. 410 ft		4	SS	11								
392.9	71.0	Very Dense, Grey SANDY SILT with a trace of clay and embedded gravel. (slightly cemented) to SILT with a trace to some clay. (GLACIAL TILL.)		6	SS	13								
390				7	SS	59								
385.4	79.5			8	SS	175/11								
380	80			9	SS	78								
374	87.5	Very Dense, Grey FINE to MED. SAND with a trace of fine grav.		10	SS	110								
370	90	END OF BOREHOLE		11	SS	154/8								
				12	SS	100/6								
				13	WS	—								
				14	SS	200/6								

SLIGHT ARTESIAN PRESSURE BELOW EL. 379 ft.

VERTICAL SCALE, 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE D.A.M. CHD

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

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

CHEM. O.H.O.

PROJECT HWY NR 401 & LESLIE ST.

LOCATION BETWEEN CAISSONS 230-3 & 330-1

DATUM ELEVATION G.S.C.

METHOD OF TESTS WASHBORNS

DRABER OF BORING

DATE JAN 5-6, 1967

ENCLOSURE NO.

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE		CONSISTENCY water content %	REMARKS
				NUMBER	TYPE	Approximate depth of Sample	20	40		
460	0	GROUND SURFACE								
450	10	Brown SAND and SILT with some gravel, trace of clay and organic matter. (FILL)		1	SS	69				
440	20	FINE SAND								
430	30	brown-grey		2	SS	14				
420	40	FIRM to STIFF Grey SILTY CLAY		3	SS	3				
410	50	Sandy below EL 410± ft		4	SS	10				
400	60									
390	70									
380	80	VERY DENSE, Grey SANDY SILT with embedded gravel		5	SS	53				
370	90	SILT with a trace of clay and gravel (TILL)		6	SS	143				
	100			7	SS	100/4				
	110			8	SS	100/4				
	120	HARD, Grey CLAYEY SILT with embedded gravel (TILL)		9	SS	100/4				
	130			9A	WS	-				
	140	VERY DENSE Medium SAND		10	SS	100/4				
	150			11	WS	-				
	160			12	SS	100/4				
	170	END OF BOREHOLE								

SLIGHT ARTESIAN PRESSURE BELOW EL. 376ft

VERTICAL SCALE 1 IN 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE: D. A. M. CHD

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

OUR REFERENCE NO. G-12-1
 Your Ref. No. W.P. 266-61
 CURVE: C.H.D.
 PROJECT: I.W.W. No. 401 S. LESLIE ST.
 LOCATION: BETWEEN CAISSONS 330-2 & 330-3
 DATUM ELEVATION: G.S.C.

METHOD OF BORING: AUGERING & WASHBORING
 DIAMETER OF BOREHOLE: 3" @ 23g
 DATE: DEC 19-23, 1966

ELEVATION E DOWN	STRATIFICATION SYMBOL	STRATIFICATION DESCRIPTION	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %		REMARKS
			NUMBER	TYPE	NO. of Advances of sampler	2.0	4.0	6.0	8.0	10.0	1	2	
470	0	GROUND SURFACE											
470		Brown SAND and SILT with some clay											
470		FILL											
467		DENSE Greenish Brown SILTY FINE SAND with organic matter	1	SS	2								
450		VERY DENSE to COMPACT SILTY FINE SAND	2	SS	106								
440		Brown Grey (wet below El. 443 ft.)	3	SS	12								
430		FIRM to STIFF Grey SILTY CLAY	4	SS	6								
420		with a trace of fine gravel and occasional fine sand seams.	5	SS	5								
410			5A	W.S.	—								
400			6	SS	7								
390			7	SS	7								
380			8	SS	13								
370			8A	W.S.	—								
360		DENSE Fine to Coarse SAND	9	SS	29								
350		VERY DENSE, Grey SILT, trace of clay (slight to no cohesion and plasticity)	10	SS	72								
340			11	SS	77.5								
330		HARD CLAYEY SILT (GLACIAL TILL)	12	SS	104.9								
320		VERY DENSE Medium to Coarse SAND	13	SS	84								
310			14	SS	142								
300			15	SS	137.10								
290			16	SS	150								
280			17	SS	—								
270		END OF BOREHOLE	18	SS	80.21								

SLIGHT ARTESIAN
 PRESSURE OBSERVED
 BELOW EL. 379-1 ft.

VERTICAL SCALE 1 IN TO 10 FT

DOMINION SOIL INVESTIGATION LIMITED

MADE O. A. M. C.H.D.

TRANET0801245AA HWY401/Lesko St Ramp N-E

RECORD OF BOREHOLE No E2

1 OF 3

METRIC

GWP 2008-E 0012

LOCATION

N-E Ramp of Lesko Street to 401 - Northing 4647329.4 & Easting 315235.5

ORIGINATED BY RK

DIST

HWY 401

BOREHOLE TYPE

Hand Stem Augers, DCPT

COMPILED BY SK

DATUM Geodetic

DATE

1/8/2010

CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		WATER CONTENT (%)	UNIT WEIGHT γ _m	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	SIRAI PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	10 20 30			
140.4 0.0	GROUND SURFACE											
	tr. pebbles, some gravel v. dense compact		1	SS	56		140					
			2	SS	27		139					
	v. loose to loose		3	SS	4		138					
	FILL: SILTY SAND tr. gravel tr. clay brown, moist		4	SS	4		137					
	compact to dense		5	SS	21		136					
			6	SS	34		135					
			7	SS	11		134					
135.3 5.1			8	SS	6		133					
	SILT some fine sand grey, dilatant, loose to compact, wet		9	SS	14		132					
			10	SS	9		131					
132.2 8.2			11	SS	2		130					
	SILTY CLAY tr. sand, tr. gravel grey, firm to stiff, wet		12	TW	PM		129					
			13	SS	2		128					
			14	SS	3		127					
125.4	freq. sand zones						126					

Continued Next Page

3 x 3

Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

TRANETO001245AA HWY 401/Leslie St Ramp N-E

RECORD OF BOREHOLE No E2

2 OF 3

METRIC

GWP 2008-E-0012

LOCATION

N-E Ramp of Leslie Street to 401 (Northing 4647329.4 & Easting 315835.3)

ORIGINATED BY RK

DIST

HWY 401

BOREHOLE TYPE

Highway Stem Augers, DCPT

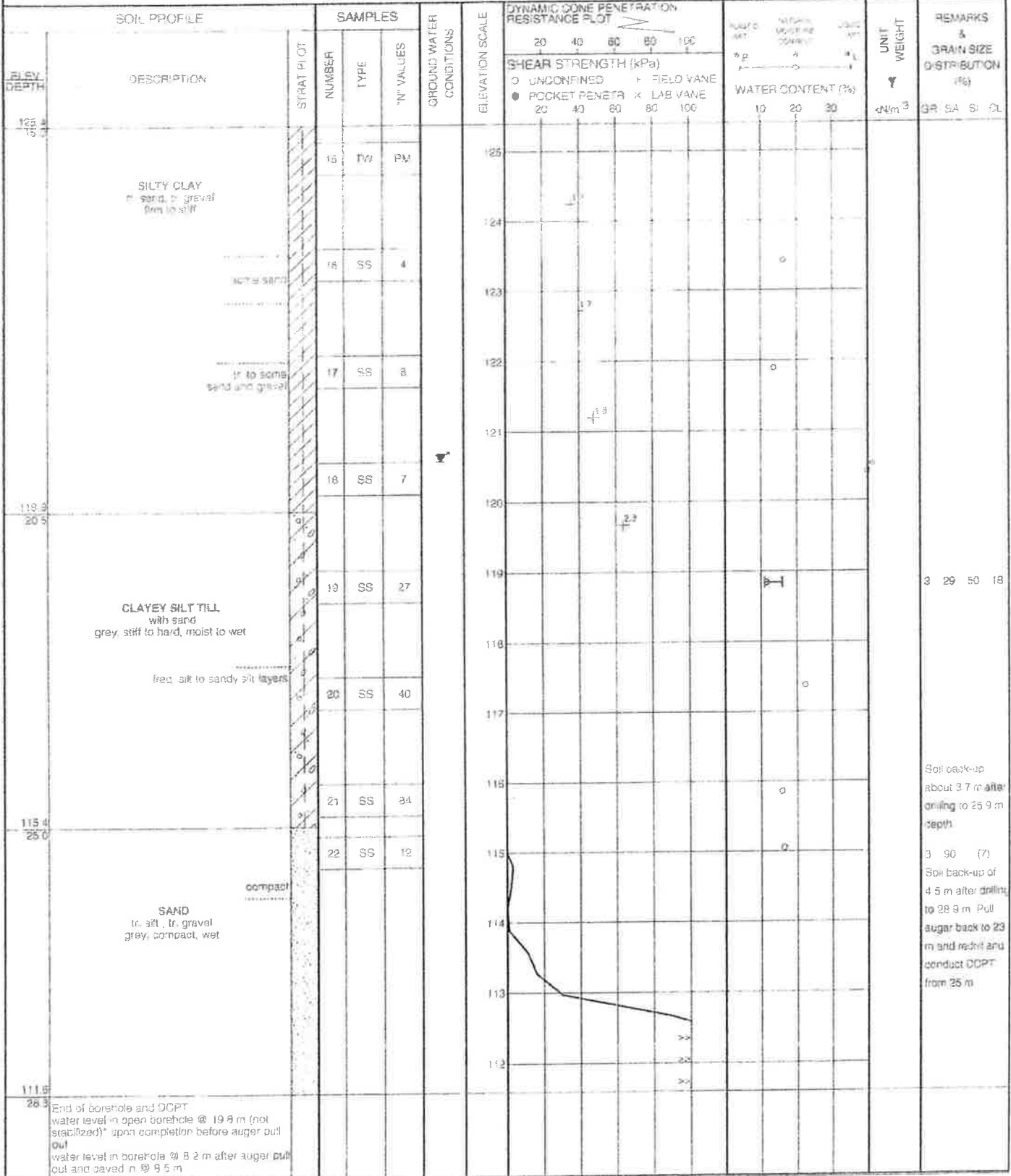
COMPILED BY SK

DATUM Geocoid

DATE

1/6/2010

CHECKED BY ZO



Continued Next Page

TRANSPORT 245AA HWY 401/ Leslie St Ramp N-E

RECORD OF BOREHOLE No E2

3 OF 3

METRIC

GWP 2008-E-0012 LOCATION N-E Ramp of Leslie Street to 401 (Northing 4847329.4 & Easting 315835.31) ORIGINATED BY RK
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers, DCPT COMPILED BY SK
DATUM Geodetic DATE 1/6/2010 CHECKED BY ZO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100			
110.4	Dynamic Cone Penetration Test (DCPT) performed from 25.4 to 28.8 m. After DCPT, borehole re-drilled to 28.8 m and soil backfill 2.5 m.													GR SA S CL

+ 1 x 3

Numbers refer to
Sensitivity

20
15
10

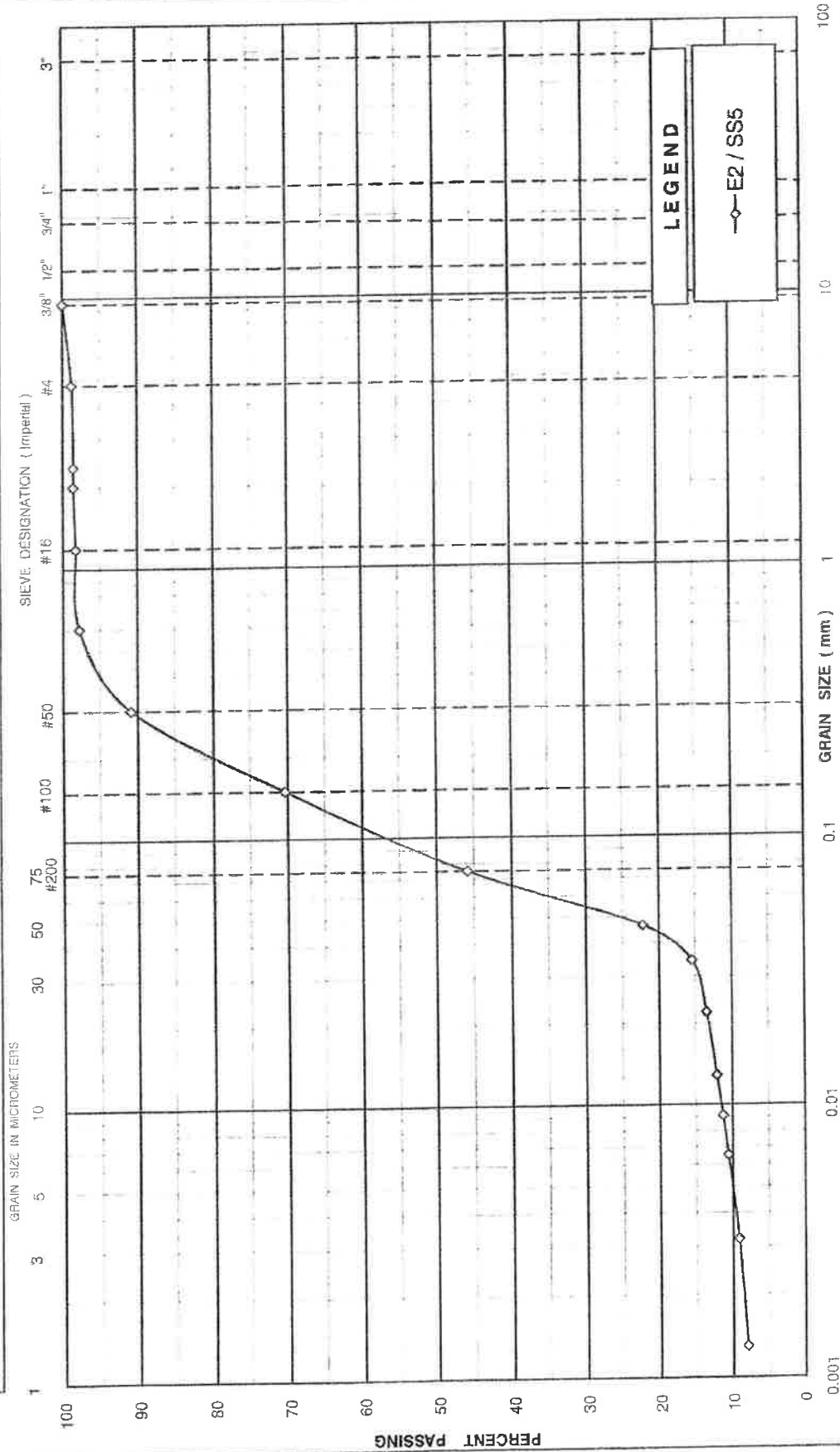
(%) STRAIN AT FAILURE

Appendix B

Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



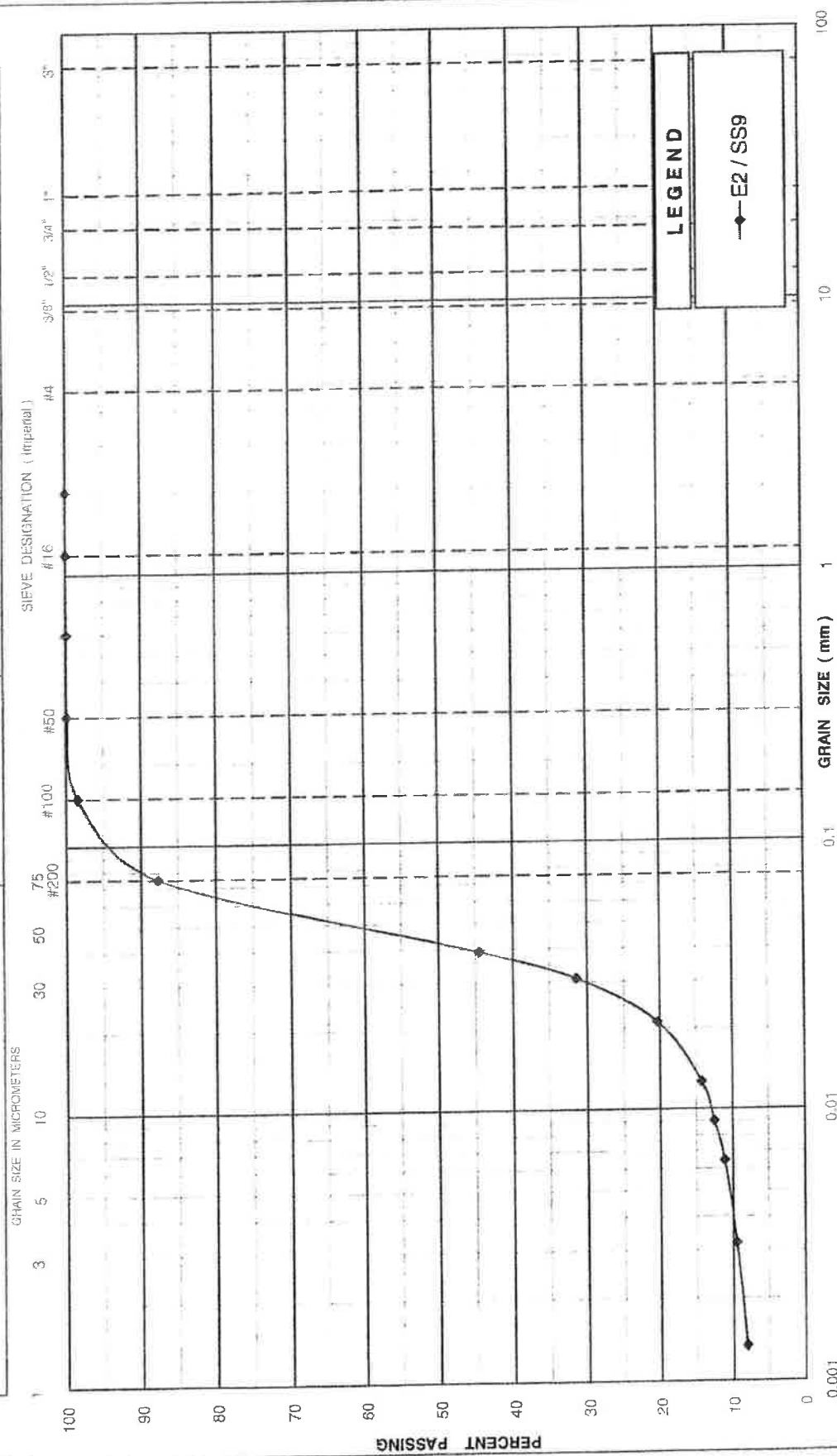


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
FILL: Silty Sand

FIGURE NO.: B-1
PROJECT NO: TRANETOB01245AA
DATE: March, 2010

CLAY AND SILT	SAND		GRAVEL	
	Fine	Medium	Fine	Coarse



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SPECIALISTS MANAGING THE EARTH

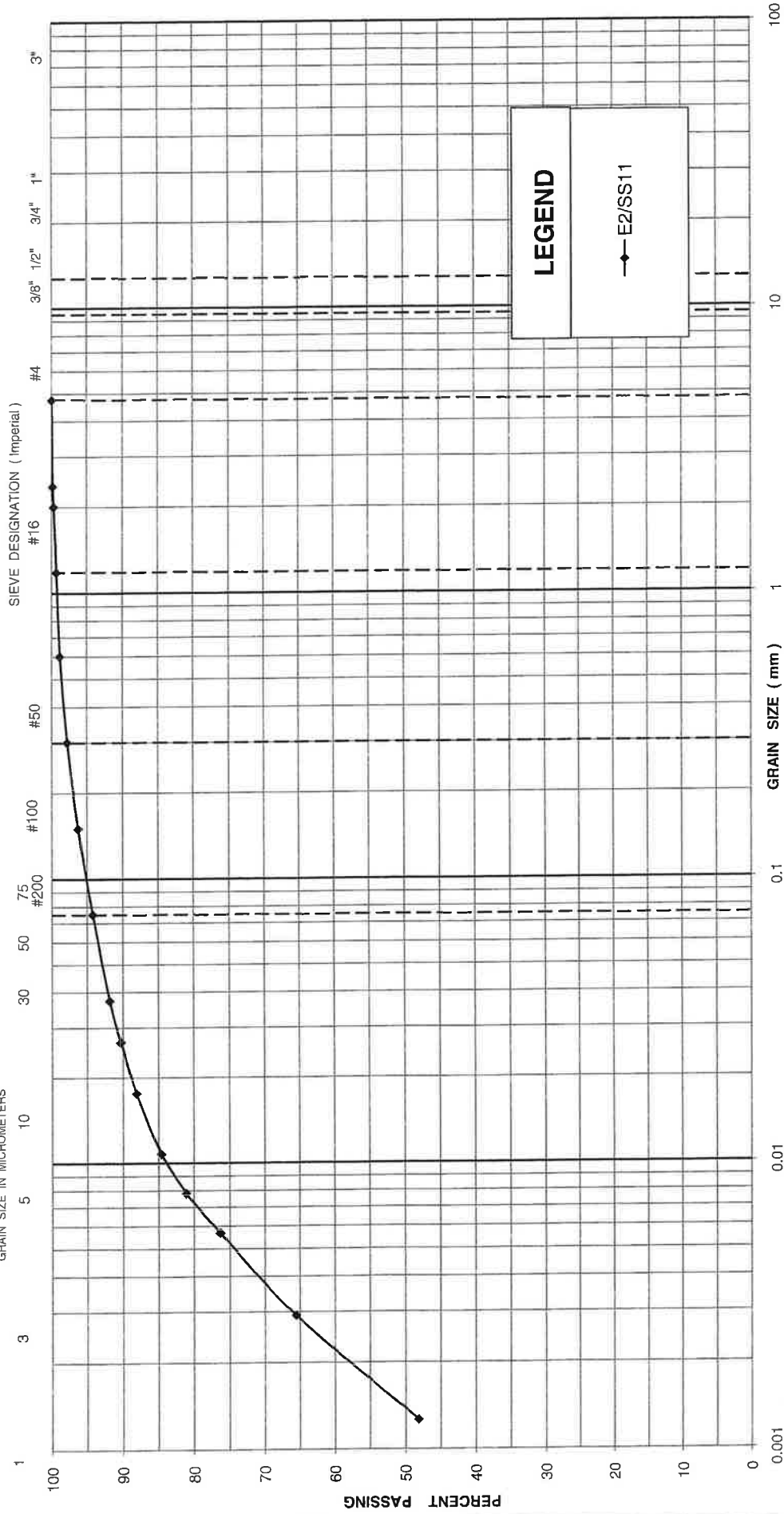
GRAIN SIZE DISTRIBUTION SILT

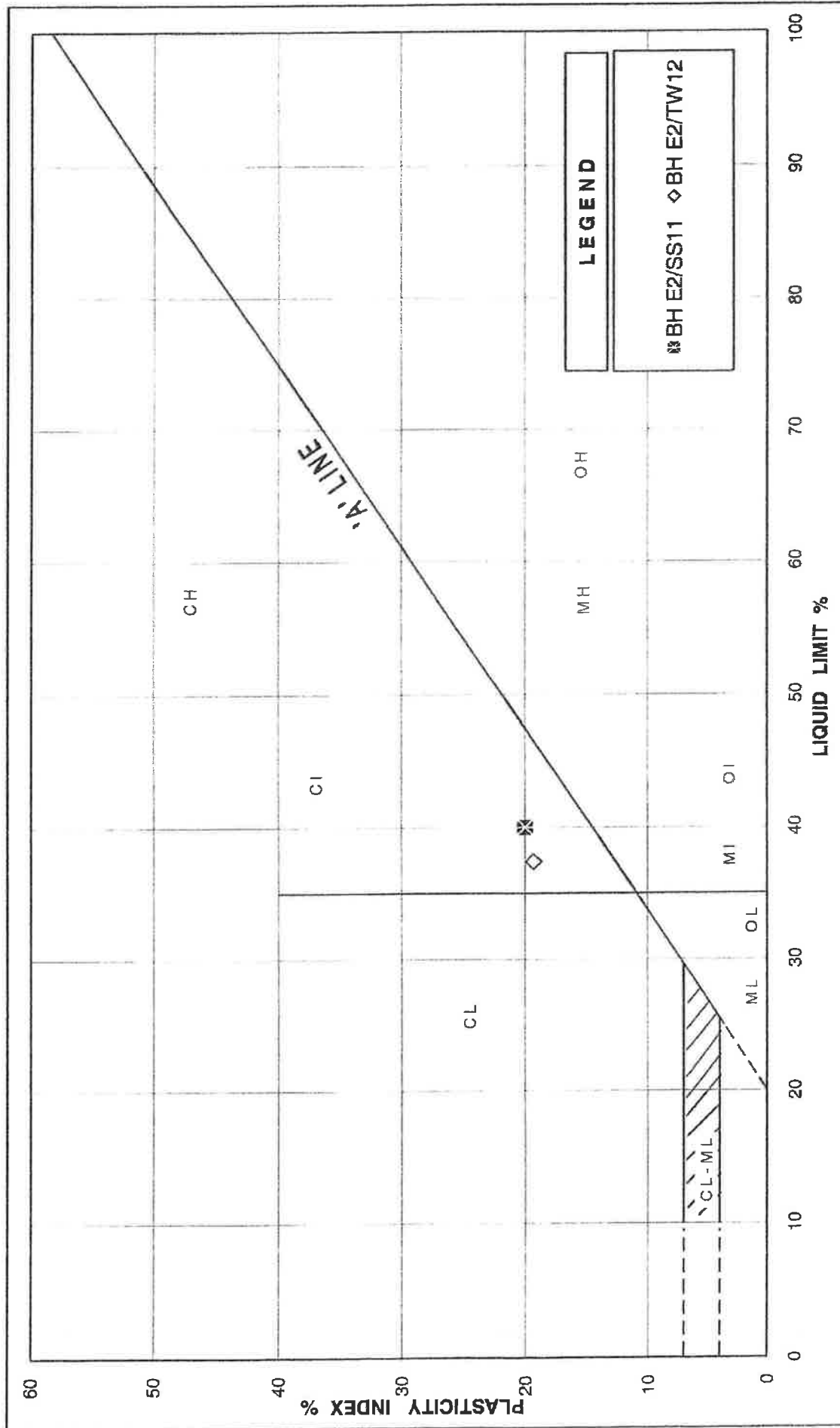
FIGURE NO.:	B-2
PROJECT NO.:	TRANETOBO1245AA
DATE:	March, 2010


UNIFIED SOIL CLASSIFICATION SYSTEM

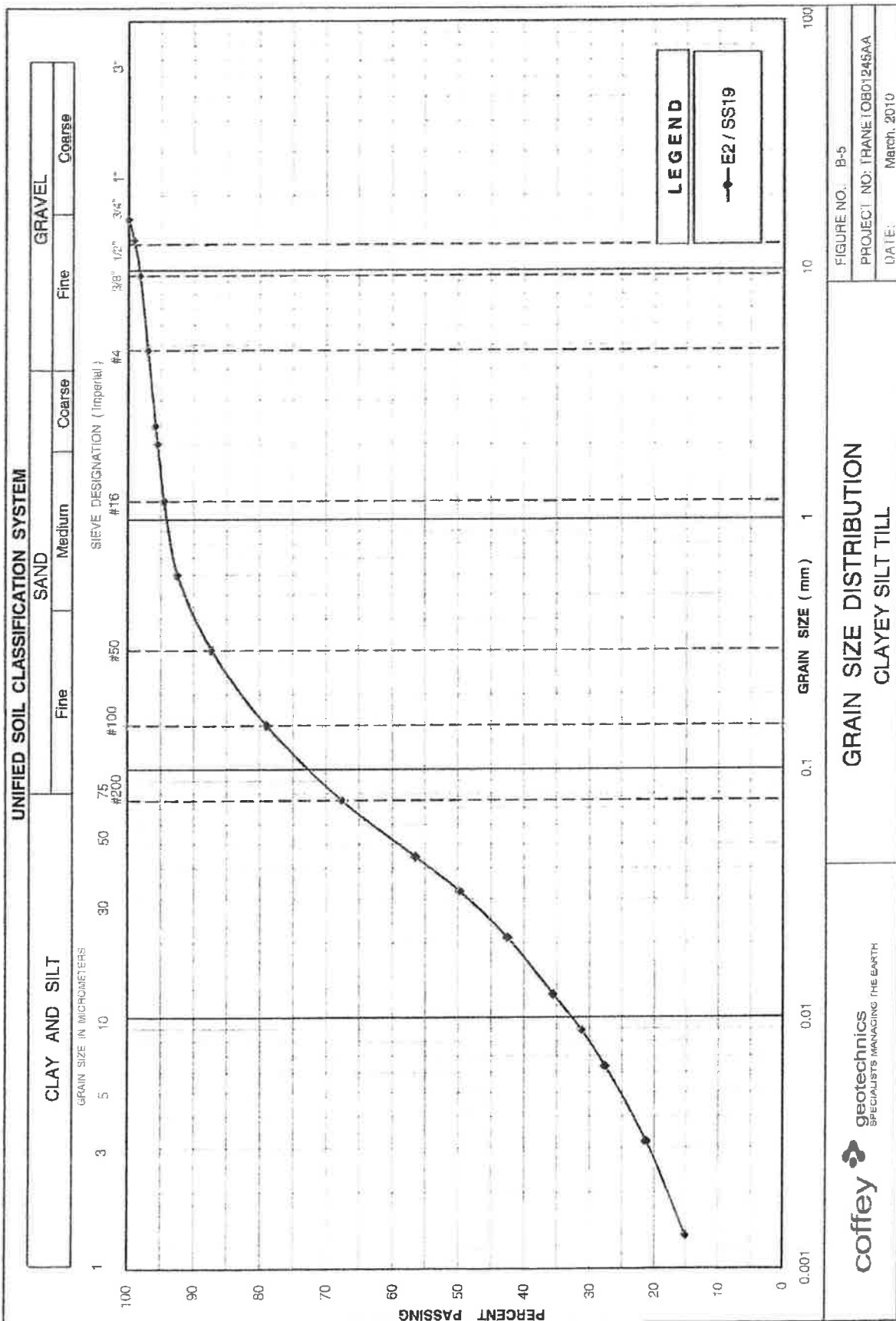
CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS





 SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART		FIGURE No. B-4
	SILTY CLAY		REF. No. TRANETOBO01245AA
			DATE March, 2010



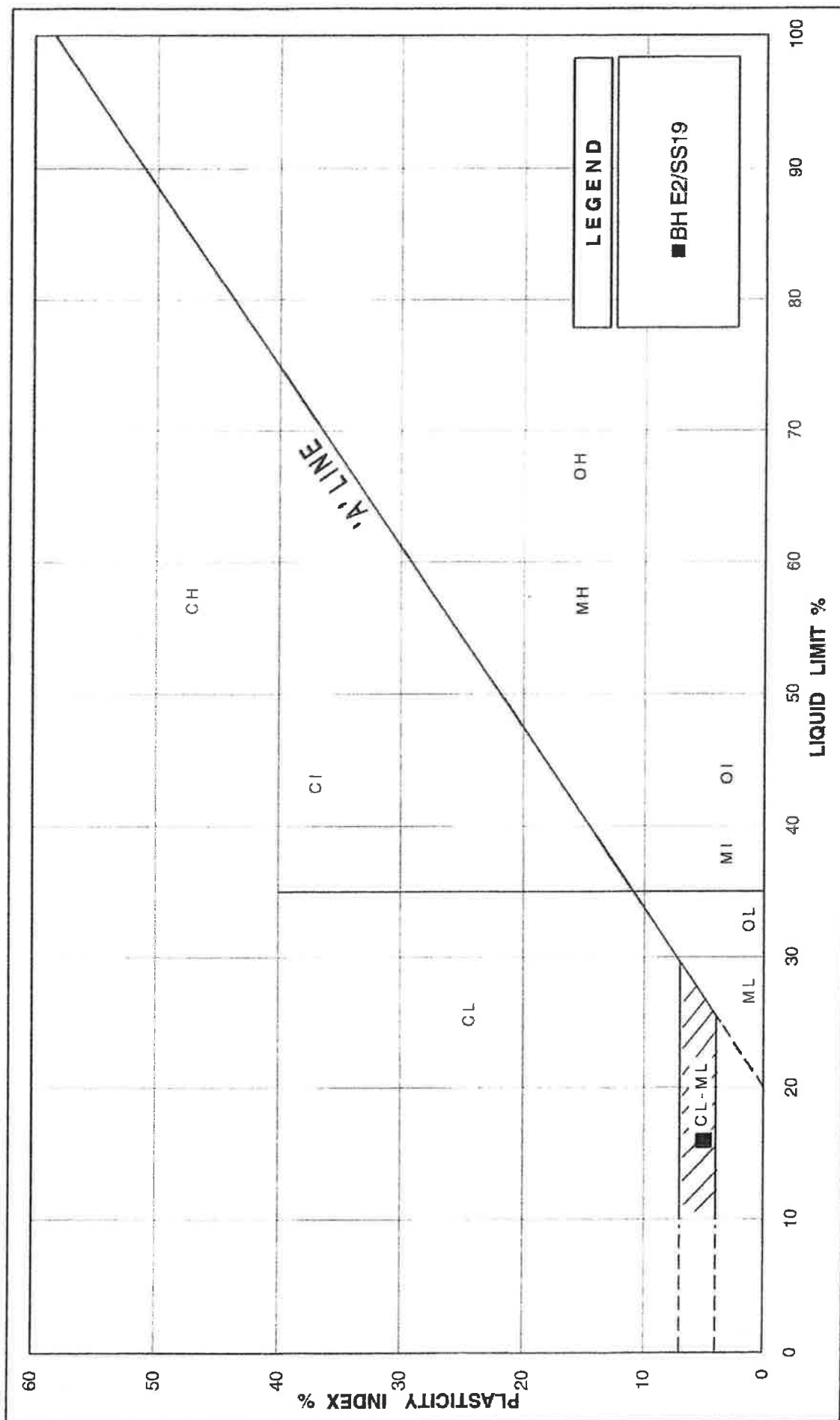


FIGURE No. B-6

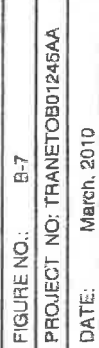
REF. No. TRANETOBO1245AA

DATE March, 2010

PLASTICITY CHART

CLAYEY SILT TILL

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION SAND

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SPECIALISTS MANAGING THE EARTH

Appendix C

**Stratigraphic Contacts - Highway 401 and Leslie Street Overpass Site
(Past Investigations)**

Table Leslie Street Overpass

References	Borehole Designation	Ground Surface (ft)	Top of Sand (ft)	Top of Clay (ft)	Top of Till (ft)	Top of Lower Sand (ft)	Water Table (ft)	Ground Surface (m)	Top of Sand (m)	Top of Clay (m)	Top of Till (m)	Top of Lower Sand (m)	Water Table (m)	Depth to Till (m)	Depth to Sand (m)
Department of Highways Ontario, 1964	Minimum	451.0	447.5	424.0	385.0	376.7	421.7	137.5	136.4	129.2	117.3	114.8	128.5	19.8	25.5
	Maximum	475.6	470.5	435.5	392.0	380.4	448.0	145.0	143.1	132.5	119.8	115.9	138.6	26.2	29.4
	Average	463.3	459.0	429.8	388.5	378.6	434.9	141.3	139.8	130.8	118.6	115.4	133.6	23.0	27.5
	B7	458.0	457.5	424.0	391.0	N/A	448.9	139.6	139.4	129.2	119.2		136.8	20.4	
	B8	471.0	470.5	426.0	385.0	N/A	445.2	143.6	143.4	129.8	117.3		135.7	26.2	
Dominion Soil Investigation Limited, 1967	B15	470.0	470.0	431.5	389.0	N/A	429.5	143.3	143.3	131.5	118.6		130.9	24.7	
	B16	451.0	450.5	429.0	386.2		421.7	137.5	137.3	130.8	117.7		128.5	19.8	
	1A	463.9	463.9	435.9	392.9	380.4		141.4	141.4	132.9	119.8	115.9		21.6	25.5
	2A	464.7	447.5	432.2	390.2	376.7		141.6	136.4	131.7	118.9	114.8		22.7	26.8
	3A	475.6	457.1	432.1	390.0			145.0	139.3	131.7	118.9	115.5		26.1	29.4

Fill or Clayey Silt, Organic (m)	Sand (m)	Clay (m)	Till (m)	Fill or Clayey Silt, Organic (ft)	Sand (ft)	Clay (ft)	Till (ft)
0.2	4.7	10.1	3.3	0.5	15.3	33.0	10.9
5.6	13.6	13.1	4.1	18.5	44.5	43.0	13.5
	5.0						
0.2	10.2	10.1		0.5	33.5	33.0	
0.2	13.6	12.5		0.5	44.5	41.0	
N/A	11.7	13.0		N/A	38.5	42.5	
0.2	6.6	13.0		0.5	21.5	42.8	
N/A	8.5	13.1	3.8	N/A	28.0	43.0	12.5
5.2	4.7	12.8	4.1	17.2	15.3	42.0	13.5
5.6	7.6	12.8	3.3	18.5	25.0	42.1	10.9

Appendix D

Site Photographs



Photograph 1. Highway 401 Leslie Street Overpass (looking north)



Photograph 2. Highway 401 Leslie Street Overpass (looking south)

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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1%	VOID RATIO	e_{min}	1%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ'	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ_r	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1%	VOID RATIO IN LOOSEST STATE	j	KN/m^2	SEEPAGE FORCE
γ_r	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**PRELIMINARY FOUNDATION DESIGN
REPORT, LESLIE STREET OVERPASS
STRUCTURE, HIGHWAY 401
REHABILITATION FROM LESLIE STREET TO
WARDEN AVENUE, MTO CENTRAL REGION,
G.W.P. 2130-01-00, GEOCRE 30M14-332**

Delcan Corporation
Project: TRANETOB01245AA-AD
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
LESLIE STREET 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

Coffey conducted a preliminary foundation investigation on this basis.

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 for the original scope of the project, 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. Foundation investigation recommendations for detail design are included in Section 5. This report provides preliminary foundation recommendations for the proposed Leslie Street overpass structure.

Based on the Drawing (See General Arrangement Drawing in Appendix F) provided to us by Delcan, the existing Leslie Street overpass structure, which carries the Highway 401 traffic over the existing Leslie Street, will be replaced with a four segment, two span (two cells) rigid frame structure. Total length (perpendicular to centreline of Highway 401) of this proposed Leslie Street overpass structure will be about 103 m. Each two span rigid frame will be about 40 m wide and 8 m high (inner cross-sectional dimension), each segment with a variable length.

It is our understanding that the proposed replacement will be carried out in stages to accommodate the Highway 401 traffic. Further details of construction staging will be developed in detail design phase.

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. An excess upward hydrostatic pressure appears to exist in the water bearing granular deposits underlying the glacial till. The previous and present investigations indicate similar overall subsurface conditions at the site. As various construction activities have taken place over the past many decades at the site, it is difficult to estimate embankment fill heights at the site. However, based on the existing topography and assuming that the original ground level was at about El. 140 m (460 ft, based on the available contract drawings), the embankment fill heights at the Leslie Street overpass structure site are estimated to be about 3 to 4 m.

4.1 Foundations

We understand that the newly proposed overpass structure will be constructed essentially at the same location as the existing overpass structures. Interference between the 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(s) will have similar minimum clearance height (about 8 m) 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 one side (either north or south) and the remaining Highway 401 structures (lanes) will be retained for the Highway 401 traffic during the construction. After the construction of one segment, construction will move to the other segments (towards the other end) and the newly built structure(s) and the remaining Highway 401 structures will support the Highway 401 traffic.

Based on the available MTO GEOCREST information, the existing Leslie Street overpass structure foundations under the Highway 401 are supported on driven steel H-pile (12 BP 53 equivalent to HP 310X79) foundations (including battered piles at abutment locations near the Leslie Street). The available contract drawings indicate that the foundations were designed to be extended into the dense to very dense till deposit (Department of Highway, Province of Ontario, Contract No. 65-205, Contract Drawings, dated 1965, Book 4 of 5). Battered piles from the existing structure (typically 3V:1H to 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 past and recent investigations, we have considered a number of foundation options as follows:

- *Shallow foundations*
- *Drilled caisson foundations*
- *Driven steel H pile foundations*
- *Driven steel tube pile foundations*
- *Micropile foundations*
- *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 foundation options. The foundation details and technical options discussed below should be further studied during detail design and detailed foundation recommendations will be made at that time.

4.1.1 Spread Footing Foundations

Based on the prevailing subsurface conditions, the use of spread footings is not considered a feasible option 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.

It is therefore recommended that a deep foundation option be adopted for this project.

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 Leslie Street overpass structure. It is noted that some of the existing Highway 401 structures (i.e. GO parking) in the vicinity of the proposed Leslie Street 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. To implement this option, the existing structure needs to be removed prior to the installation of the caissons, with a staging plan.

Based on the local design practice, caissons are socketed into the very dense/ hard till ($N > 50$ blows/0.3 m) a sufficient distance, whereby the caissons are designed for a combination of shaft friction/adhesion and end bearing resistance in competent till. For this particular project, for sake of simplicity, we recommend that the caissons be designed for an end bearing resistance of 2000 and 3000 kPa at SLS and ULS, respectively, for a minimum of 2.0 m embedment (socket) in the very dense / hard till, plus a friction/adhesion factor. If very dense to hard till thickness is found to be less than 2.0 m (i.e. Borehole E2), the above mentioned resistances would be revised. For preliminary design purposes, revised caisson capacities with 1 m penetration into the very dense to hard till option is also presented in Table 4.1.2.1. Less than 1 m penetration is not recommended. As well, due to the observed excessive upward hydrostatic pressure, more than 2 m penetration into the till may also be objectionable, unless the till below the base of the caisson is sufficiently thick. In summary, we recommend that with the available subsurface data the design be based on a minimum 1 m and a maximum 2 m penetration into the very dense/hard glacial till deposit, subject to possible revision in light of the findings of the detailed foundation investigation. It is therefore recommended that additional deeper boreholes with piezometric instrumentation be advanced in detail design phase to confirm groundwater and subsoil conditions at the proposed caisson locations.

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

Table 4.1.2.1 Recommended Caisson Resistances

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

* SLS for 25 mm settlement

These caisson sizes are recommended for efficiency in installation in consideration of the prevailing subsurface conditions. Higher caisson resistances would be available for greater penetration into the competent till (where the very dense or hard glacial till is relatively thicker) but with the presently available subsurface data, 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 clear distance between any two adjacent caissons is not less than 2.5 diameters centre to centre. It should be noted that the installation of new caissons may be difficult due to the presence of densely distributed existing piles and caissons supporting the existing structure at the site. Locations and sizes of new caissons need to be carefully selected considering the existing foundations (i.e. pile cap, battered pile, large diameter caisson etc).

Table 4.1.2.2 presents the anticipated caisson depth/elevation at Borehole E2.

Table 4.1.2.2 Anticipated Caisson Depth/Elevation

Borehole No.	Existing Ground Elevation (m)	Anticipated Caisson Depth (m)	Anticipated Caisson Bottom Elevation (m)	Anticipated Socket and Base Subgrade Type
E2	140.4	24.5	115.9	Clayey Silt Till*

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

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

The ULS geotechnical resistance (unfactored) to lateral loading can be calculated using passive earth pressure theory outlined in section C 6.8.7 of the *Commentary* to the CHBDC. For a single pile/caisson in non-cohesive soil, the passive resistance may be estimated by calculating passive earth pressure over an equivalent wall area having a depth from the ground surface equal to six times the pile/caisson diameter, and a width that is equivalent 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 equal 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, based on local experience.

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

Difficulties may be encountered during the installation of the caissons due to the presence of surficial granular overburden below groundwater table and anticipated cobbles and boulders in 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 and its foundations may be required for installation of new caissons where they interfere with the existing foundations. This aspect needs to be carefully investigated.

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

4.1.3 Driven Steel Piles

4.1.3.1 Steel H-Piles

From the geotechnical point of view, the boreholes show that the subsurface conditions at the site are suitable for the use of driven steel H-piles to support the proposed Leslie Street overpass structure(s). 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 vibrations induced by pile driving, this option may not be a favourable option for this project for environmental reasons, as the site is surrounded by residential areas and hospitals in the City of Toronto. The vibration monitoring programme should be carried out during pile driving. Special Provisions for vibration monitoring was included in Appendix I.

If piles are to be used, the existing immediate adjacent overpass structures will need to be removed prior to driving the piles. Steel H-piles (HP310 x 110) driven to practical refusal in the competent glacial till materials (at about El. 117 to 115 m) can be designed for 1700 kN/pile for factored U.L.S. and 1250 kN/pile for S.L.S (for 25 mm settlement). These values can be increased by 50 kN/pile for HP 310 x 125 piles, due to the increase in the steel cross-sectional area. Normally, somewhat higher resistances are available for the pile sizes recommended. However, in view of the upward hydrostatic gradients, possible artesian pressures and known past problems experienced at this interchange, the use of higher capacities is not recommended. The anticipated approximate pile tip (refusal) elevation at Borehole E2 is given in Table 4.1.3.1.1.

Table 4.1.3.1.1 Anticipated Pile Tip Depth/Elevation

Borehole No.	Existing Ground Elevation (m)	Anticipated Pile Tip Depth (m) below the existing ground	Anticipated Pile Tip Elevation (m)
E2	140.4	24.6	115.8*

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

Pile caps need to be installed below the frost depth and therefore the anticipated average pile length would typically be about 19 m based on the GA drawing provided to us by Delcan.

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.

Pile caps should have a minimum of 1.2 m permanent frost protection.

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

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

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

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

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

The removal of the existing structure and foundations may be required prior to driving piles where the existing structure and the existing foundations may interfere with the new foundations. This aspect needs to be carefully considered for this particular project.

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

4.1.3.2 Steel Tube Piles

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

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

Steel tube piles of 300 mm nominal diameter with a suitable steel thickness (e.g. 324 mm x 9.4 mm), driven at least 1 m to 2 m into the very dense/hard soil, can be expected to provide a Factored Axial Resistance at U.L.S. of 1050 kN and an Axial Resistance at S.L.S. equal to 700 kN at about the tip elevations (approximately 0.6 m higher) quoted for steel H-piles given in Section 4.1.3.1. The anticipated pile tip elevation at Borehole E2, advanced by Coffey for this project in 2010, is at El. 140.6 m or about 0.2 m higher than the anticipated elevation given for H-piles in Table 4.1.3.1.1.

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

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

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

4.1.4 Micropiles

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

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

Based on the prevailing subsurface conditions at the site, geotechnical resistance of a micropile primarily depends on the bond length within the competent glacial till and basal granular deposits and the type of micropile/installation method. For preliminary design purposes, axial resistances of up to about 500 kN/micropile may be available at ULS (for 260 mm diameter micropile) with a penetration not less than 6 m into the competent soil deposits (i.e. very dense soils). The lateral resistances would depend on the diameter and reinforcement of micropile and needs to be checked with a specialist contractor. But in the case of micropiles, the horizontal loads can easily be accommodated using inclined piles, provided there is no interference with existing foundations that will remain in use.

As mentioned before, the use of micropiles are likely to be less economical than other conventional deep foundations due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install conventional deep foundations. However, use of micropiles may shorten the construction period because micropiles can be installed under the existing structure without traffic disruption. As well, they may offer greater flexibility in choosing foundation locations to avoid existing foundations in congested areas, such as the present case.

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

4.1.5 Continuous Flight Auger Piles (CFA)

CFA piles are a type of drilled foundation in which the pile is drilled to the final depth in one continuous process using 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

From the geotechnical engineering point of view, the use of steel H-piles is the recommended option, for the prevailing subsurface conditions. However, considering environmental, issues related to vibrations generated by driven piles (including noise, damage to existing structures, earth slopes, etc.) caissons or micropile options may need to be resorted to. Micropiles have the advantage of being suitable for use in confined areas but is costlier of all the options discussed. Advantages, disadvantages, risk & consequences and relative costs of each foundation option are given in a tabular form in Appendix H.

4.2 Preliminary Recommendations for the Proposed Retaining Walls

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

Typical retaining wall options are as follows;

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

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

4.2.1 Conventional Cast-in-place Reinforced Concrete Retaining Walls

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

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

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

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

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

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

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

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

4.2.2 Retained soil system (RSS)

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

4.2.3 Retaining Wall Backfill

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

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

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

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

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

4.3 Lateral Earth Pressures

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

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

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

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

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$

$K_o = 0.43$

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$

$K_o = 0.47$

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the rigid frame structure is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of C.H.B.D.C. When selecting the parameters vibrations from the train traffic may also need to 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 bridges 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 when excavating to these depths, some dewatering will be required to provide a stable excavation base. Pumping from shallow filtered sumps along the perimeter ditches will probably suffice, if needed. However, type of the dewatering need to be decided after the details of construction are available.

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

OPSS 539 – Construction Specification for Temporary Protection Systems

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

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

Fill	Type 3 soil above water level
	Type 4 soil below water level

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

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

Table 4.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

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

It should be pointed out that the cobble size particles and even boulders can possibly occur within the overburden, as well possibly in the fill. If encountered, these can cause some problems during the installation of shoring units.

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

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

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

4.5 Frost Protection

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

5 RECOMMENDED SCOPE OF WORK REQUIRED FOR DETAILED FOUNDATION INVESTIGATION

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

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.

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
WP No



HIGHWAY 401
LESLIE STREET OVERPASS
GENERAL ARRANGEMENT

Delca

NORTH FOR CONSTRUCTION



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.

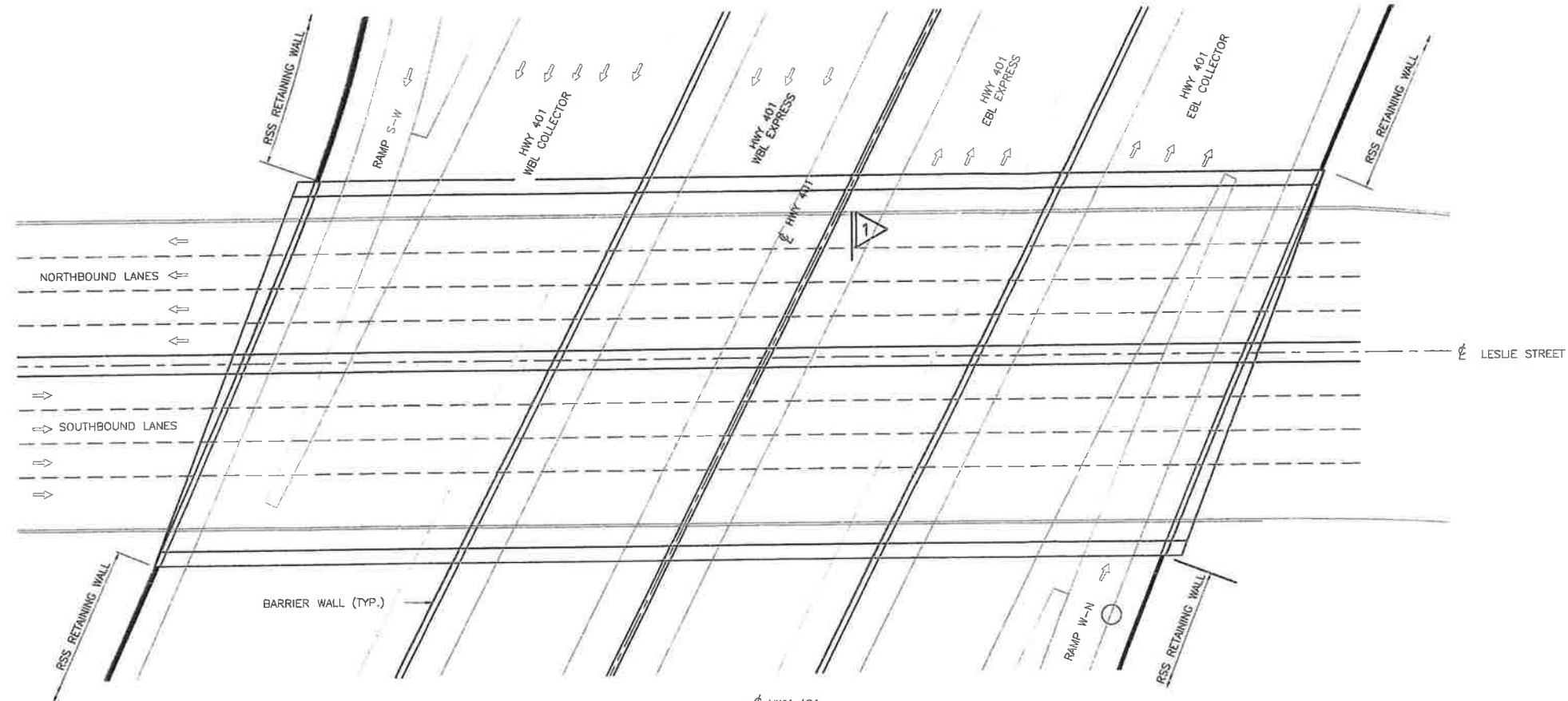
CONSTRUCTION NOTES

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.

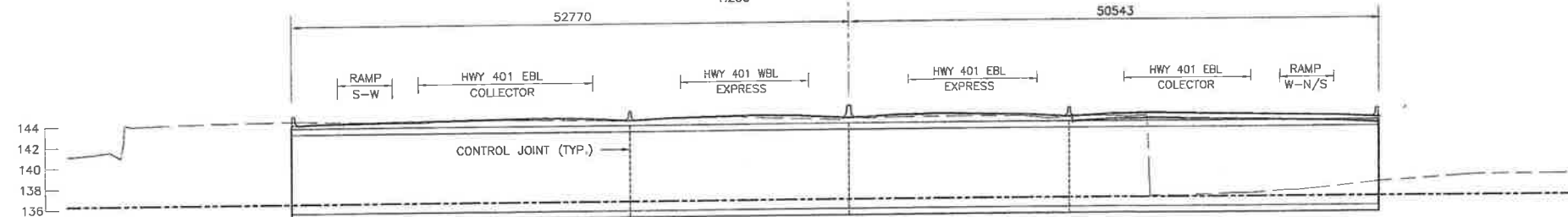
LIST OF ABBREVIATIONS:

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.



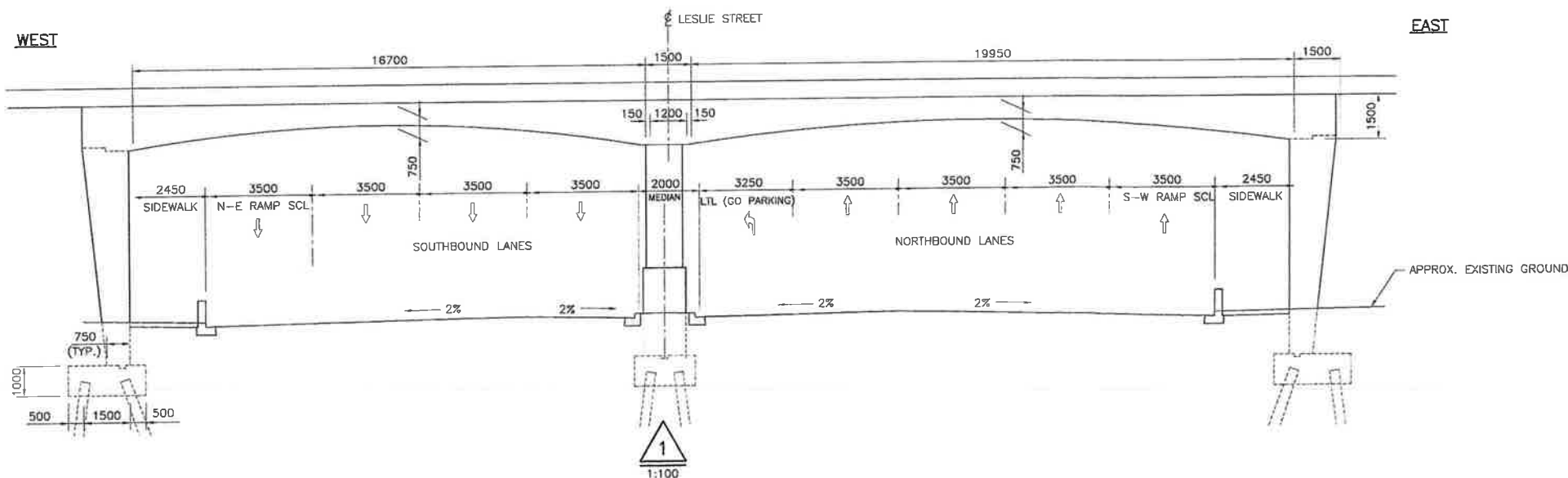
PLAN

1:250



ELEVATION

1:250



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DESCRIPTION	
1		DESIGN Y.F.O. CHK N.W. CODE CHBDC-06 LOADCL-625-0M DATE SEPT 2010	
2		DRAWN A.K.L. CHK Y.F.O. SITE	STRUCT SCHEME DWG P1

Appendix G

Foundation Elements - Highway 401 and Leslie Street Interchange

Table G-1
Foundation Elements - Highway 401 and Leslie Street Interchange
Project Number: TRANEOT0801245AA

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 as-built data Not Clear indicates data either not available or could not be read drawing. Note data based on interpretation of drawings and/or tables. No as-built construction records were located. Two piles went to 45 m depth during the installation	Notes
37-206/1	418	12BP53 H-Pile	150	*			45.7			1 to 3		
	418	12BP53 H-Pile	88	*			26.8					
	419	Concrete Caisson	n/a	n/a	36		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			n/a			1 to 6	design load 60 Ton/Pile	
	422	Concrete Caisson	n/a	n/a	36						working load 250 Ton/Caisson	
	423	Concrete Caisson	n/a	n/a	36						working load 250 Ton/Caisson	
	424	Concrete Caisson#1	81	n/a	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	
	425	12BP53 H-Pile	82		30	42	25.0	762	1067		working load 250 Ton/Caisson	
	425	Concrete Caisson#1	76		30	42	23.2	762	1067		working load 250 Ton/Caisson	
	425	Concrete Caisson#2	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	
	427	12BP53 H-Pile	77		30	42	23.5	762	1067		working load 250 Ton/Caisson	
	427	Concrete Caisson#1	77		30	42	23.5	762	1067		working load 250 Ton/Caisson	
	427	Concrete Caisson#2	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	
	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	21.3	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	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 6		

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)	(m)		(in)	(in)	(ft)	(mm)	(mm)		As-Built	As-Built	
37-206/2 Hwy 401 overpass at Leslie St. WB/Colliders (33 spans)	101	12BPS3 H-Pile	54	16.5	*	30	36	16.5	762	914	1 to 3			working load 250 Ton/caisson
	102	Concrete Caisson	50	15.2	*	30	36	15.2	762	914				working load 250 Ton/caisson
	103	Concrete Caisson	50	15.2	*	30	36	15.2	762	914				working load 250 Ton/caisson
	104	Concrete Caisson	51	15.5	*	30	36	15.5	762	914				working load 250 Ton/caisson
	105	Concrete Caisson	52.5	16.0	*	30	36	16.0	762	914				working load 250 Ton/caisson
	106	Concrete Caisson	63	19.2	*	30	36	19.2	762	914				working load 250 Ton/caisson
	107	Concrete Caisson	69	21.0	*	30	36	21.0	762	914				working load 250 Ton/caisson
	108	Concrete Caisson	74	22.6	*	30	36	22.6	762	914				working load 250 Ton/caisson
	109	Concrete Caisson	72	21.9	*	30	36	21.9	762	914				working load 250 Ton/caisson
	110	Concrete Caisson	67	20.4	*	30	36	20.4	762	914				working load 250 Ton/caisson
	111	Concrete Caisson	87	26.5	*	30	36	26.5	762	914				working load 250 Ton/caisson
	112	Concrete Caisson	80.5	24.6	*	30	36	24.6	762	914				working load 250 Ton/caisson
	113	Concrete Caisson	70.5	21.3	*	30	36	21.3	762	914				working load 250 Ton/caisson
	114	Concrete Caisson	70.5	21.3	*	30	36	21.3	762	914				working load 250 Ton/caisson
	115	Concrete Caisson	72	21.9	*	30	36	21.9	762	914				working load 250 Ton/caisson
	116	Concrete Caisson	74	22.6	*	30	36	22.6	762	914				working load 250 Ton/caisson
	117	Concrete Caisson	74	22.6	*	30	36	22.6	762	914				working load 250 Ton/caisson
	118	Concrete Caisson	80	24.4	*	30	36	24.4	762	914				working load 250 Ton/caisson
	119	Concrete Caisson	n/a	n/a	n/a									design load 60 Ton/pile
	120	12BPS3 H-Pile	n/a	n/a	n/a									design load 60 Ton/pile
	121	12BPS3 H-Pile	n/a	n/a	n/a									working load 250 Ton/caisson
	122	Concrete Caisson	76	23.2	*	30	36	23.2	762	914				settlement problem, caisson replaced with 6 new tube piles of 220 mm diameter, also replaced the Pier
	123	Concrete Caisson	76	23.2	*	30	36	23.2	762	914				working load 250 Ton/caisson
	124	Concrete Caisson	76	23.2	*	30	36	23.2	762	914				working load 250 Ton/caisson
	125	Concrete Caisson	78	23.8	*	30	36	23.8	762	914				settlement problem, caisson replaced with 6 new tube piles of 220 mm diameter, also replaced the Pier
	126	Concrete Caisson	77	23.5	*	30	36	23.5	762	914				working load 250 Ton/caisson
	127	Concrete Caisson	78	23.8	*	30	36	23.8	762	914				working load 250 Ton/caisson
	128	Concrete Caisson	78	23.8	*	30	36	23.8	762	914				working load 250 Ton/caisson
	129	Concrete Caisson	78	23.8	*	30	36	23.8	762	914				working load 250 Ton/caisson
	130	Concrete Caisson	78	23.8	*	30	36	23.8	762	914				working load 250 Ton/caisson
	131	12BPS3 H-Pile	64	19.5	*			19.5			1 to 6			
	132	12BPS3 H-Pile	64	19.5	*			19.5			1 to 6			
	133	12BPS3 H-Pile	64	19.5	*			19.5			1 to 6			
	134	12BPS3 H-Pile	78	23.8	*			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 as-builts	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
37-206/3	318	12BP53 H-Pile	78	*	30	42	23.8	762	1067	not clear		
How 401 over pass at Leslie St.	319	Not Clear										
EBL Express	320	Not Clear										
(16 Spans)	321	Not Clear										
	322	Not Clear										
	323	Not Clear										
	324	12BP53 H-Pile	78	*	30	42	23.8	762	1067	not clear		
	325	Concrete Caisson#1	85		30	42	25.9	762	1067		Previously recognized as caisson, H pile design load 60 Ton	
	326	Concrete Caisson#2	96		30	42	29.9	762	1067		working load 250 Ton/caisson	
	327	Concrete Caisson#3	98		30	42	29.9	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	81		30	42	24.7	762	1067		working load 250 Ton/caisson	
	330	Concrete Caisson#3	81		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	28.0	762	1067		working load 250 Ton/caisson	
	333	Concrete Caisson#3	92		30	42	28.0	762	1067		working load 250 Ton/caisson	
	334	Concrete Caisson#1	79		30	42	24.1	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	23.5	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	91		30	42	27.7	762	1067		working load 250 Ton/caisson	
	342	Concrete Caisson#3	91		30	42	27.7	762	1067		working load 250 Ton/caisson	
	343	12BP53 H-Pile	64	*			19.5			1 to 6		
	344	12BP53 H-Pile	64	*			19.5			1 to 6		
	345	12BP53 H-Pile	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	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-builts	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. Encumbrance failure north of the bent pile up at surficial sand deposit	Notes
37-206/4	218	Not clear										
Hwy 401 over pass at Leslie St.	219	12BP53 H-Pile	Installed to surficial sand									
WBL Express	220	12BP53 H-Pile	Installed to v.dense till									
(16 Spans)	221	12BP53 H-Pile	Installed to v.dense till									
	222	12BP53 H-Pile	Installed to v.dense till									
	223	12BP53 H-Pile	Installed to v.dense till									
	224	12BP53 H-Pile	Installed to surficial sand									
	225	Concrete Caisson#1	84		30	42	75.6	762	1067	not clear	Previously recognized as caisson. H pile design load 60 Ton	
	226	Concrete Caisson#2	84		30	42	75.6	762	1067		working load 250 Ton/caisson	
	227	Concrete Caisson#3	84		30	42	75.6	762	1067		working load 250 Ton/caisson	
	228	Concrete Caisson#1	78		30	42	73.8	762	1067		working load 250 Ton/caisson	
	229	Concrete Caisson#2	78		30	42	73.8	762	1067		working load 250 Ton/caisson	
	230	Concrete Caisson#3	82		30	42	75.0	762	1067		working load 250 Ton/caisson	
	231	Concrete Caisson#1	77		30	42	73.5	762	1067		working load 250 Ton/caisson	
	232	Concrete Caisson#2	77		30	42	73.5	762	1067		working load 250 Ton/caisson	
	233	Concrete Caisson#3	82		30	42	75.0	762	1067		working load 250 Ton/caisson	
	234	Concrete Caisson#1	74		30	42	72.6	762	1067		working load 250 Ton/caisson	
	235	Concrete Caisson#2	74		30	42	72.6	762	1067		working load 250 Ton/caisson	
	236	Concrete Caisson#3	79		30	42	74.1	762	1067		working load 250 Ton/caisson	
	237	Concrete Caisson#1	76		30	42	73.2	762	1067		working load 250 Ton/caisson	
	238	Concrete Caisson#2	76		30	42	73.2	762	1067		working load 250 Ton/caisson	
	239	Concrete Caisson#3	77		30	42	73.2	762	1067		working load 250 Ton/caisson	
	240	Concrete Caisson#1	73		30	42	72.8	762	1067		working load 250 Ton/caisson	
	241	Concrete Caisson#2	73		30	42	72.8	762	1067		working load 250 Ton/caisson	
	242	Concrete Caisson#3	80		30	42	74.4	762	1067		working load 250 Ton/caisson	
	243	Concrete Caisson#1	84	*			19.5			1 to 6		
	244	Concrete Caisson#2	84	*			19.5			1 to 6		
	245	Concrete Caisson#3	84	*			19.5			1 to 6		
	246	Concrete Caisson#1	78	*			23.8			1 to 3		

Table G-1
Project Number: Foundation Elements - Highway 401 and Leslie Street Interchange
TRANET0B01245AA

Structure No.	Bent No.	Foundation Type	Depth (ft)	* Estimated	Diameter (in)	Base Diameter ** Estimated based on table no as-builts	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
37-206/5 Hwy 401 over pass at Leslie St. RAMP W-N/S (8 Spans)	618	12BP53 H-Pile	50	*			27.4			1 to 3	working load 250 Ton/caisson	
	619	Concrete Caisson	85	*	30	42	25.9	762	1067	1 to 6	working load 250 Ton/caisson	
	620	12BP53 H-Pile	60	*			18.3			1 to 6		
	621	12BP53 H-Pile	56	*	30	42	17.1	762	1067	1 to 6	working load 250 Ton/caisson	
	622	Concrete Caisson	77	*	30	42	23.5	762	1067	1 to 3	working load 250 Ton/caisson	
	623	Concrete Caisson	77	*	30	42	23.5	762	1067	1 to 3	working load 250 Ton/caisson	
37-206/6 Hwy 401 over pass at Leslie St. RAMP N-E (2 Spans)	624	12BP53 H-Pile	88	*			26.8			1 to 3		
	728	12BP53 H-Pile	84	*			25.6			1 to 3	working load 250 Ton/caisson	
	729	Concrete Caisson#1	75		30	42	22.9	762	1067		working load 250 Ton/caisson	
	729	Concrete Caisson#2	75		30	42	22.9	762	1067		working load 250 Ton/caisson	
37-206/7 Hwy 401 over pass at Leslie St. RAMP N-W (6 Spans)	518	Concrete Caisson	74	*	30	36	22.6	762	914		working load 250 Ton/caisson	
	519	Concrete Caisson	72.5	*	30	36	22.1	762	914	1 to 6	working load 250 Ton/caisson	
	520	12BP53 H-Pile								1 to 6		
	521	12BP53 H-Pile								1 to 6		
	522	Concrete Caisson	77	*	30	36	23.5	762	914		working load 250 Ton/caisson	
	523	Concrete Caisson	88	*			26.8				working load 250 Ton/caisson	

Appendix H

Evaluation of Foundation Alternatives

Table H-1. Foundation Options for Leslie Street. Overpass Structure

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

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

Appendix 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

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

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

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

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

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

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

CAISSON PILES - Item No.

Special Provision

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

903.07.03 Caisson Piles

903.07.03.01 General

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

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

The Contractor should also note that there is a possibility of the presence of cobbles and boulders in overburden in the area where caisson piles are to be installed. If these obstacles are encountered, the Contractor shall employ the necessary measures to comply with the requirements of OPSS 903.

903.10 BASIS FOR PAYMENT

903.10.02 Caisson Piles - Item

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

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

Appendix J

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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