coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORTS EXISTING VIADUCT WEST OF HIGHWAY 401 AND LESLIE STREET, HIGHWAY 401 REHABILITATION FROM LESLIE STREET TO WARDEN AVENUE, MTO CENTRAL REGION, G.W.P. 2130-01-00, GEOCRES 30M14-335

Delcan Corporation
Project: TRANETOB01245AA-AE
December 19, 2011

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

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SPECIALISTS MANAGING THE EARTH

December 19, 2011

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

Attention: Ms. Draga Daniel, P.Eng.

Dear Madam:

RE: **Preliminary Foundation Investigation and Design Report**
Existing Viaduct West of Highway 401 and Leslie Street, Highway 401 Rehabilitation from Leslie Street to Warden Avenue, MTO Central Region, G.W.P. 2130-01-00

Please find attached our preliminary foundation investigation 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

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1 INTRODUCTION

As part of the proposed rehabilitation of Highway 401 from Leslie Street to Warden Avenue, three highway ramp structures were originally identified within the scope of Foundation Engineering, for field investigation and preliminary foundation recommendations. On that basis, Coffey Geotechnics Inc. (Coffey) was retained by Delcan Corporation (Delcan) to carry out a preliminary foundation investigation at the site of the proposed rehabilitation of the following existing highway ramp structures.

Structure Name	MTO Structure Number
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 Foundation Engineering scope of work was changed in late 2010, based on the overall preliminary design recommendations. This happened after the completion of Coffey's foundation investigation program. The new scope was defined based on the following:

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

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

Based on the information provided to us by Delcan, the existing N-W ramp and the Highway 401 westbound Collector structures, carrying highway traffic entering from Leslie Street and Highway 401 collector traffic, will be replaced with a CNR overpass structure and embankments to the east and west of CNR, conditions permitting. This treatment will eliminate the existing viaduct and will place the the traffic on a raised embankment. Preliminary construction staging drawings of the proposed works (prepared by

Delcan), including a high permanent retaining wall on the north side, were provided to Coffey. It is our understanding that further details of the new embankment will be developed during detail design phase.

This preliminary foundation investigation report was prepared for the proposed new embankment at the viaduct (part of the existing Bridge Site #37-206/2).

2 SITE DESCRIPTION AND GEOLOGY

The site is located west of the existing CN track(s), as shown on Drawing 1. In general, the existing grade along Highway 401 falls from the west, from about El. 152 m, to the east to about El.144 m, above Leslie Street.

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

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

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

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

3 SUBSURFACE CONDITIONS

3.1 Past Reports

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

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

The purposes of this study were to assess the embankment failure which took place during its construction of the west approach of the core lanes and to provide remedial measure recommendations for the proposed embankments. Nineteen (19) explorations were advanced for this study (Designated G-series. on Drawing 1) and Boreholes G1, G2, G3, G6 and G7 were found to be advanced close to the newly proposed viaduct.

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

The purposes of this study were to assess if there had been strength gain in the underlying clay soils as a result of embankment loading and comment on if a reduction of the previously recommended berm requirements could be altered. Three (3) boreholes were advanced in proximity to the previously drilled boreholes (Designated 1, 2 and 3 on Drawing 1). All three (3) boreholes drilled for this project were advanced in the vicinity of the newly proposed viaduct.

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

The purpose of this study was to determine the depth to the underlying dense till layer in order to establish the lengths of piles to be used to support the proposed structures associated with the widening of the existing overpass. Eighteen (18) sampled boreholes and two dynamic cone penetration tests were performed (Designated B1, etc. on Drawing 1). Borehole B18 was drilled close to the newly proposed viaduct.

❖ **Department of Highways Ontario, Foundation 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). Only one borehole log (Borehole 211) was available for purposes of our data compilation. However, subsurface data were obtained by scaling from the cross section provided (see Drawing 3). Boreholes were designated 211, etc. on Drawing 1. All sixteen (16) boreholes drilled for this project were advanced in the vicinity of the newly proposed viaduct.

❖ **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 track(s), 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 were advanced (Designated 1-1, etc. on Drawing 1). All five boreholes drilled for this project were advanced in the vicinity of the newly proposed viaduct.

3.2 Compiled Subsurface Conditions Based on Previous Data

3.2.1 Background

In order to gain a better understanding of the subsurface conditions at the proposed embankment and retaining wall, west of the existing CN rail track, 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 C-1 in Appendix C). The stratigraphy was based on the borehole logs, drawings and descriptions provided in the various reports.

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

Based on the findings of past investigation, estimated subsurface profiles along the West Bound, Core and East Bound Collector Lanes are presented in Drawing 2. Borehole locations and soil strata drawing prepared by the Department of Highway, Ontario investigation (1965) are also included in this report (See Drawing 3).

The following assumptions were made for this project:

- Elevations were assumed to be based on the geodetic datum.
- The surface elevation was based on those indicated on the borehole logs and no correction was made; existing topography was unavailable.
- Depths noted below, for the various units, were based on measurements below the existing ground surface, at the time the explorations were completed, no correction was made.
- Imperial elevations were directly converted to metric and no correction factor was used.
- Locations of boreholes were approximated based on those indicated on the drawings provided in the referenced reports.
- Due to the age of some of the documents and the quality of the original scanning, some of the data were difficult to read. Where borehole locations and/or data could not be properly interpreted, these data were not plotted and/or not used.
- Depths and elevations noted below for most of 200-series boreholes (Borehole 201 through 210, Department of Highway Ontario, 1967) were obtained by scaling from the soil strata drawing which is available in MTO Geocres information system (See Drawing 3).

The following provides a compiled overview of the subsurface conditions encountered at the surrounding area of the existing viaduct location, based on a summary of the available 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 - Leslie Street interchange 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 viaduct location.

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
The Foundation Companies of Canada, 1953	G1	142.1	136.4	No
	G2	140.9	120.6	No
	G3	142.1	129.2	No
	G6	148.7	123.9	Piezometer Installed - no reading
	G7	140.5	123.3	No
Geocon Limited, 1960	1	144.9	123.1	No
	2	140.8	123.7	No
	3	144.8	130.5	No
DHO*, 1964	B18	140.7	120.4	No
DHO*, 1965	201	144.8	118.6	No
	202	144.7	122.2	No
	203	145.0	121.9	No
	204	145.6	110.3	No
	204A	145.6	111.6	No
	204B	145.6	124.1	No
	204C	145.6	123.4	No
	204D	145.6	113.4	No
	204E	145.6	115.8	No
	205	145.8	121.6	No
	206	146.6	124.4	No
	207	146.7	128.3	No
	208	147.8	129.2	No
	209	148.4	131.4	No
	210	145.7	121.9	No
	211	141.0	121.0	No

Company / Year of Investigation	Borehole No.	Existing Ground Surface Elevation (m)	Bottom Elevation of Boreholes (m)	Piezometer
Ministry of Transportation Ontario, 1990	1-1	150.8	123.1	No
	1-2	151.3	123.6	Piezometer installed to El. 139 m
	1-3	151.7	119.7	Piezometer installed to El. 136 m
	1-4	152.0	129.0	No
	1-5	152.3	133.7	No

*DHO=Department of Highway, Ontario

Table C-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, Pavement, Pavement Fill and Other Fills

Topsoil, topsoil and granular fill (The Foundation Companies Canada, 1953 and Geocon Limited, 1960), pavement structure and granular fill (Boreholes 1-1 through 1-5, Ministry of Transportation Ontario, 1990), and cohesive fill (Boreholes 201 through 210, Department of Highway Ontario, 1965) were generally encountered from the ground surface at each borehole location (except for Boreholes G3 and B18) to depths ranging from approximately 0.2 m to 11.6 m below the ground surface at the time of the explorations, or to elevations of approximately El. 148.3 m to 138.7 m. About 0.3 m thick buried topsoil was encountered at Borehole G6 immediately below the fill. 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 average thickness of the topsoil, pavement and fill materials encountered at the site was approximately 4.4 m, based on past explorations, extending to an average elevation of about El. 142 m.

In Boreholes 1-1 through 1-5 (Ministry of Transportation Ontario, 1990), an about 0.5 m thick pavement structure (asphalt pavement and granular road base material – sand some gravel, recorded N values from 24 to in excess of 100 blows/0.3 m, compact to very dense condition) was encountered at the existing grade.

The granular fill material at the site has been described as an indigenous till-derived fill (The Foundation Companies of Canada, 1953, recorded N-values from 22 to 52 blows/0.3 m, compact to very dense condition), fill (Geocon Limited, 1960, recorded N-values from 8 to 43 blows/0.3 m, loose to dense condition) and sandy silt fill with irregular clayey silt and sand (Ministry of Transportation Ontario, 1990, recorded N-values from 6 to 37 blows/0.3 m, loose to dense condition). A cohesive fill consisting of clayey silt with some sand and trace of fine gravel was also encountered in Department of Highway Ontario investigation (1965).

A grain size distribution curves (prepared by Foundation Companies, Canada) for a sample from the fill contacted in Borehole G6* is presented in Figure B-1 in Appendix B. Seven grain size analyses, carried out on samples of the sandy silt fill (Ministry of Transportation Ontario, 1990) are presented in an envelope form in Figure B-2 in Appendix B, in their original form. As also indicated on the Record of Borehole Sheets in Appendix A, the results are as follows;

Gravel:	0-3 %
Sand:	18-40 %
Silt:	41-50 %
Clay:	9-37 %

*These boreholes were renamed / renumbered by Coffey for this report, as detailed in Table 3.2.1.1

The results of Atterberg limits tests, performed by Ministry of Transportation (1990) on seven samples from the irregular layers of clayey silt within the sandy silt fill are given in Figure B-3 in Appendix B. This plasticity chart was revised from its original MTO chart (based on the Record of Borehole Sheets), as it contained an error. The tests show the following index values;

Liquid Limit:	16-22 %
Plastic Limit:	12-14 %
Plasticity Index:	2-10
Natural Moisture Content:	9 – 16 %

These results are characteristics of clayey soils of low plasticity. The measured natural moisture contents are generally near or below the measured plastic limits.

It should be noted that conditions and compositions of berm fill is not clear because no sampling and in-situ testing was done for 200 series boreholes, which were drilled mostly in the berm area.

3.2.3 Surficial Granular Soils

In investigations conducted in 1953, 1964, and 1965, underlying the topsoil and fill, and from the ground surface (Boreholes G3 and B18), a surficial granular soil, which consists of sand, fine sand, fine sand some gravel, clayey silt & sand, silt, and silty fine sand at approximate elevations of 143.3 m to 138.7 m was found. Boreholes drilled by the Ministry of Transportation Ontario (1990) and Boreholes 207, 208 and 209 of the 1965 investigation do not report the presence of this particular stratum. The thickness of the surficial granular soils encountered ranged between 0.3 and 5.8 m, with an average thickness of approximately 2.7 m.

Two grain size analyses were carried out on samples of the surficial granular soil layer from Borehole G1 and analyses results are also presented in Figure B-1 in Appendix B.

These surficial granular deposits can be considered granular (i.e. non-cohesive) soils.

SPT 'N'-values of 3 to 47 blows/0.3 m were recorded within these surficial granular soils, indicating a very loose to dense relative density. Typically the stratum was described as compact.

3.2.4 Cohesive Soils

A cohesive soil deposit was encountered, generally below the surfacial granular soils, at Elevations 148.3 to 135.1 m (Average El. 140 m). This deposit was found to be approximately 7.8 to 15.6 m thick (average 12.2 m). Three boreholes (Borehole G1, G3 and 3) were terminated within this deposit after 2.7 to 7.1 m penetration into the deposit (at El. 136.4 to 129.2 m).

The stratum was described as grey, stratified clay with silt layers and some gravel (The Foundation Companies of Canada, 1953), clay with layers of silt, sand and pebbles (Geocon Limited, 1960), silty clay some sand (Department of Highway Ontario, 1964), silty clay with occasional layers of sand and gravel (Department of Highway Ontario, 1965) and frequently varved clayey silt to silty clay with occasional silt, sand zones and boulders (Ministry of Transportation Ontario, 1990). In Ministry of Transportation Ontario investigation (1990), traces of organics were noted at the top portion of this cohesive deposit and sandy silt to silt layers were also encountered within this cohesive soil deposit at various depths. In some reports (The Foundation Companies Canada, 1953 and Geocon Limited, 1960), there was a distinction between an upper clay (with layers of silt and some gravel, 1953 investigation and with layers of silt, sand and pebbles, 1960 investigation) and a lower till like stratum. However, it is our opinion that this cohesive till layer can be combined with and categorized under one cohesive soil unit with the upper clay layer, from foundation engineering viewpoint, based on the strength parameters measured by in-situ and laboratory tests.

Three grain size analyses were carried out on representative samples from this cohesive soil layer from Boreholes G2 and G6 (The Foundation Companies Canada, 1953) and results are also presented in Figure B-1 in Appendix B.

Twenty grain size analyses were carried out on samples (including samples from granular soil zones within the cohesive soils) from this cohesive soil deposit for Ministry of Transportation Ontario investigation in 1990. Grain size distribution curves of Ministry of Transportation Ontario (1990) are presented in an envelope form in Figure B-4 in Appendix B. The results of the twenty grain size analyses indicate the following grain-size distribution*:

Gravel:	0 - 22 %
Sand:	7 - 64 %
Silt:	9 - 63 %
Clay:	5 - 67 %

*The grain size distribution percentages given above are based on Record of Borehole Sheet information and not on the grain size curve given in an envelope form in Figure B-4.

Atterberg Limits tests were performed by MTO in 1990 on thirty seven samples. Figure B-5 includes the records on nineteen tests conducted on basically cohesive samples (i.e. excluding eighteen tests on sample from non-cohesive zones).

Liquid Limit: 14 - 69% (average 15% for basically non-plastic samples and 33% for cohesive samples)

Plastic Limit: 9 - 26% (average 12% for basically non-plastic samples and 16% for cohesive samples)

Plasticity Index: 1 - 43 (average 3 for basically non-plastic samples and 17 for cohesive samples)

Natural Moisture Contents: 10-57 % (average natural moisture contents 13% for basically non-plastic samples and and 26% for cohesive samples)

**Silty layers were encountered within the clayey deposit in Geocon Limited investigation (1960)

The values that were obtained from the clayey (cohesive) samples are characteristic of a cohesive soil of low to high plasticity.

Measured N-values in these cohesive soils range from 1 to 39 blows/0.3m (average 12 blows/0.3 m), while field vane tests yielded undrained in-situ shear strengths of about 10 to 60 kPa. The results of field vane tests by the Foundation Companies Canada (1953) are included in Appendix B (see Figure B-6). Relatively higher N values of 5 to in excess of 100 blows/0.3 m (average over 20 blows/0.3 m) were measured within the basically non-plastic zones. These result indicate that consistency of cohesive soils can be described as a very soft to stiff (typically firm) while relative density of granular soil within the cohesive soil deposit can be described as loose to very dense (typically compact).

The results of undrained triaxial and unconfined compression laboratory tests, conducted by the Foundation Companies Canada, on samples from this stratum, are given in Figures B-7, B-8 and B-9 in Appendix B. In general, the measured values were lower than the vane results probably as a result of the inevitable disturbance of the tested samples. The measured bulk unit weight of the samples range from 17 to 21 kN/m^3.

The results of six oedometer (one dimensional consolidation) tests performed on samples obtained from this deposit from the boreholes previously drilled in the viaduct area (5 samples) and from the vicinity (1 sample) are presented in Figures B-10 and B-11 in Appendix B.

The tests indicate a compression index C_c of about 0.2 to 0.6 and rebound value C_r of 0.02 to 0.06. Estimated difference between possible pre-consolidation pressure and the existing overburden pressure was about 50 kPa (slightly over-consolidated, Geocon Limited, 1960). Average coefficient of consolidation (C_v) of about $1x10^{-3}$ cm^2/s was also measured in the past investigation (The Foundation Companies Canada, 1953). The measured bulk unit weights of samples from the deposit range from 17.5 to 19.2 kN/m^3.

Figure B-12 in Appendix B presents an envelope of the vane test results from the historic subsurface explorations conducted at the site.

Due to their mode of deposition, the presence of cobbles and boulders may be anticipated in the lower portion of this deposit (till-like stratum).

3.2.5 Glacial Till

Underlying the cohesive soil deposit, a glacial till was encountered in the boreholes at depths ranging from about 10.7 m to 25.3 m below the existing ground surface at the time of the explorations or at about elevations of 137.5 m to 122.4 m (average El. 128.0 m), except for Boreholes G1, G3 and 3 which were terminated within the overlying cohesive soil deposit. Immediately below the cohesive soil deposit (i.e. clayey silt), an about 3.7 m thick very loose (SPT N-value of 3 blows/0.3 m) sand layer was encountered in Borehole G6 at depth of 19.8 m or El. 128.9 m and was underlain by the glacial till. The till deposit was described as a sandy till (Geocon Limited, 1960), heterogeneous mixture of clayey sillt, sand and gravel (Department of Highway Ontario, 1964 and 1965), heterogeneous mixture of sandy silt to silty sand, occasional clayey silt zones (Ministry of Transportation Ontario, 1990). Occasional boulders were also noted within this layer. The Foundation Companies of Canada (1953) detail subsurface information about this layer is not legible.

The till is generally a heterogeneous mixture of sand, silt, clay and gravel. In some past investigations, the till was classified as a basically granular (i.e. non-cohesive) soil. But it also exhibited some apparent cohesion, due to its clay content, especially where the clay content was relatively high.

Six grain size analyses are available on samples from this glacial till. Four of these results (grain size distribution curves for Ministry of Transportation Ontario investigation, 1990) are presented in an envelope form (see Figure B-13 in Appendix B). The results shown on Figure B-13 indicate the following grain-size distribution:

Gravel:	1-10 %
Sand:	13-44 %
Silt & Clay size particles	46-80 %

Atterberg Limits tests performed on samples from the deposit (probably performed on samples exhibiting cohesion, i.e. cohesive zones) indicated the following index values (See Figure B-14, Appendix B):

Liquid Limit:	16-32 %
Plastic Limit:	11-14 %
Plasticity Index:	2-17
Natural Moisture Content:	7 – 16 %

The above values are characteristic of a cohesive soil of low to medium plasticity.

All boreholes were terminated within this till deposit after 0.3 to 16.5 m penetration into the till deposit (average 5.6 m).

SPT 'N'-values ranging from 25 to in excess of 100 blows/0.3m were recorded within the till indicating a compact to very dense condition for the granular till (typically very dense) and a hard consistency for the cohesive till.

Dynamic cone penetration tests refusals were also noted within this till layer in Boreholes G2, G6, G7 and 211.

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

3.2.6 Groundwater Conditions

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

Table 3.2.6.1: Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Water Level Measurement Depth (m)*	Water Level Measurement Elevation (m)*	Remarks
G1	142.1	1.7	140.4	-
G2	140.9	2.1	138.8	-
G3	142.1	2.2	139.9	-
G6	148.7	-	-	Piezometer Installed – not legible or no readings
G7	140.5	1.0	139.5	-
1	144.9	5.8	139.1	-
2	140.8			-
3	144.8	5.4	139.4	-
B18	140.7	2.0	138.7	-
201	144.8	8.6	136.2	-
202	144.7	5.1	139.6	-
203	145.0	-	-	-
204	145.6	6.0	139.6	-
204A	145.6	6.0	139.6	-
204B	145.6	-	-	-
204C	145.6	-	-	-
204D	145.6	-	-	-
204E	145.6	-	-	-
205	145.8	-	-	-
206	146.6	-	-	-
207	146.7	-	-	-
208	147.8	-	-	-
209	148.4	-	-	-
210	145.7	-	-	-
211	141.0	1.7	139.3	-
1-1	150.8	3.5	147.3	-
1-2	151.3	12.2	139.1	Piezometer installed to about El. 139 m
1-3	151.7	2.1	149.6	Piezometer installed to about El. 136 m
1-4	152.0	8.5	143.5	-
1-5	152.3	5.3	147.0	-

* may not be stabilized

As can be seen from the table, the observed groundwater levels ranged between elevations of approximately 136 and 149 m. In addition to the observed groundwater condition, a perched water condition could possibly be encountered at the site due to the accumulation of the surface water in the fill materials and in the underlying surficial granular soils, overlying the less permeable cohesive soil deposit, especially during rainy periods/spring thaw.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The groundwater may also be influenced 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.

3.3 Recent Investigation Procedures

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

Table 3.3.1: Borehole Locations and Drilling Depths

Borehole No.	Location (Coordinates)		Existing Ground Surface Elevation (m)	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
	Northing	Easting			
N4	315597.1	4847415.3	144.2	24.7	No
N5	315578.4	4847412.9	144.5	24.7	Yes
N7	315519.3	4847406.6	144.9	24.5	No

The locations of these three boreholes advanced by Coffey are shown on the Site and Borehole Location Plan, Drawing No.1.

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

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

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

A dynamic Cone Penetration Test (DCPT) was performed from the ground surface adjacent to Borehole N7. 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 stiffness / compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects which in some cases affect the SPT results.

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in Borehole N5 to enable groundwater level monitoring over a prolonged period of time without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

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

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content, grain size analyses 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 was mentioned, three (3) boreholes (Boreholes N4, N5 and N7) were advanced adjacent to the existing viaduct. These borehole were put down for a then proposed ramp structure (i.e. ramp N-W) rehabilitation.

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

In general, the sub-surface stratigraphy in the three boreholes comprises fill materials and surficial granular soil deposits overlying silty clay, which is, in turn, underlain by cohesive and granular glacial till deposits. Boreholes were terminated within the glacial till deposit after 3.9 to 6.5 m penetration into the till deposits.

Details of the sub-surface conditions encountered in Boreholes N4, N5 and N7 are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to amplify and complement these data.

3.4.1 Topsoil

All three boreholes contacted a 0.1 m thick topsoil layer at the ground surface.

3.4.2 Fill

The boreholes contacted a fill material extending to depths of 4.4 to 5.1 m below the ground surface or to El. 140.5 to 139.4 m.

The fill typically consists of silty sand to sandy silt, traces to some clay and traces of gravel. Buried concrete pieces were noted under the topsoil in Borehole N4. Near the bottom of this layer in all three boreholes, an about 0.5 m thick peaty topsoil was encountered. The fill is considered to be a typically granular (non-cohesive) material but also contains zones of cohesive soil. Traces of rootlets and organics were encountered in Borehole N5. This fill was probably placed when the embankment was widened in the 1990's to accommodate drilling of caisson foundations for the presently existing N-W ramp.

The grain-size distribution of a sample from a cohesive soil zone within the fill is given in Figure B-15, in Appendix B, with the following grain-size distribution: 2 % gravel, 34 % sand and 38 % silt and 26 % clay size particles.

The Atterberg limits test results, performed on two samples from a cohesive zone in the fill, encountered in Boreholes N4 and N5 are given in Figure B-16 in Appendix B. The test yielded the following index values:

Liquid Limit:	22-23 %
Plastic Limit:	14 %
Plasticity Index:	8-9
Natural moisture content:	11-12

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

Standard penetration tests performed in the basically granular fill portion yielded N-values of 8 to 37 blows/0.3 m. These results indicate that the relative density of the granular fill can be described as loose to dense but typically compact, while the typical consistency of cohesive zones in the fill can be described as stiff to very stiff (N-values from 9 to 20 blows/0.3 m). These results indicate that, at most locations, the fill has received some degree of compaction when it was first placed; however the quality of the material was not fully controlled. As well, it appears that the existing peaty topsoil was not stripped and was thereby mixed with the fill during its placement.

It should be noted that Coffey boreholes are drilled from the top of the widened portion of the berm placed in 1990. Therefore results of boreholes drilled may not be representative of embankment fill.

3.4.3 Surficial Granular Soils

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

3.4.3.1 Sand

Below the fill, Boreholes N4 and N7 contacted a 0.9 to 2.3 m thick surficial sand deposit at depths of 4.5 and 4.4 m below the ground surface or at El. 139.7 and 140.5 m, respectively. This sand layer was found to extend to depths of 6.8 and 5.3 m below the ground surface or to El. 137.4 and 139.6 m, respectively. The sand layer contains traces to some silt. Traces of rootlets were also encountered in Borehole N4 at the interface with the overlying fill to the sand

The grain size distribution of two samples from the sand deposit was determined in laboratory which showed 0-1 % gravel, 83-86 % sand and 14-16 % silt and clay size particles (see Figure B-17 in Appendix B). As can be seen from the grain-size distribution curves presented, the material has a uniform grain size distribution, primarily within the fine sand range, and is considered to have a mass coefficient of permeability (k) of the order of 5 x 10^{-3} cm/s. The sand is a granular (non-cohesive) soil type.

Standard Penetration tests performed in the deposit yielded N-values of 8 to 39 blows/0.3 m, indicating a loose to dense relative density.

3.4.3.2 Silty Sand to Sandy Silt

Below the fill in Borehole N5, a relatively finer granular soil (i.e. silty sand to sandy silt) was encountered at a depth of 5.1 m or El. 139.4 m and was found to extend to a depth of 8.3 m or El. 136.2 m. The silty sand to sandy silt deposit contains traces of clay in the lower zones of this deposit.

The grain size distribution of a sample from the silty sand to sandy silt deposit was determined in laboratory which showed 0% gravel, 54 % sand and 37 % silt and 9 % clay size particles (see Figure B-18 in Appendix B). Based on the grain size distribution curves, the deposit is considered to be somewhat less pervious than the sand materials discussed in Section 3.4.3.1. N-values recorded range from 14 to 38 blows/0.3 m which indicates a compact to dense condition.

3.4.3.3 Silt

Below the sand in Boreholes N4 and N7, a 1.5 to 1.6 m thick silt deposit was contacted at depths of 5.3 to 6.8 m or El. 139.6 to 137.4 m. The silt deposit contains some fine sand and traces to some clay size particles and exhibits a dilatant behaviour in the presence of water. It is considered a fine grained granular (i.e. non-cohesive) soil type.

The grain size distribution of a sample from the silt deposit was determined in laboratory which showed 0% gravel, 23 % sand and 66 % silt and 11 % clay size particles (see Figure B-19 in Appendix B). Based on these results, the deposit is considered to be somewhat finer and less pervious in comparison with the silty sand to sandy silt deposit.

Standard penetration test conducted in this surficial silt deposit gave N-values ranging from 17 to 24 blows/0.3 m which indicate compact relative density.

3.4.4 Silty Clay

All three boreholes contacted an 11.3 to 12.4 m thick silty clay deposit below the surficial granular soils deposit, at depths ranging from 6.8 to 8.4 m or at El. 138.2 to 135.8 m. The deposit was found to extend to depths of 18.0 to 20.8 m below the ground surface or to El. 126.9 to 123.4 m. This cohesive soil deposit frequently has a varved like or stratified structure and contains thin seams or interbeds of silty sand to sandy silt and sand. The presence of traces to some sand and gravel size particles is also noted within the deposit.

The grain size distribution of a sample from the silty clay deposit was determined in the laboratory which showed 4 % gravel, 24 % sand and 39 % silt and 33 % clay size particles (see Figure B-20 in Appendix B).

The result of Atterberg limits test, performed on two samples from the deposit, is presented in Figure B-21 in Appendix B. The test results yielded the following index values:

Liquid Limit:	32-38 %
Plastic Limit:	16-19 %
Plasticity Index:	16-19
Natural Moisture Content:	26-34 %

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

Grain size analysis and Atterberg limits test performed on a sample from a sandy zone within the silty clay and these results are presented in Figures B-22 and B-23 in Appendix B.

Standard penetration tests conducted in the silty clay deposit gave N-values which range from 1 to 9 blows/0.3 m. Undrained-shear strengths as measured by MTO "N" type field vane varied from 28 to in excess of 100 kPa indicating a firm to very stiff consistency. The variation of the undrained shear strength as measured by vane tests with elevation is given in Figure B-12. It should however be pointed out that some higher undrained shear strengths may have been obtained from the silty sand to sandy silt and clayey silt interbeds within the silty clay deposit.

3.4.5 Glacial Till

Underlying the silty clay, all boreholes contacted a glacial deposit consisting of a heterogeneous mixture of silty sand to sandy silt and clayey silt. Traces to some gravel were encountered within the glacial till deposit. As well, the presence of cobbles and boulders can be expected in the glacial till deposits, owing to their mode of deposition. Auger grinding was observed during the investigation at various depths within the till deposit giving evidence to this aspect. As mentioned, the composition of the deposit was found to range primarily from a basically cohesive deposit consisting of clayey silt till to sandy silt-silty sand till, as discussed in Sections 3.4.5.1 and 3.4.5.2, below.

3.4.5.1 Clayey Silt Till

Boreholes N4 and N7 contacted a glacial deposit consisting of clayey silt till underlying the silty clay at depths of 20.8 and 18.0 m or at El. 123.4 and 126.9 m, respectively. Thickness of the clayey silt till was found to be 2.6 m in Borehole N7. Borehole N4 was terminated within the clayey silt till after 3.9 m penetration into this layer.

The grain-size distribution of two samples from the clayey silt till deposit is presented in Figure B-24 in Appendix B. This indicates the following grain-size distribution:

Gravel:	1-3 %
Sand:	24-32 %

Silt:	45-48 %
Clay:	20-27 %

The results of Atterberg limits tests, performed on two samples from the clayey silt till, are given in Figure B-25 in Appendix B. The tests yielded the following index values;

Liquid Limit:	20-21 %
Plastic Limit:	13-14 %
Plasticity Index:	7
Natural Moisture Content:	10-12 %

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

SPT N-values of 38 to in excess of 100 blows/0.3 m were recorded. Based on these field test results, the clayey silt till can be described as having a hard consistency.

3.4.5.2 Sandy Silt to Silty Sand Till

Underlying the silty clay in Borehole N5 and the clayey silt till in Borehole N7, a coarser (i.e. basically granular) sandy silt to silty sand till was contacted, at depths of 20.0 and 20.6 m below the ground surface or at or El. 124.5 and 124.3 m, respectively. Boreholes N5 and N7 were terminated within this granular till layer after 4.7 and 3.9 m penetration into the layer, respectively. Cobbles and boulders were inferred within the deposit.

The grain size distribution of a sample from the sandy silt to silty sand till deposit was determined in laboratory which showed 9-15 % gravel, 34-50 % sand and 32-36 % silt and 9-15 % clay size particles (see Figure B-26 in Appendix B).

The results of an Atterberg limits test, performed on a sample from a zone exhibiting some apparent cohesion in the sandy silt till, show the following index values (see Figure B-27 in Appendix B);

Liquid Limit:	15 %
Plastic Limit:	11%
Plasticity Index:	4
Natural Moisture Content:	6 %

These results are close to a non-plastic soil.

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

Standard penetration tests performed in this basically granular (i.e. non-cohesive) till yielded N-values typically in excess of 100 blows/0.3 m. An N-value of 35 blows/0.3 m was recorded at the top of the deposit in Borehole N5. Based on the test results, the relative density of the glacial till can be generally described as very dense, with a dense zone in Borehole N5 near the surface of the deposit.

3.4.6 Groundwater Conditions

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

Table 3.4.6.1 Groundwater Conditions

Borehole	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Remark
N4	144.2	-	19.8/124.4*	Upon completion	Caved-in** @ 10.7 m
N5	144.5	24.4/120.1	5.0/139.5	59 days after completion	Water level in the piezometer measured at 7.1 m three days after completion and rose to 5.0 m / El. 139.5 m (e.g. to about the o.g. level after 59 days of completion)
N7	144.9	-	13.1/131.8*	Upon completion	Caved-in** @ 7.0 m Soil back-up noted while drilling below El. 124.0 m

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

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

From the recorded stabilized water level in the piezometer installed in Borehole N5, the piezometric level emanating from the sandy silt till was at El. 139.5 m. 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 influenced by the water level in the watercourse, known to be a branch of the East Don River, located about 0.2 km east of Leslie Street.

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

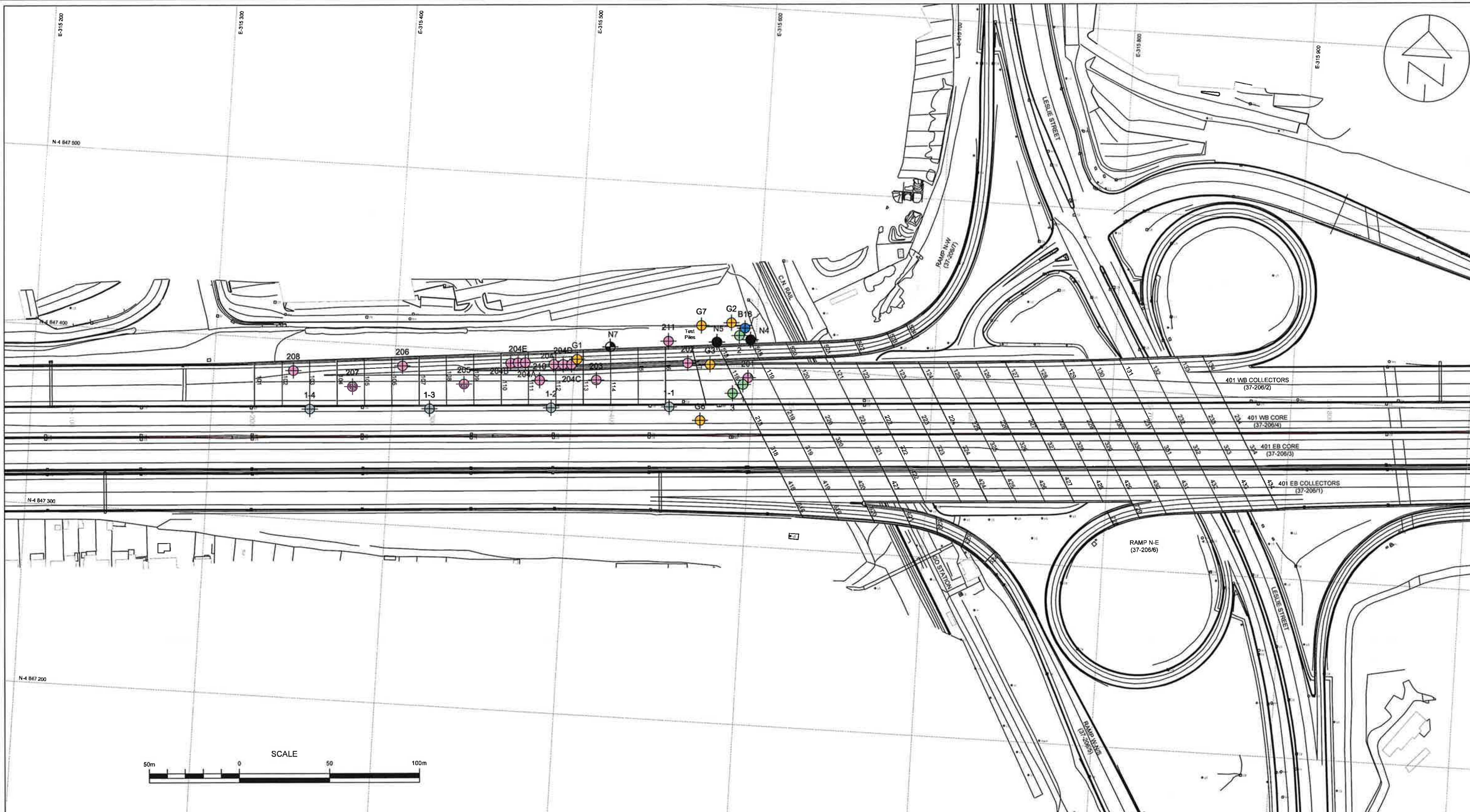
LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.

Zuhtu Ozden, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Drawings



LEGEND

- Borehole (Coffey, 2009)
- Borehole + DCPT (Coffey, 2009)
- Borehole (Department of Highways Ontario, 1964)
- 119 Bent Number (TYP)
- Borehole (Ministry of Transportation, 1990)
- Borehole (Department of Highways Ontario,1965)
- Borehole (Geocon Ltd., 1960)
- Borehole (The Foundation Company of Canada/Geocon, 1953)

NOTES

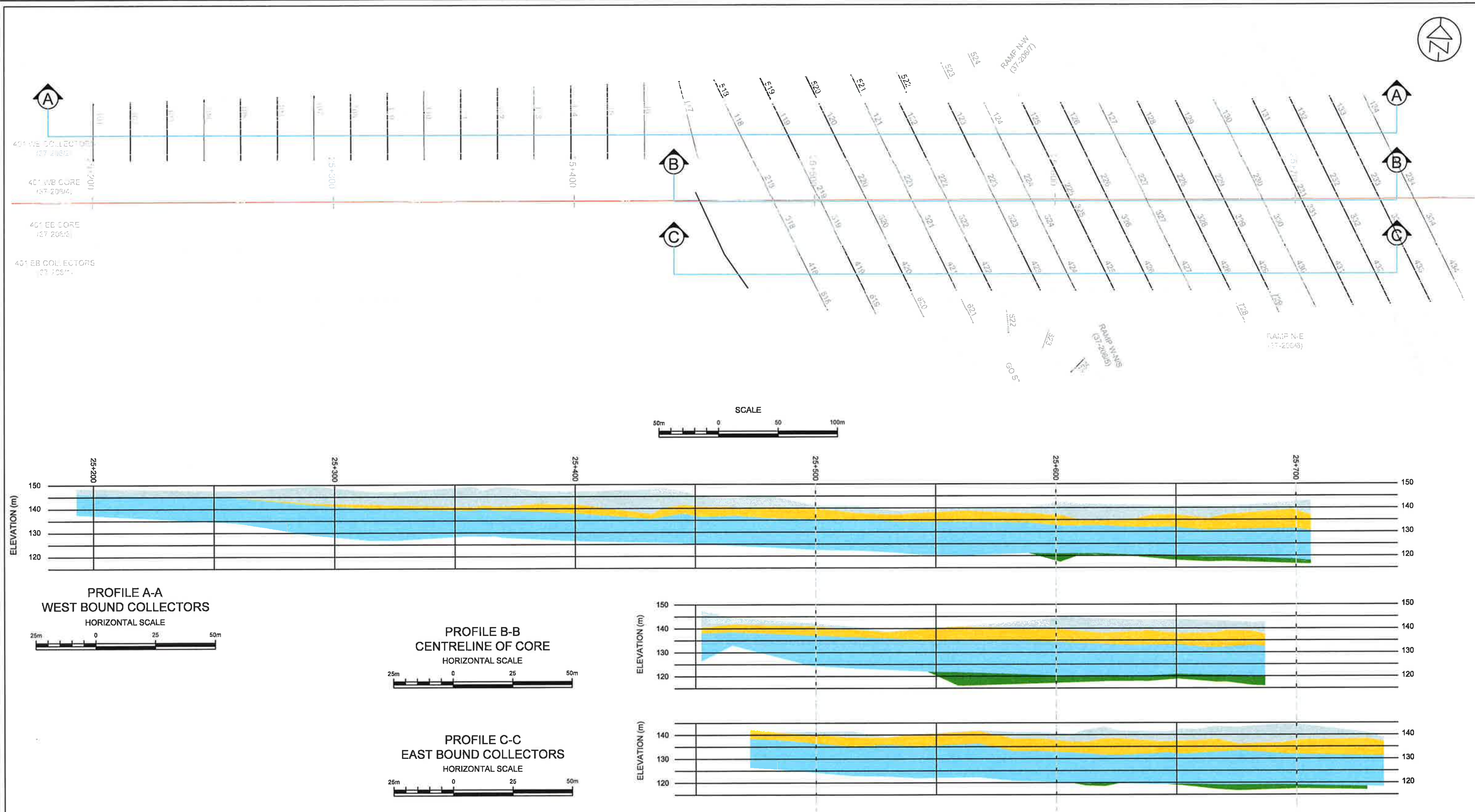
1. This drawing forms part of the report (project number as referenced) and should only be used in conjunction with this report.
2. Base plan provided by Delcan.

drawn	SH
approved	RDP
date	Oct.03, 2011
scale	As Shown
original size	Tabloid

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

client:	DELCAN CORPORATION
project:	VIADUCT HIGHWAY 401 AND LESLIE STREET INTERCHANGE TORONTO, ONTARIO
title:	SITE AND BOREHOLE LOCATION PLAN

Project No.	TRANETOB01245AA-AE	Geocres No.	30M14-335	drawing no:	1



LEGEND

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

NOTES

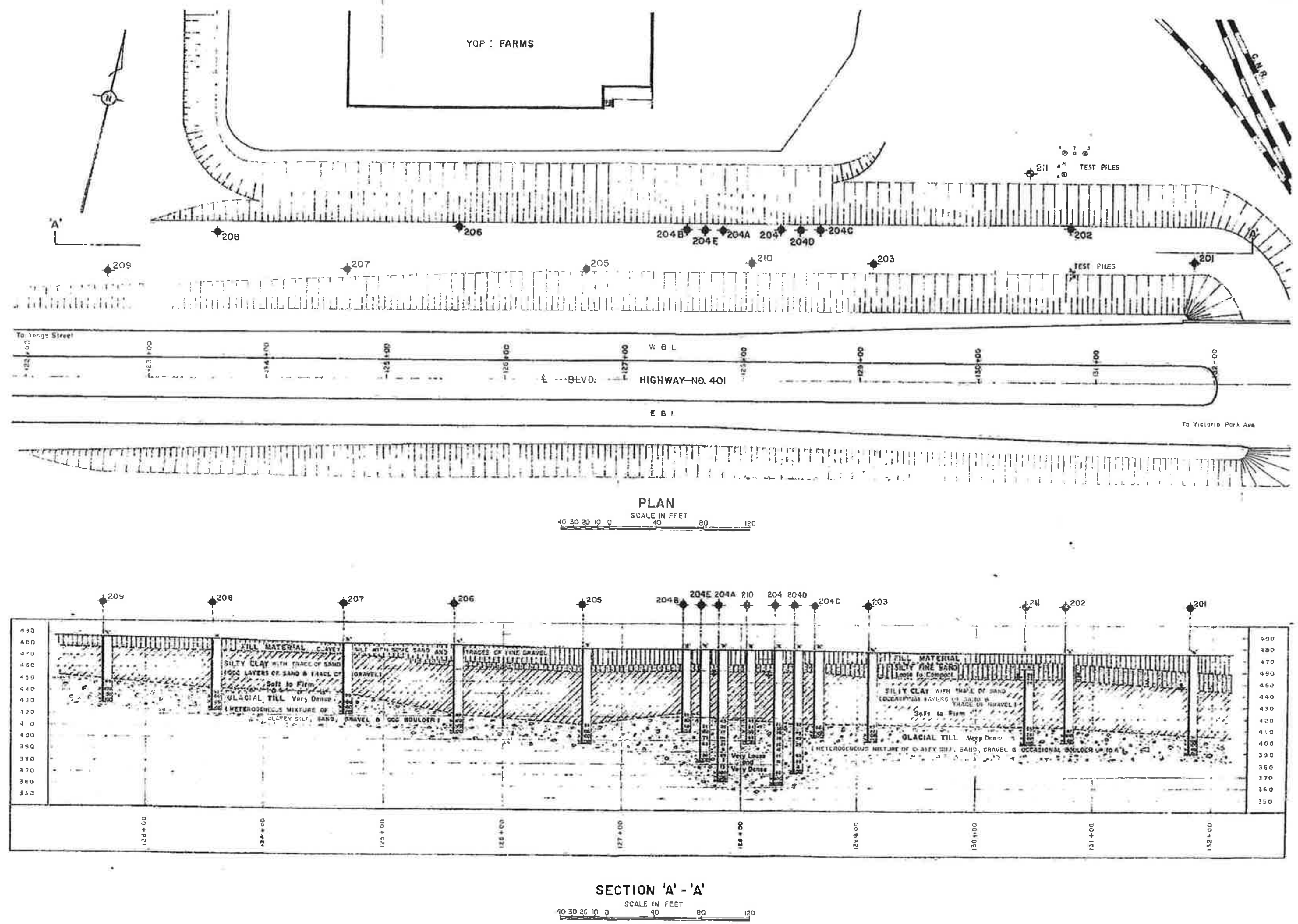
1. The topography has been interpreted from historic borehole data completed at the site by others. Between boreholes the topography was assumed from geological evidence. The topography between boreholes may vary from that shown.
2. For strata details see borehole logs appended to this report.
3. This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

drawn	SH
approved	RDP
date	Oct.03, 2011
scale	As Shown
original size	Tabloid

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

client:	DELCAN CORPORATION
project:	VIADUCT HIGHWAY 401 AND LESLIE STREET INTERCHANGE TORONTO, ONTARIO
title:	ESTIMATED SUBSURFACE PROFILES

Project No.	Geocres No.	drawing no:
TRANETOB01245AA-AE	30M14-335	2



LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation Nov 1963

NO.	ELEVATION	STATION	OFFSET
201	[illegible]	[illegible]	[illegible]
202	474.8	130+78	133 0' LT
203	475 7	129+105	101 7' LT
204	477 8	129+32	130 5' LT
204A	477 8	127+82	130 5' LT
204B	477 8	127+52	130 5 LT
204C	477 8	128+66	130 5' LT
204D	477 8	128+49	130 5' LT
204E	477 8	127+67	130 5 LT
205	478 5	126+68	95 0' LT
206	481 0	125+59	131 1' LT
207	481 4	124+67	94 0' LT
208	484 9	123+57	125 9' LT
209	486 9	122+67	91 2' LT
210	478 0	128+07	101 7 LT
211	[illegible]	130+45	[illegible]

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS		
MAR. 65	D.A.M.	SUBSOIL MODIFICATIONS
MAR. 65	D.A.M.	TEST PILE LOCATIONS
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

LESLIE STREET
(AT YORK FARMS)

KING'S HIGHWAY NO. 401 DIST: NO. 6
CO. YORK METROPOLITAN TORONTO
TWP NORTH YORK LOT CON.

BOREHOLE LOCATIONS & SOIL STRATA

SUBM'D B.G	CHECKED	W.P. NO. 150-61	M.& R. DRAWING NO. 63-F-129A
DRAWN H.R.	CHECKED	JOB NO. 63-F-129	
DATE 16 DEC. 1963		SITE NO.	BRIDGE DRAWING NO.
APPROVED		CONT. NO	

Drawing No. 3

Appendix A

Record of Borehole Sheets

G 1A

C 148

THE FOUNDATION COMPANIES CANADA

OFFICE REPORT ON SOIL EXPLORATION

30M 14-73
GEOCRES No.

DRILL RIG [illegible] JOB [illegible] PENETEST 1A
CASING — (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM [illegible] DATE REPORT 3 SEP. '53
SAMPLER HAMMER. WT. [illegible] DROP [illegible] INCHES COMPILED BY [illegible] CHECKED BY [illegible] BORING DATE 11 AUG '53

SAMPLE CONDITION

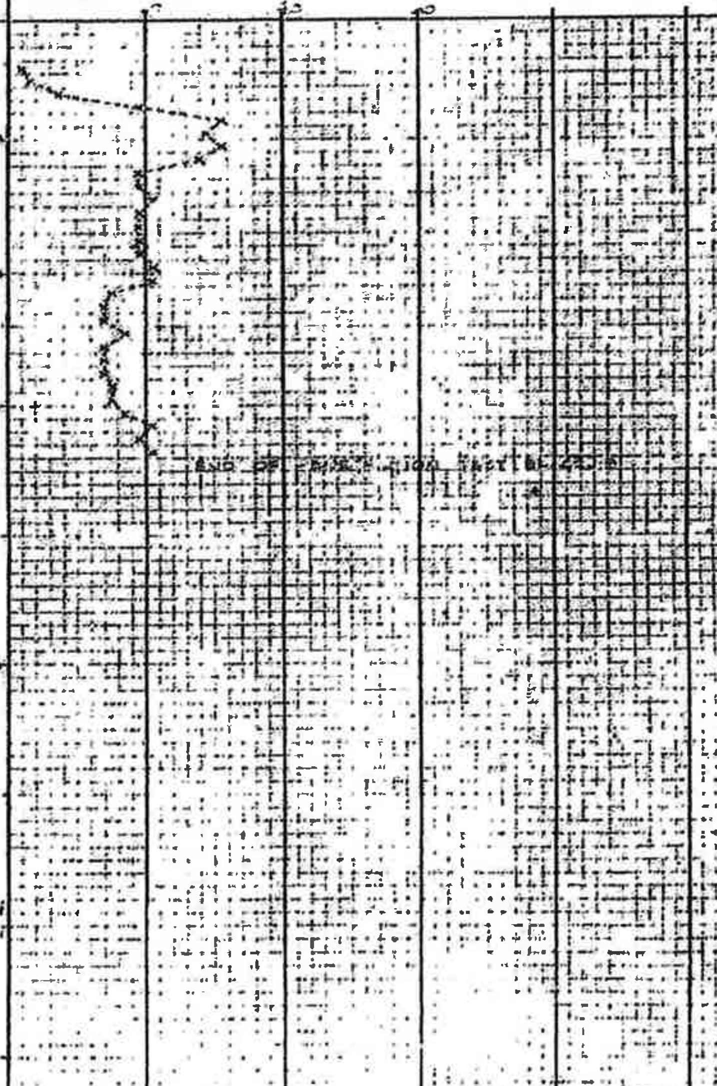
- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

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

ABBREVIATIONS

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

SOIL PROFILE						SAMPLES					
ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W % — ○ NAT. LW. PW. / PENETRATION TEST RESISTANCE BLOWS PER FOOT	OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECO. %
[illegible] 0.0		[illegible]									

G1

C 10 294-5-21

THE FOUNDATION COMPANIES CANADA

OFFICE REPORT ON SOIL EXPLORATION

30M14-93
GEOCRES No.

DRILL RIG. MACHINE JOB 27143 BORING 1

CASING (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM GEODETIC DATE REPORT SEP. 3/58

SAMPLER HAMMER WT. 3__ DROP 12½" INCHES COMPILED BY J.C.C. CHECKED BY ___ BORING DATE 11 AUG 58

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

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

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

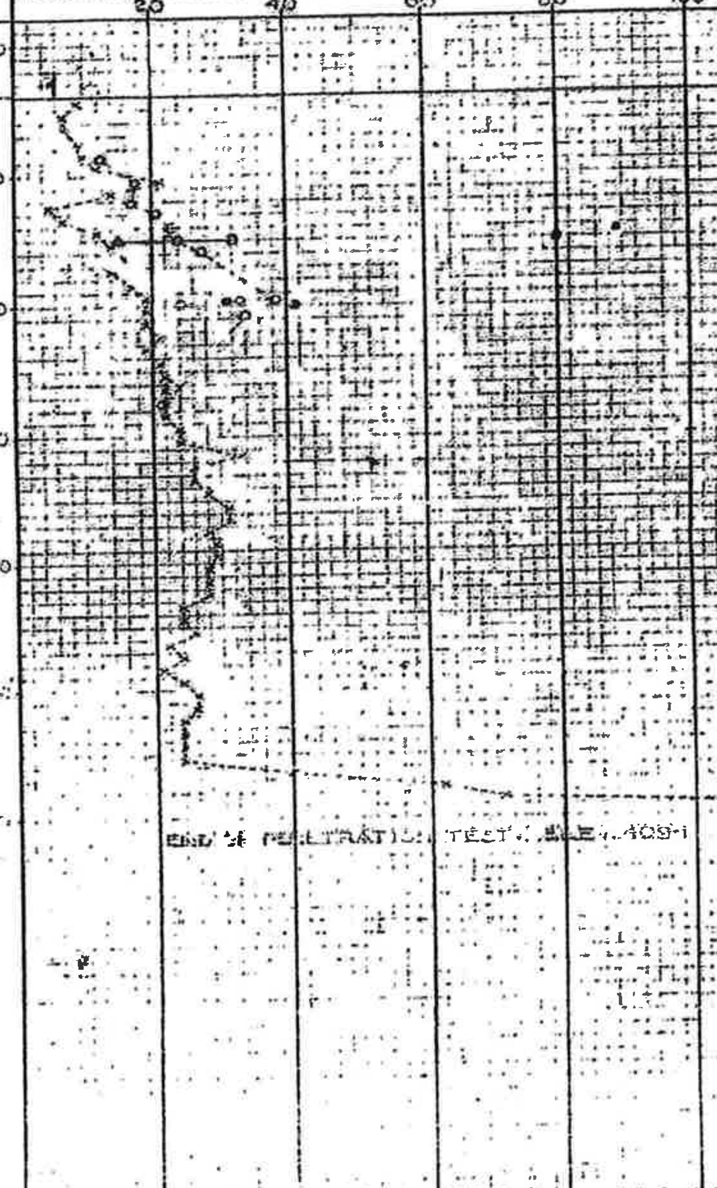
SOIL PROFILE

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT
466.1	W.L. EL. 460.6 AUG 12 '58	GROUND LEVEL	
465.8 0.3		BLACK [illegible]	
15		COMPACT [illegible] BROWN [illegible] AND FINE SAND	
456.3 9.8		[illegible] GREY [illegible] CLAY WITH SILT LAYERS [illegible] SOME GRAVEL	
447.5 18.5		END OF HOLE	

ELEVATION SCALE: 470, 460, 450, 440, 430, 420, 410

WATER CONTENT W% — O NAT. □ LW. Δ Pw.

PENETRATION TEST RESISTANCE BLOWS PER FOOT 20 40 60 80 100



END OF PENETRATION TEST, ELEV. 409.1

SAMPLES

OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECOV. %
					466.1 20
		3⅜"O.O.	1	18	461.6
		3⅜"O.O.	2	30	
		3⅜"D.F.	3	32	458.3 100
		2"T.O.	4	12	454.1 45
		2"T.O.	5	PUSH 5	450.8 58
		2"T.O.	6	6	

THE FOUNDATION COMPANIES CANADA

G2

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG __M[illegible]__ JOB __C[illegible]__ BORING # __2__

CASING __4[illegible]__ (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM __[illegible]__ DATE REPORT __SEP. 4/53__

SAMPLER HAMMER. WT. __[illegible]__ DROP __[illegible]__ INCHES COMPILED BY __[illegible]__ CHECKED BY __[illegible]__ BORING DATE __AUG. 13-15/53__

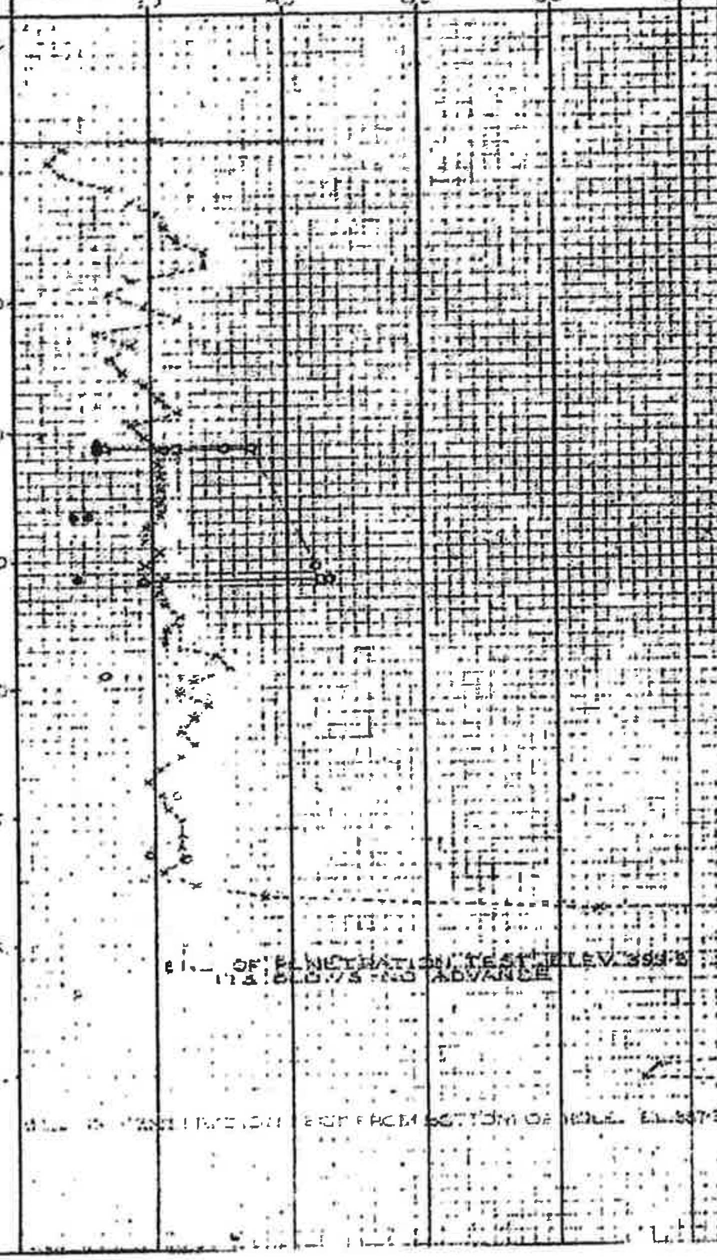
SAMPLE CONDITION: DISTURBED, FAIR, GOOD, LOST

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

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

SOIL PROFILE

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
462.2	W.L. @ EL. 455.3 AUG. 25/53	GROUND LEVEL
[illegible] 0.0		[illegible]
		COMPACT [illegible] BROWN MEDIUM AND FINE SAND SOME GRAVEL
443.2 / 19.0		SOFT GREY STRATIFIED CLAY WITH SILT LAYERS & SOME GRAVEL
423.0 / 39.2		FIRM GREY TILL [illegible] CLAYEY TO [illegible]
401.7 / 60.5		VERY DENSE GR. SAND T[illegible]
395.7 / 66.5		END OF [illegible]



SAMPLES

OTHER TESTS	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECOV. %
	C.S.	—	—	462.2
	3½"TO	1	44	457.2 / 75
	3½"TO	2	17	456.9 / 45
	3½"TO	3	16	448.2 / 0
	3½"TO	4	5	442.1 / 100
γ=116	3½"TO	5	2	438.2 / 84
γ=111 Consolidation	3½"TO	6	3	435.2 / 100
γ=111 γ=159	2"TO	7	PUSH	430.9 / 100
	2"TO	8	PUSH	423.2 / 75
	2"TO	9	2	418.2 / 50
	2"TO	10	1	407.2 / 25
278 557 1104/ft	2"DF	11	54	402.2 / 100
	2"DF	12	142	397.2 / 75

C 140 [illegible]

THE FOUNDATION COMPANIES CANADA

G3

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG: MACHINE — JOB: [illegible] — BORING: [illegible]
CASING: 4", BX (STANDARD SAMPLERS TO FIT UNLESS NOTED) — DATUM: [illegible] — DATE REPORT: [illegible]
SAMPLER HAMMER. WT.: 272 # — DROP: [illegible] INCHES — COMPILED BY: [illegible] CHECKED BY: [illegible] — BORING DATE: [illegible]

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST

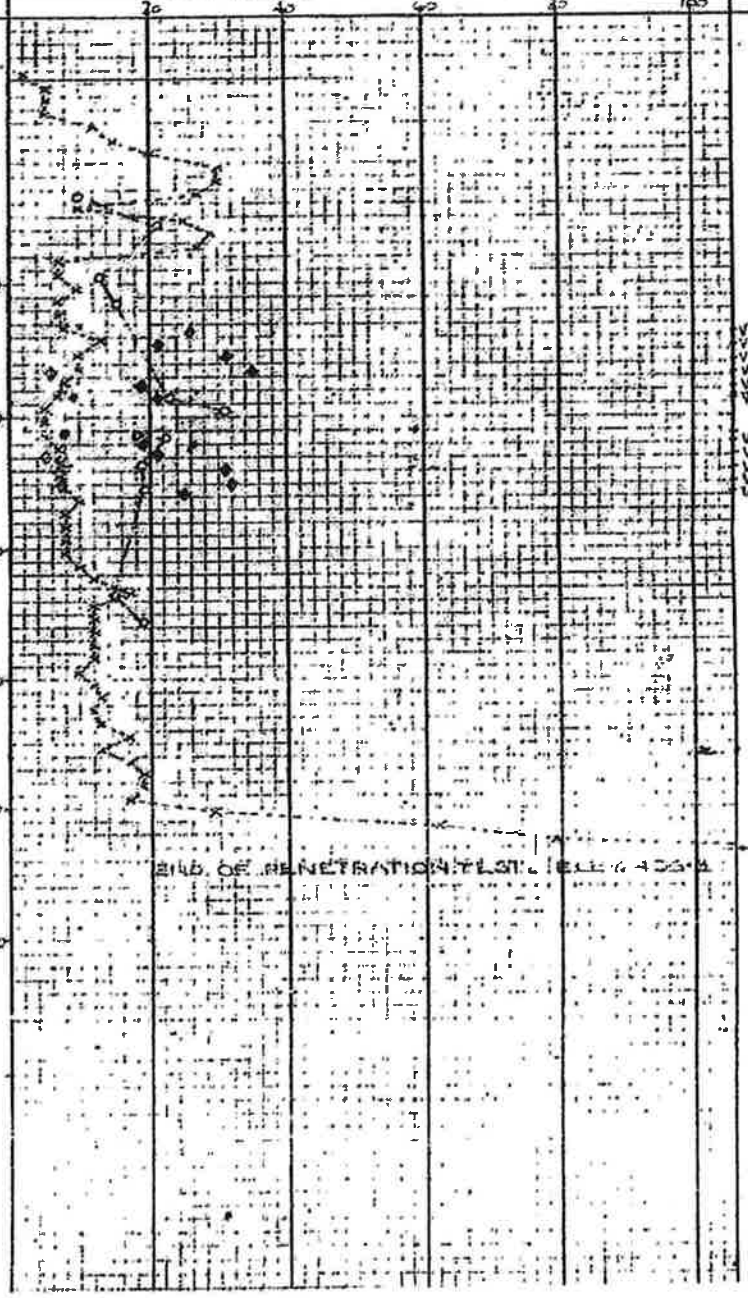
SAMPLE TYPES
- C.S. - CHUNK
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- D.P. - DRIVE PISTON
- T.O. - THIN WALLED OPEN
- T.P. - THIN WALLED PISTON
- F.S. - FOIL SAMPLE
- B.A. - BARREL AUGER
- S.A. - SPIRAL AUGER
- W.S. - WASHED SAMPLE
- R.C. - ROCK CORE

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

SOIL PROFILE

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT
466.2 / 0.0	W.L. [illegible] SEPT.	[illegible] LEVEL / LOOSE [illegible]	
		COMPACT [illegible] BROWN MEDIUM & FINE SAND SOME GRAVEL	
447.4 / 18.9		SOFT GREY STRATIFIED CLAY WITH SILT LAYERS AND SOME GRAVEL	
428.0 / 38.5		FIRM GREY SILTY TILL	
424.4 / 41.8		END OF HOLE	

ELEVATION SCALE / UNDRAINED SHEAR STRENGTH / WATER CONTENT W % (O NAT., □ LW., Δ PW.) / PENETRATION TEST RESISTANCE BLOWS PER FOOT



SAMPLES

OTHER TESTS	CONDITION	TYPE	No.	PENETRATION RESISTANCE	ELEV. / RECOV. %
		3½" TO	1	17	456.2 / 67
		3½" TO	2	8	451.7 / 75
		3½" TO	3	3	448.4 / 100
V		3½" TO	4	PUSHED 2	441.5 / 100
V		2" TO	5	PUSHED	[illegible] / 82
V		2" TO	6	PUSHED	
		2" TO	7	PUSHED	428.5 / 100

E 143 299-9-45

G 6

THE FOUNDATION COMPANIES CANADA

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG [illegible] JOB [illegible] BORING # [illegible]

CASING [illegible] (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM [illegible] DATE REPORT [illegible]

SAMPLER HAMMER WT. [illegible] # DROP [illegible] INCHES COMPILED BY [illegible] CHECKED BY [illegible] BORING DATE AUG. [illegible]

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

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

ABBREVIATIONS

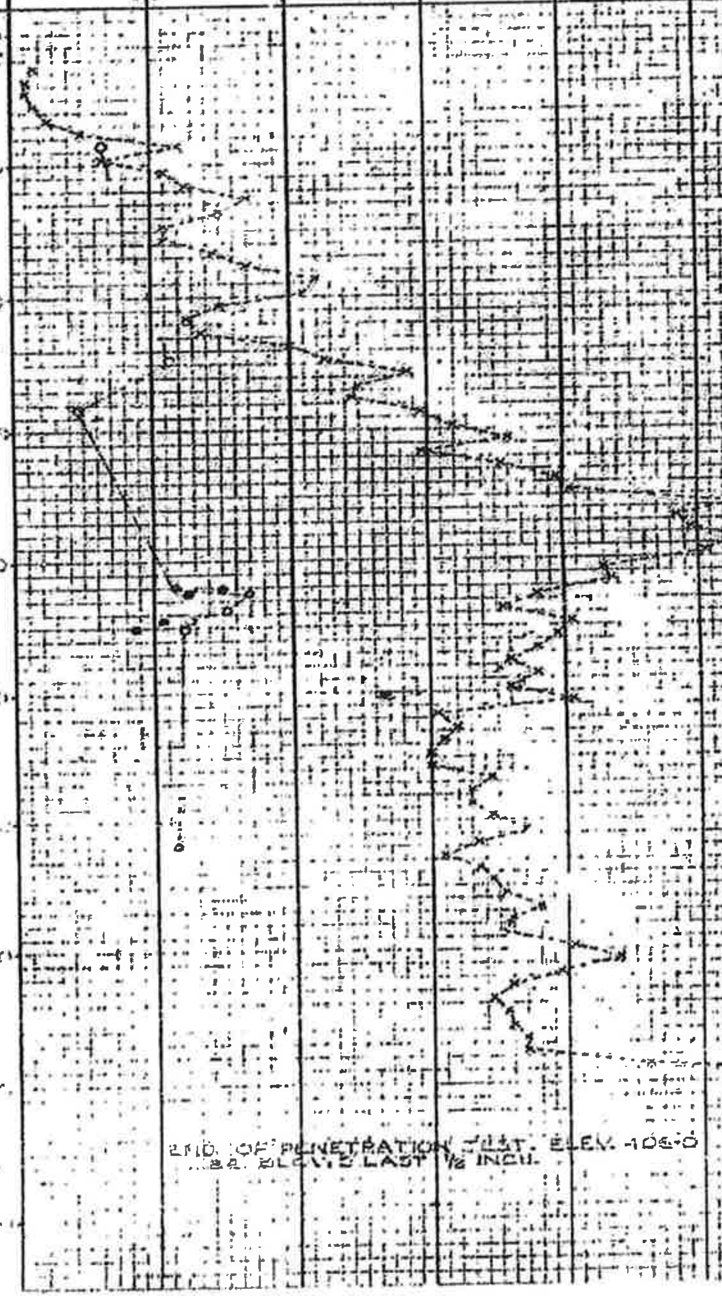
- V. - IN-SITU VANE SHEAR TEST
- M. - MECHANICAL ANALYSIS
- U. - UNCONFINED COMPRESSION
- Qc. - TRIAXIAL CONSOLIDATED QUICK
- Q. - TRIAXIAL QUICK
- S. - TRIAXIAL SLOW
- γ. - UNIT WEIGHT
- K. - PERMEABILITY
- C. - CONSOLIDATION
- CA. - CASING
- WL. - WATER LEVEL IN CASING
- WT. - WATER TABLE IN SOIL

SOIL PROFILE

ELEVN. DEPTH	DESCRIPTION
467.5 / 0.0	[illegible] LEVEL
	COMPACT OLIVE BROWN TILL FILL
451.5 / 15.0	[illegible]
	COMPACT MEDIUM FINE OLIVE BROWN SAND
448.4 / 19.5	SOFT GREY SILTY CLAY WITH SILT [illegible]
PIEZOMETER No. 2	
422.5 / 45.0	LOOSE GREY MEDIUM SAND
417.5 / 50.0	LOOSE GRADED [illegible]
410.9 / 70.0	[illegible]
406.5 / 61.5	END OF [illegible]

WATER CONTENT W% — O NAT. □ LW. Δ PW.

PENETRATION TEST RESISTANCE BLOWS PER FOOT 20 40 60 80 100



SAMPLES

OTHER TESTS	TYPE	No.	PENETRATION RESISTANCE	ELEV. RECOV. %
γ=15.3	3½" TO	1	24	482.9 / 65
	3½" DO	2	52	472.9 / 100
	3½" DO	3	49	467.9 / 100
				464.9 / 100
M	3½" TO	4	22	
				457.9 / 85
	3½" TO	5	32	
	3½" TO	6	17	
	3½" DF	7	16	
γ=12.3 γ=120	consolidation			100
γ=12.8 / γ=24	2½" TO	8	PUSH	443.9 / 0
	3½" TO	9	PUSH	438.9 / 0
	3½" TO	10	PUSH	
	3½" DF	11	PUSH	
	V.T.	13	—	432.9
	3½" DF	14	PUSH	0.75
	3½" TO	12	6	425.9 / 100
	3½" DF	15	4	425.9 / 100
	2" DF	16	3	
	2" TO	17	4	100
	2" DF	18	3	
	2" DF	19	PUSH	419.9 / 100
	2" DF	20	3	414.9 / 100
[illegible]	[illegible]	21	55	410.9
[illegible]	[illegible]	22	47	407.9
[illegible]	[illegible]	23	32	45

G 7

THE FOUNDATION COMPANIES CANADA

OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG. MACHINE
CASING 4" EX (STANDARD SAMPLERS TO FIT UNLESS NOTED)
SAMPLER HAMMER. WT. 300 DROP 18 INCHES
JOB
DATUM
COMPILED BY CHECKED BY
BORING 7
DATE REPORT SEP. 8/53
BORING DATE

SAMPLE CONDITION: DISTURBED, FAIR, GOOD, LOST

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

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

SOIL PROFILE

ELEVN. DEPTH	DESCRIPTION
465.0 / 0.0	GROUND LEVEL
	GRAVEL ...
	MEDIUM TO FINE
	OLIVE GREY ...
	SAND ...
449.5 / 15.5	SOFT GREY
	STRATIFIED
	CLAY WITH SILT
	LAYERS AND
	SOME GRAVEL
425.0 / 40.0	...
	FIRM
	...
	CLAYEY ...
413.0 / 52.0	... SANDY TILL
404.5 / 60.5	...

WATER CONTENT W% — D NAT. — Δ Pw.

PENETRATION TEST RESISTANCE BLOWS PER FOOT 20 40 60 80 100

Elevation scale: 470, 460, 450, 440, 430, 420, 410

... PENETRATION TEST ELEV. 403.0 ... 165 BLOWS LAST INCH

SAMPLES

OTHER TESTS	TYPE	No.	PENETRATION RESISTANCE
	2"DO	1	3
	2"DO	2	18
	2"DO	3	20
	3½TO	4	32
	3½DF	5	35
	3½TO	6	30
	3½TO	7	1
	3½TO	8	1
	2"TO	9	PUSH
	2"TO	10	PUSH
	2"TO	11	PUSH
	2"DO	12	51
	2"DO	13	54

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPEX. I

CONTRACT 57002 BORING # 1 AND 1A DATUM GEODETIC CASING BX

BORING DATE OCT. 27 - NOV. 3, 1959 REPORT DATE MARCH 11, 1960 COMPILED BY MW CHECKED BY

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

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

ABBREVIATIONS

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

SOIL PROFILE

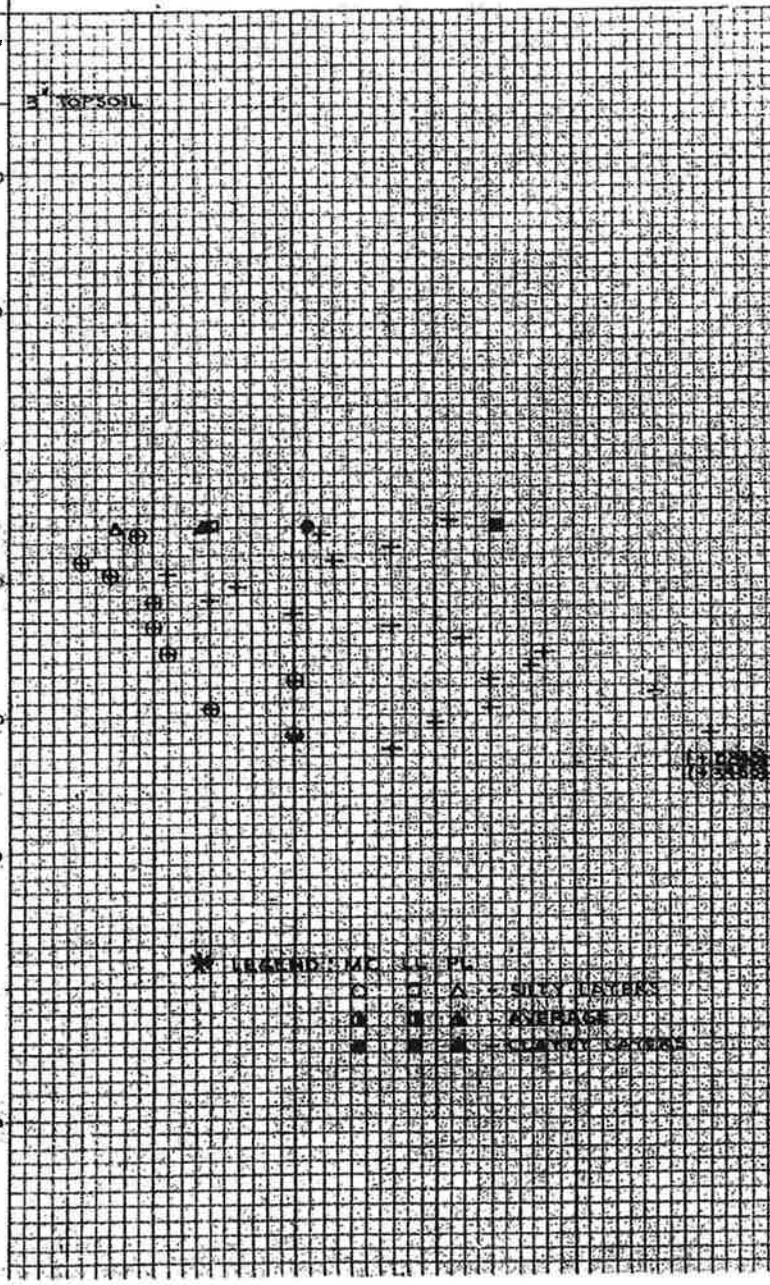
ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
475.5 / 0.0		GROUND LEVEL
		COMPACT TO DENSE FILL
461.0 / 14.5	W.L. @ EL. 456.5 NOV. 2, 1959	COMPACT BROWN SAND
448.5 / 27.0		SOFT TO FIRM GREY CLAY WITH LAYERS OF SILT, SAND, PEBBLES
426.5 / 49.0		COMPACT GREY SILTY SAND
51.0		FIRM GREY CLAYEY TILL
407.5 / 68.0		VERY DENSE GREY SANDY TILL
403.8 / 71.7		END OF HOLE

ELEVATION SCALE: 480, 470, 460, 450, 440, 430, 420, 410, 400

SHEAR STRENGTH LBS./SQ.FT. 200 400 600 800 1000 — UNDRAINED TRIAXIAL TEST ● (NAT.) ○ (REM.) — VANE TEST + (NAT.) ⊕ (REM.)

WATER CONTENT W% 20 40 60 80 100 — ○ NAT. □ LW △ PW *

DYNAMIC PENETRATION TEST BLOWS PER FOOT



OTHER TESTS: C

SAMPLES

TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
2" DO	1A	8
DO	2A	43
DO	3A	47
DO	4A	29
DO	5A	20
WS	6A	—
5" PS	1	HYD PUSH
PS	2	HYD PUSH
PS	3	HYD PUSH
PS	4	HYD PUSH
PS	5	HYD PUSH
PS	6	—
PS	7	HYD PUSH
2" DO	7A	5
SO	8A	PUSH
DF	9A	—
SO	10A	4
SO	11A	8
SO	12A	>100
DF	13A	—

GEOCON

2

OFFICE REPORT ON SOIL EXPLORATION

APPEX. I

CONTRACT 57002 BORING # 2 AND 2A DATUM GEODETIC CASING BX

BORING DATE NOV. 4 - 6, 1959 REPORT DATE MARCH 11, 1960 COMPILED BY M.W. CHECKED BY ML

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

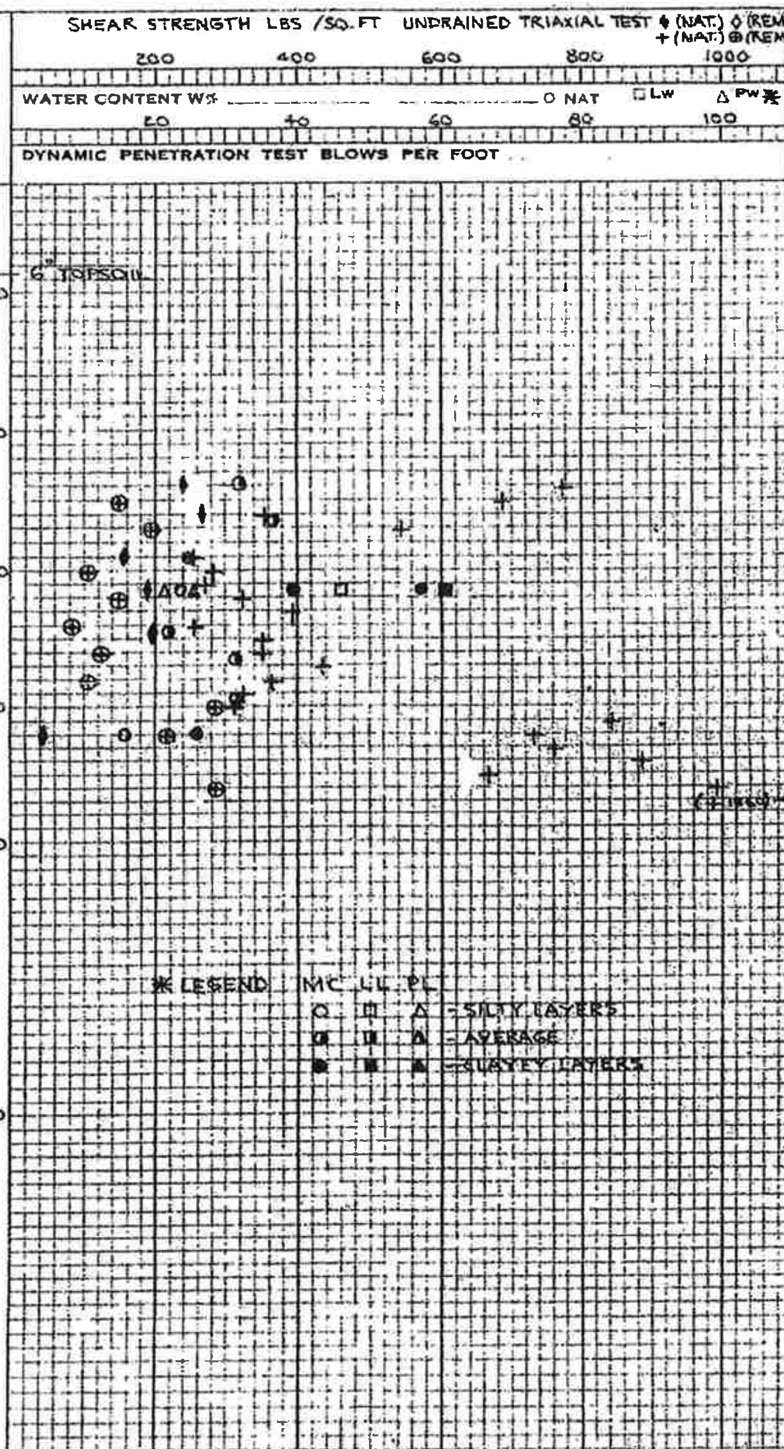
- A.S. - AUGER SAMPLE
- S.T. - SLOTTED TUBE
- W.S. - WASHED SAMPLE
- D.O. - DRIVE-OPEN
- D.F. - DRIVE-FOOT VALVE
- C.S. - CHUNK SAMPLE
- F.S. - FOIL SAMPLE
- S.O. - SLEEVE-OPEN
- S.F. - SLEEVE-FOOT VALVE
- T.O. - THIN WALLED OPEN
- R.C. - ROCK CORE
- P.S. - PISTON SAMPLE

ABBREVIATIONS

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

SOIL PROFILE

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION
461.8 / 0.0		GROUND LEVEL
		COMPACT BROWN SAND
447.3 / 14.5		SOFT TO FIRM GREY CLAY WITH LAYERS OF SILT, SAND, PEBBLES
428.8 / 33.0		FIRM GREY CLAYEY TILL
406.8 / 55.0 / 56.0		END OF HOLE VERY DENSE GREY SANDY TILL



OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
		2" DO	1A	31
		DO	2A	26
γ=117		3" PS	1	HYD PUSH
γ=111		PS	2	"
γ=121		PS	3	"
γ=121 C		PS	4	"
γ=116		PS	5	"
		PS	6	"
		PS	7	"
γ=122		PS	8	"
		2" SO	3A	4
		SO	4A	4
		SO	5A	4
		DO	6A	90

GEOCON

3

OFFICE REPORT ON SOIL EXPLORATION

APPEX. I

CONTRACT 57002 BORING # 3 AND 3A DATUM GEODETIC CASING BX

BORING DATE JAN. 21 - FEB. 4, 1960 REPORT DATE MARCH 14, 1960 COMPILED BY M.W. CHECKED BY

SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLE TYPES

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

ABBREVIATIONS

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

SOIL PROFILE

ELEVN. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
475.0 / 0.0		GROUND LEVEL		470
		COMPACT TO DENSE FILL		
459.0 / 16.0		COMPACT BROWN SAND		460
449.5 / 25.5	W.L. @ EL. 457.4 FEB. 4, 1960	SOFT TO FIRM GREY CLAY WITH LAYERS OF SILT, SAND, PEBBLES		450 / 440 / 430
428.0 / 47.0		END OF HOLE		420

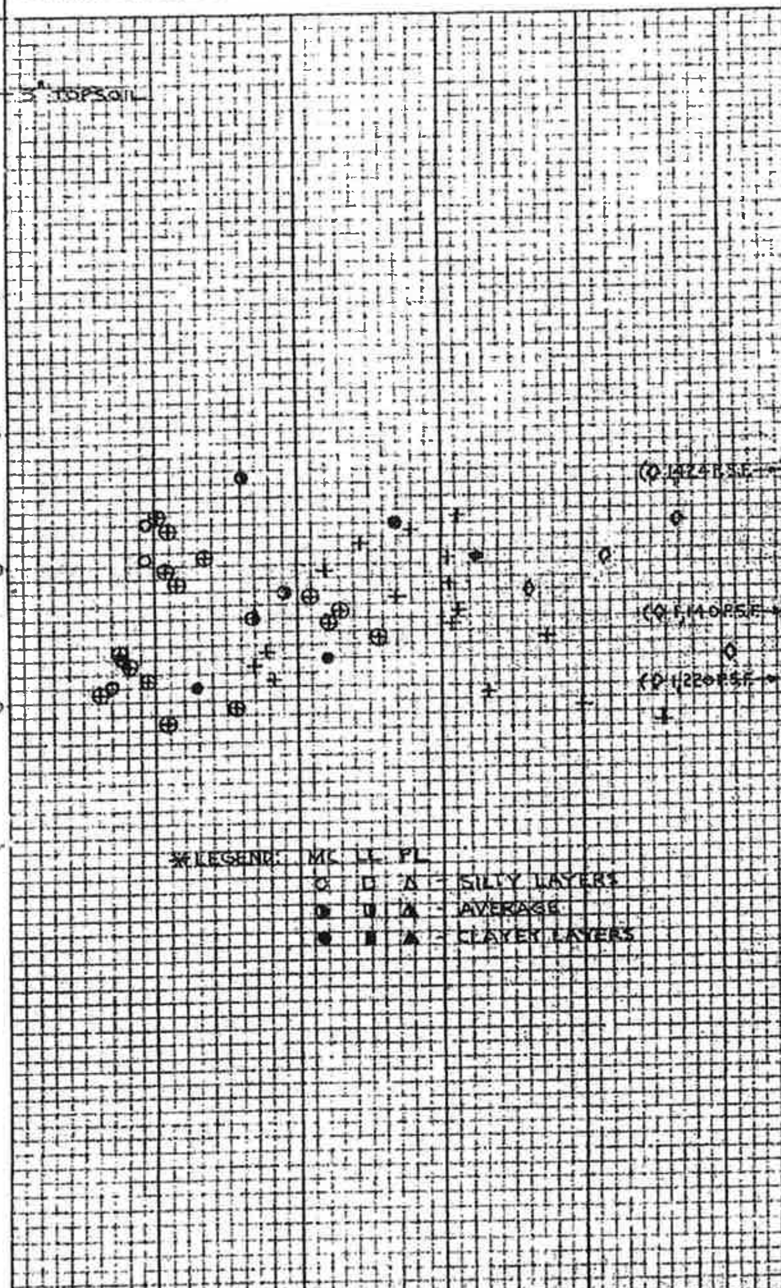
SHEAR STRENGTH LBS./SQ.FT. — UNDRAINED VANE TEST; TRIAXIAL TEST (NAT.) (REM.)

200 400 600 800 1000

WATER CONTENT W% — O NAT. ◫ LW △ PW

20 40 60 80 100

DYNAMIC PENETRATION TEST BLOWS PER FOOT



OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
γ=119		PS	1A	HYD. PUSH
γ=124		PS	2A	"
γ=125		PS	3A	"
γ=115		PS	4A	"
γ=128		PS	5A	"
γ=117		PS	6A	"
γ=119		PS	7A	"
		DO	8A	"
		DO	9A	"

FORM ...

OFFICE REPORT ON SOIL EXPLORATION

B18

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 18

FOUNDATION SECTION

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

LOCATION Stn. 124+80 and 194' Lt. of ℄ Hwy. 401
BORING DATE June 22, 1964.
BOREHOLE TYPE Washboring using BX casing.

ORIGINATED BY B.M.Q.
COMPILED BY B.M.Q.
CHECKED BY M.D.

ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	REMARKS
461.5	Groundlevel					
0.0	Clayey silt and sand. Brown.				460	W.L. at El. 455
					450	
446.5 / 15.0	Silt Grey					
17.0	Grey silty clay with some sand. Grey.				440	
					430	
					420	
					410	
403.0 / 58.6	Heterogeneous mixt. of clayey silt, sand and gravel up to 2" Ø, (Glacial till) Boulder at 66'-6" V. dense. Grey.	1	SS	100 for 6"	400	
		2	SS	100 for 6"		
395.0 / 66.6	End of Bore Hole					

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F.
LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W
WATER CONTENT %
BULK DENSITY γ P.C.F.

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 211

FOUNDATION SECTION

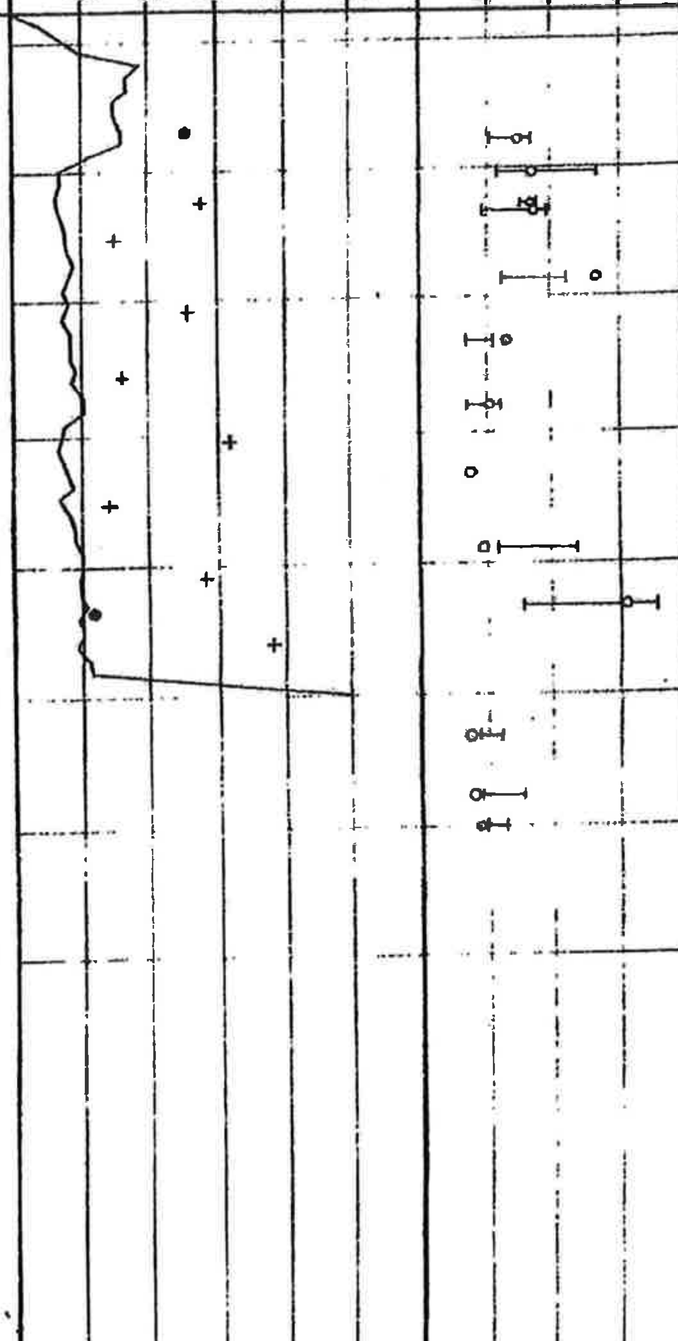
JOB 63-F-129
W.P. 150-61
DATUM G.S.C.

LOCATION Sta. 130/43 & 181' left of ℄ Hwy. 401
BORING DATE Nov. 28, 1963.
BOREHOLE TYPE Washboring using HX and BX casings.

ORIGINATED BY B.M.G.
COMPILED BY B.M.G.
CHECKED BY M.D.

ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
462.5	Groundlevel					
	Topsoil					
1'-3"	Silty fine sand, (Traces of organics to El. 459). Compact. Brown to br. grey.	1	SS	29		
		2	SS	23		WL at El. 457.0
452.5 / 10.0		3	SS	14	119	
	Silty clay and trace of fine sand. (Occasional 6" to 9" layers of sand and gravel up to 1" Ø, below El. 420.0). Soft to firm. Grey.	4	TW	P		S = 3.5
		5	TW	P		S = 4.3
		6	TW	P		S = 5.2
		7	TW	P		S = 2.3
		8	TW	P		S = 2.3
		9	TW	P		S = 2.3
		10	TW	P	102	S = 4.7
413.0 / 49.6		11	TW	P		S= 3.0
	Heterogeneous mixt. of clayey silt, sand and gravel. 6" Boulder around El. 398. (Glacial Till). V. dense. Grey.	12	SS	55		Gr 2% Sa 28 Si) Cl) 70
		13	SS	74		
		14	SS	>100		Gr 2% Sa 20 Si) Cl) 78
		15	SS	>100		
397.0 / 65.6	End of borehole.	16	SS	>100		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH P.S.F.: + - Field Vane Test; ● - Unconfined Comp. Test. 200 400 600 800 1000
LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w
WATER CONTENT %: 15 30 45
ELEV SCALE: 460, 450, 440, 430, 420, 410, 400, 390



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOC...

FIG. 6

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 1-1

METRIC

W.P. 280-88-01/A — LOCATION Co-ords. N 4 847 154.1; E 315 554.4 — ORIGINATED BY MS

DIST 6 HWY 401 — BOREHOLE TYPE Solid Stem Auger, Cone Test — COMPILED BY KA

DATUM Geodetic — DATE 88 05 26-27 — CHECKED BY DD

Soil Profile

ELEV / DEPTH	DESCRIPTION
150.8	Ground Surface
0.0	Asphalt; Sand, Some Gravel; Sandy Silt with Irregular Layers of Clayey Silt and Sand, Loose to Dense (Fill)
139.2 / 11.6	Trace Organics; Silty Sand, Compact to Dense; Clayey Silt (CL) to Silty Clay (CI), Frequently Varved (1 cm), Some Sand, Trace Gravel, Occ. Silt, Sand Zones and Boulders, Firm to Very Stiff
125.5 / 25.3	Heterogeneous Mixture Sandy Silt to Silty Sand, Some Clay, Trace Gravel, Occ. Clayey Silt Zones, Occ. Boulders, Very Dense
123.1 / 27.7	End of Borehole

* W.L. Recorded on 86-06-10

Samples

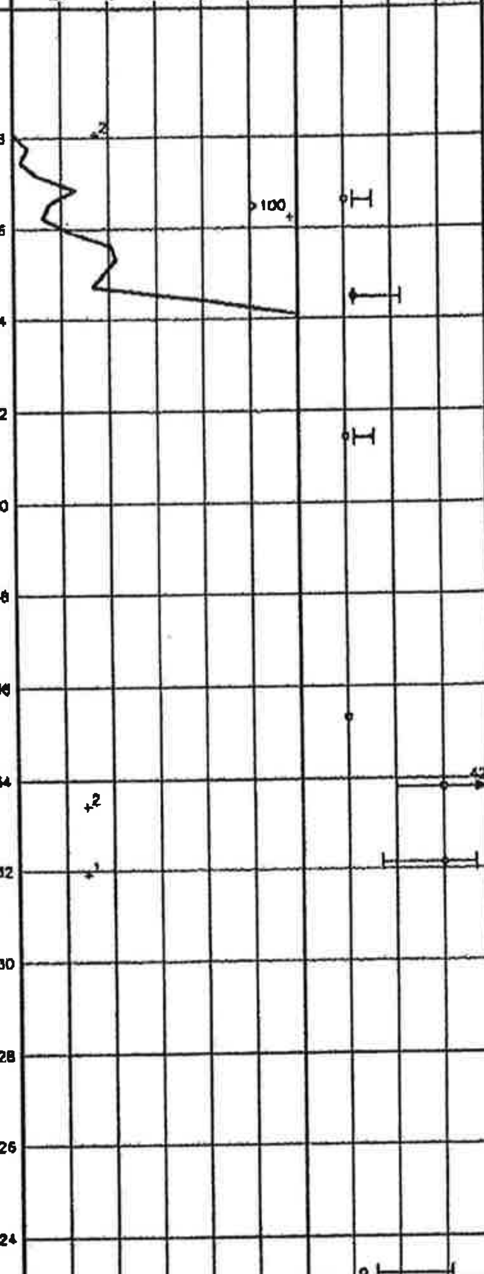
NUMBER	TYPE	'N' VALUES
1	SS	32
2	SS	16
3	SS	6
4	SS	8
5	TW	PH
6	SS	8
7	SS	14
8	SS	19
9	SS	37
10	SS	14
11	SS	18
12	SS	14
13	SS	24
14	SS	31
15	SS	15
16	SS	9
17	SS	5
18	TW	PH
19	SS	13
20	SS	6
21	SS	100
22	SS	80 /15cm

GROUND WATER CONDITIONS: water level at approx. elev. 147.5 (*)

ELEVATION SCALE: 148, 146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100



PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

Remarks & Grain Size Distribution (%)

Sample	GR	SA	SI	CL
6	2	36	50	12
9	2	33	43	22
11	2	32	46	20
15	22	64	9	5
16	0	11	29	60
17	2	11	30	57
22	3	22	37	38

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

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 1-2

1 OF 1 METRIC

W.P. 250-86-01/A LOCATION Co-ords. N 4 847 150.0; E 315 488.6 ORIGINATED BY MS

DIST 5 HWY 401 BOREHOLE TYPE Hollow Stem Auger COMPILED BY KA

DATUM Geodetic DATE 88 05 30-31 CHECKED BY DD

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
151.3	Ground Surface
0.0	Asphalt
	Sand, Some Gravel
	Sandy Silt with Irregular Layers of Clayey Silt and Sand Compact (Fill)
142.5 / 8.8	Trace Organics
	Silty Sand Compact
	Clayey Silt (CL) to Silty Clay (CI) Frequently Varved (1 cm) Some Sand, Trace Gravel Occ. Silt, Sand Zones and Boulders Firm to Hard
	Silt Loose
128.7 / 22.6	Heterogeneous Mixture Sandy Silt to Silty Sand Some Clay, Trace Gravel Occ. Clayey Silt Zones Occ. Boulders Very Dense
123.6 / 27.7	End of Borehole W.L. on 88-06-10

SAMPLES

NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	24	
2	SS	22	1 40 50 9
3	SS	11	
4	SS	26	
5	SS	13	
6	SS	22	2 34 43 21
7	SS	25	
8	SS	21	
9	SS	35	
10	SS	18	1 7 33 59
11	SS	8	
12	TW	PH	
13	TW	*	
14	SS	5	
15	SS	9	3 36 46 15
16	SS	6	20 32 36 12
17	SS	13	0 8 25 67
18	SS	52	
19	SS	50/10cm	10 44 32 14

GROUND WATER CONDITIONS: P1

ELEVATION SCALE: 150, 148, 146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

50 (at elevation 130)

* GROUND WATER CONDITIONS

PIEZO. NO.	GROUND WATER ELEVATION (Metres)
1	139.1

* TW sank by its own weight

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1-3

METRIC

W.P. 260-86-01/A LOCATION Co-ords. N 4 847 145.9; E 315 420.7 ORIGINATED BY MS

DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KA

DATUM Geodetic DATE 88 05 01-02-03 CHECKED BY DD

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
151.7	Ground Surface
0.0	Asphalt
	Sand, Some Gravel
	Sandy Silt with Irregular Layers of Clayey Silt and Sand Compact to Dense (Fill)
144.4 / 7.3	Clayey Silt (CL) to Silty Clay (CI) Frequently Varved (1 cm) Some Sand, Trace Gravel Occ. Silt, Sand Zones and Boulders — Trace Organics; Sandy Silt Loose to Compact
128.8 / 22.9	Heterogeneous Mixture Sandy Silt to Silty Sand Some Clay, Trace Gravel Occ. Clayey Silt Zones Occ. Boulders Dense to Very Dense

SAMPLES

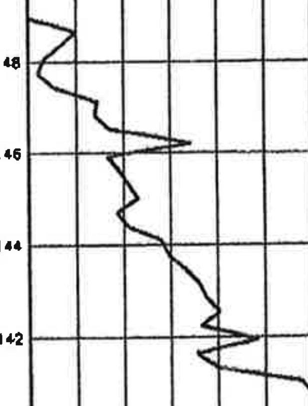
NUMBER	TYPE	'N' VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	37	
2	SS	14	0 18 45 37
3	SS	35	
4	SS	34	
5	SS	23	0 32 43 25
6	SS	36	
7	SS	39	
8	SS	14	
9	SS	19	2 31 45 22
10	SS	11	3 33 40 24
11	SS	8	
12	SS	6	1 25 50 24
13	SS	13	6 38 44 12
14	SS	14	
15	SS	11	
16	SS	9	
17	SS	8	
18	SS	5	10 18 63 9
19	SS	8	
20	TW	PH	1 32 53 14
21	SS	33	
22	SS	57	
23	SS	100 /25cm	1 20 72 7
24	SS	25	

GROUND WATER CONDITIONS: PI

ELEVATION SCALE: 150, 148, 146, 144, 142, 140, 138, 136, 134, 132, 130, 128, 126, 124, 122

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED; + FIELD VANE; • QUICK TRIAXIAL; x LAB VANE: 20 40 60 80 100



PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued

$+^3$, x^3: Numbers refer to Sensitivity

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

Continued

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 1-3

METRIC

W.P. 260-86-01/A LOCATION Co-ords. N 4 847 145.9; E 315 420.7 ORIGINATED BY MS

DIST 5 HWY 401 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KA

DATUM Geodetic DATE 88 08 01-02-03 CHECKED BY DD

SOIL PROFILE: ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued										
30.5	***		25	SS	70						
119.7							120				
32.0	End of Borehole										

*** Heterogeneous Mixture
Sandy Silt to Silty Sand
Some Clay, Trace Gravel
Occ. Clayey Silt Zones
Occ. Boulders
Very Dense

W.L. Recorded on 88-06-10

• GROUND WATER CONDITIONS

PIEZO. NO.	GROUND WATER ELEVATION (Metres)
1	149.6

$+^3, \times^5$: Numbers refer to Sensitivity $15 \overset{20}{\underset{10}{\oplus}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 1-4

METRIC

W.P. 250-86-01/A
LOCATION Co-ords. N 4 847 141.3; E 315 354.3
ORIGINATED BY MS
DIST 6 HWY 401
BOREHOLE TYPE Hollow Stem Auger
COMPILED BY KA
DATUM Geodetic
DATE 88 06 06-08
CHECKED BY DD

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
152.0	Ground Surface
0.0	Asphalt
	Sand, Some Gravel
	Sandy Silt with Irregular Layers of Clayey Silt and Sand Compact to Very Dense (Fill)
145.9 / 6.1	Trace Organics
	Clayey Silt (CL) to Silty Clay (CI) Frequently Varved (1 cm) Some Sand, Trace Gravel Occ. Silt, Sand Zones and Boulders Firm to Hard
	Sandy Silt, Compact
	Sandy Silt, Compact
133.7 / 18.3	Heterogeneous Mixture Sandy Silt to Silty Sand Some Clay, Trace Gravel Occ. Clayey Silt Zones Occ. Boulders Very Dense
129.0 / 23.0	End of Borehole

SAMPLES

NUMBER	TYPE	'N' VALUES	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	60 /15cm	
2	SS	24	
3	SS	20	3 36 41 20
4	SS	17	
5	SS	19	
6	SS	31	4 13 36 47
7	SS	17	
8	SS	14	
9	SS	16	
10	SS	23	6 39 44 11
11	SS	15	
12	SS	17	
13	SS	8	
14	SS	25	
15	SS	55	
16	SS	100 /23cm	
17	SS	60 /10cm	
18	SS	75 /15cm	

ELEVATION SCALE: 150, 148, 146, 144, 142, 140, 138, 136, 134, 132, 130

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

* W.L. Recorded on 88-06-10

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

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 1-5

METRIC

W.P. 280-86-01/A LOCATION Co-ords. N 4 847 137.2; E 315 290.6 ORIGINATED BY MS

DIST 6 HWY 401 BOREHOLE TYPE Hollow Stem Auger, Cone Test COMPILED BY KA

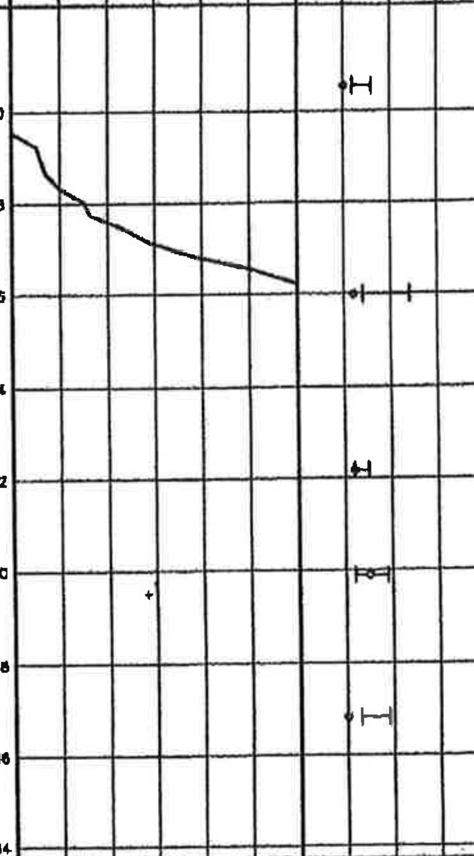
DATUM Geodetic DATE 88 06 09 CHECKED BY DD

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
152.3	Ground Surface					
0.0	Asphalt					
	Sand, Some Gravel					
	Sandy Silt with Irregular Layers of Clayey Silt and Sand Compact (Fill)	1	SS	34		1 37 44 18
		2	SS	15		
148.3 / 4.0	Trace Organics	3	SS	14	▽ *	
		4	SS	28		5 29 45 21
		5	SS	13		
	Sandy Silt Compact	6	SS	10		
		7	SS	12		7 40 42 11
		8	SS	120/0cm		
	Clayey Silt (CL) to Silty Clay (CI) Frequently Varved (1 cm) Some Sand, Trace Gravel Occ. Silt, Sand Zones and Boulders Stiff to Hard	9	SS	12		
		10	SS	5		1 30 49 20
		11	TW	**		
		12	TW	**		
		13	SS	38		
137.1 / 15.2	Heterogeneous Mixture Sandy Silt to Silty Sand Some Clay, Trace Gravel Occ. Clayey Silt Zones Occ. Boulders Very Dense	14	SS	73		7 13 63 17
		15	SS	60/10cm		
133.7		16	SS	60/15cm		
18.6	End of Borehole					

ELEVATION SCALE: 150, 148, 146, 144, 142, 140, 138, 136, 134

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED + FIELD VANE • QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100



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

UNIT WEIGHT γ kN/m³

* W.L. recorded on 88-06-10

** TW sank by its own weight

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

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB01245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N4

METRIC

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

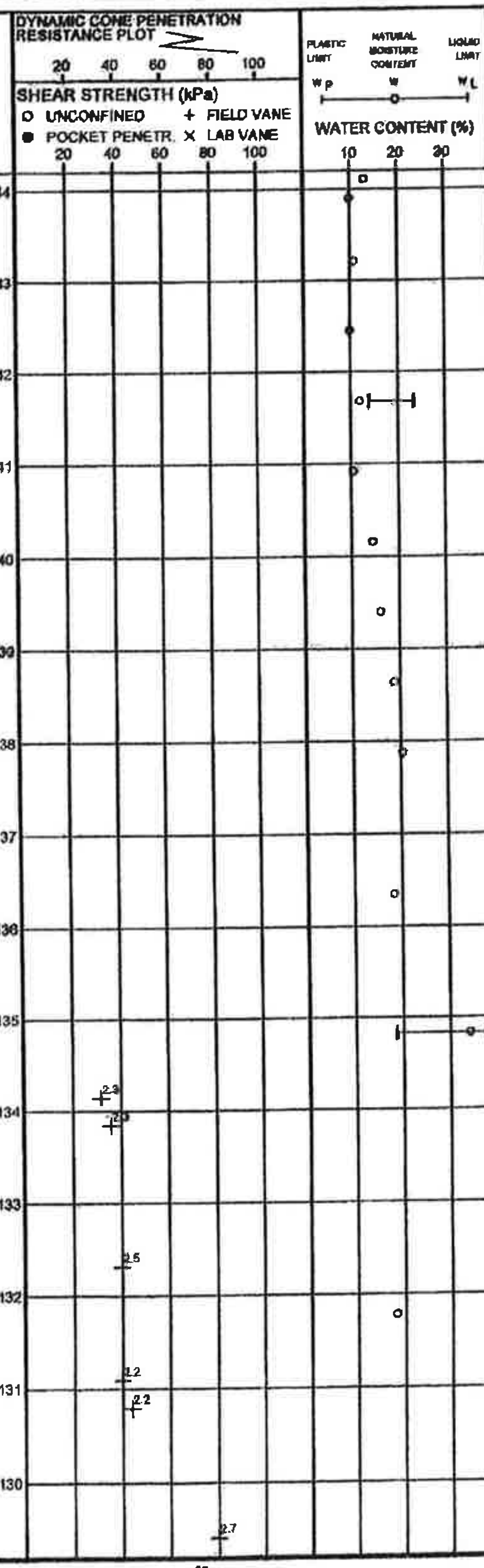
SOIL PROFILE

ELEV. DEPTH	DESCRIPTION	STRAT PLOT
144.2	GROUND SURFACE	
0.0	0.1 m Topsoil; concrete pieces; FILL: SANDY SILT TO SILTY SAND, tr. to some clay, tr. gravel, loose to dense, moist; clayey; brown; grey; dark brown peaty Topsoil	Fill
139.7 / 4.5	SAND, tr. to some silt, brown, loose to dense, wet; tr. rootlets	Sand
137.4 / 6.8	SILT, with fine sand, some clay, grey, dilatant, compact, wet	Silt
135.8 / 8.4	SILTY CLAY, some sand, tr. gravel, grey, firm to stiff, wet	Clay
129.2	Continued Next Page	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8	
2	SS	37	
3	SS	11	
4	SS	12	
5	SS	6	
6	SS	13	
7	SS	8	
8	SS	39	spoon wet; 0 86 (14)
9	SS	28	spoon wet
10	SS	17	spoon wet; 0 23 66 11
11	SS	1	4 24 39 33
12	TW	PM	
13	SS	1	
14	TW	PM	

GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³



+³, ×³ Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB01245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N4

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847415.3 & Easting 316597.1) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK

DATUM Geodetic DATE 12/1/2009 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
129.2 15.0	SILTY CLAY some sand, tr gravel grey, firm to stiff, wet	15	SS	1		
		16	SS	1		
		17	SS	3		
		18	SS	4		
123.4 20.8	CLAYEY SILT TILL some sand grey, hard, moist; sandy	19	SS	100 / 23 cm		3 32 45 20
	cobbles/boulders inferred	20	SS	100/10 cm		spoon wet; auger grinding @ 23.5 and 24.1 m
119.5 24.7	some gravel, tr. sand	21	SS	100 / 29 cm		spoon wet

End of Borehole
Water level @ 19.8 m upon completion before auger pull out
Borehole caved-in @ 10.7 m after auger pull out

Elevation scale: 129, 128, 127, 126, 125, 124, 123, 122, 121, 120

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

20 15 ⊕ 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No N5

METRIC

GWP 2006-E-0012	LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847412.9 & Easting 315576.4)		ORIGINATED BY RK
DIST ____ HWY 401	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY SK
DATUM Geodetic	DATE 12/7/2009		CHECKED BY ZO

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
144.5	GROUND SURFACE				
0.0	0.1 m Topsoil	1	SS	9	
	FILL: SANDY SILT TO SILTY SAND, tr gravel, tr to some clay, loose to compact, moist; clayey silt some sand	2	SS	19	
	tr. rootlets	3	SS	16	
		4	SS	9	
	brown / grey	5	SS	20	2 34 38 26
	tr. rootlets, clayey; tr. organics and rootlets	6	SS	15	
	peaty Topsoil	7	SS	11	
139.4 / 5.1	SILTY SAND TO SANDY SILT, brown, compact to dense, wet	8	SS	38	spoon wet
	silt	9	SS	16	
	brown / grey, tr clay	10	SS	14	spoon wet; 0 54 37 9
136.2 / 8.3	SILTY CLAY, tr. to some sand, tr gravel, grey, wet	11	SS	1	spoon wet
		12	TW	PM	
		13	SS	1	
	firm / stiff to v. stiff	14	SS	1	
129.5					

Field vane sensitivity values plotted: 2.2, 2.2, 2.5, 2.4, 2.1

Continued Next Page

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

RECORD OF BOREHOLE No N5

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847412.9 & Easting 315578.4) ORIGINATED BY RK

DIST ______ HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY BK

DATUM Geodetic DATE 12/7/2009 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
129.5 / 15.0	sandy	15	SS	1	129	3 40 36 21
	SILTY CLAY tr. to some sand, tr. gravel grey, stiff, wet				128	
		16	TW	PM		
					127	
		17	SS	1	126	
					125	
124.5 / 20.0	**SANDY SILT TILL** some clay grey, dense to v. dense, moist	18	SS	35		
					124	
		19	SS	100/35	123	15 34 36 15; auger grinding @ 21.6 and 22.5 m
	cobbles/boulders inferred				122	
		20	SS	100/33	121	auger grinding @ 23.6 m
119.8 / 24.7		21	SS	100 / 30	120	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane values (sensitivity): 2.2, 2.6, 2.9, 2.4

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

End of Borehole.
water level in borehole @ 23,2 m (not stabilized)* upon completion before auger pull out
Borehole caved-in @ 6,1 m after auger pull out
Piezometer installed to 24.4 m
Piezometer water level records :

Dec, 15, 2009	7.1 m
Dec. 16, 2009	6,9 m
Dec, 22, 2009	5 4 m
Jan. 06, 2010	5,3 m
Feb. 04, 2010	5.0 m

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

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

Ministry of Transportation
Ontario

Foundation Design

TRANETOB01245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N7

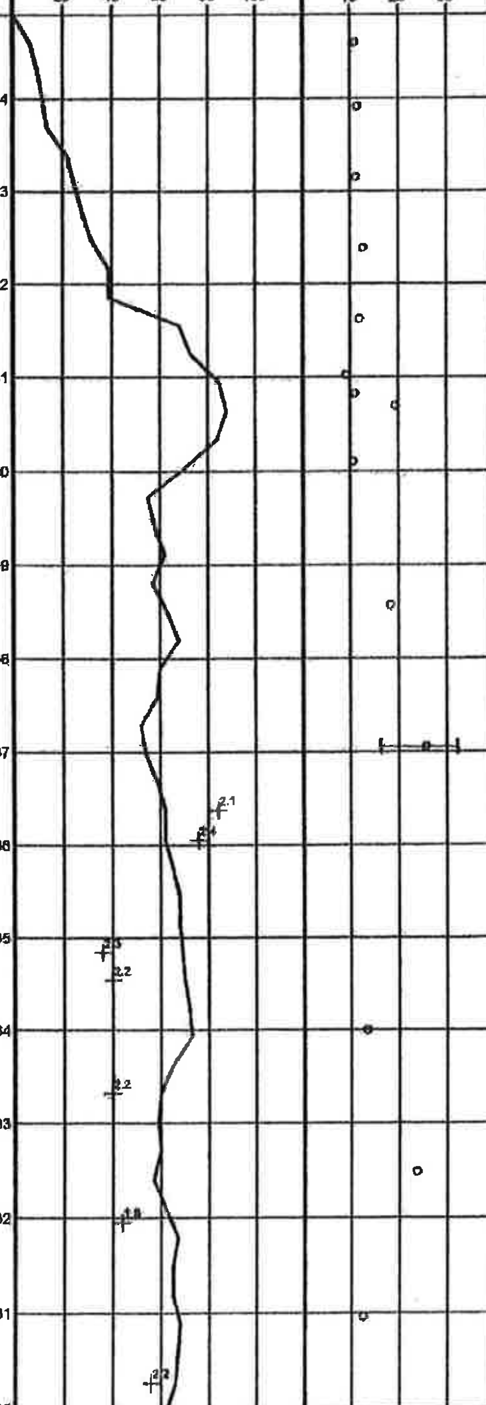
METRIC

GWP 2008-E-0012 — LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847408.6 & Easting 315519.3) — ORIGINATED BY RK

DIST ____ HWY 401 — BOREHOLE TYPE Hollow Stem Augers — COMPILED BY SK

DATUM Geodetic — DATE 12/15/2009 — CHECKED BY ZO

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
144.9 / 0.0	GROUND SURFACE					
	0.1 m Topsoil	1	SS	8		
	FILL: SANDY SILT TO SILTY SAND some clay brown, loose to compact, moist	2	SS	27	144	
	clayey	3	SS	20	143	
		4	SS	19	142	
		5	SS	23	141	
	peaty topsoil	6	SS	25		
140.5 / 4.4	SAND tr. to some silt brown, compact, wet	7	SS	21	140	1 83 (16)
139.6 / 5.3	SILT some fine sand, tr. to some clay brown, dilatant, compact, wet				139	spoon wet
		8	SS	24		
138.2 / 6.8	SILTY CLAY tr. sand, tr. gravel grey, wet				138	
		9	SS	5	137	
	stiff				136	
	firm	10	TW	PM		
					135	
		11	SS	2	134	
					133	
		12	SS	1	132	
		13	SS	1	131	
	stiff					
129.9					130	



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$+^3, \times^3$ Numbers refer to Sensitivity — 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

TRANETOB01245AA: HWY401/Leslie St/Ramp N/W

RECORD OF BOREHOLE No N7

METRIC

GWP 2008-E-0012 LOCATION N-W Ramp of Leslie Street to 401 (Northing 4847406.6 & Easting 315519.3) ORIGINATED BY RK

DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SK

DATUM Geodetic DATE 12/15/2009 CHECKED BY ZO

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
129.9 / 15.0	SILTY CLAY tr. gravel, tr. sand grey, stiff, wet	14	TW	PM	129	
		15	SS	9	128, 127	
126.9 / 18.0	CLAYEY SILT TILL tr. sand grey, hard, moist	16	SS	38	126	1 24 48 27
		17	SS	50	125	
124.3 / 20.6	SILTY SAND TILL cobbles/boulders inferred grey, v. dense, wet	18	SS	100 / 20 mm	124, 123	soil back-up 0,6 m; 9 50 32 9; auger grinding @ 22,3 m
		19	SS	100 / 20 mm	122, 121	soil back-up 0,5 m; auger grinding @ 23,2 m and 23.5 m
120.4 / 24.5		20	SS	100 / 30 mm		

End of Borehole.
Water level @ 13.1 m (not stabilized)* upon completion before auger pull out
Borehole caved-in @ 7.0 m after auger pull out
Dynamic Cone Penetration Test (DCPT) performed adjacent to borehole from ground surface to 18.7 m

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane values: 2.1, 2.2

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

UNIT WEIGHT γ kN/m³

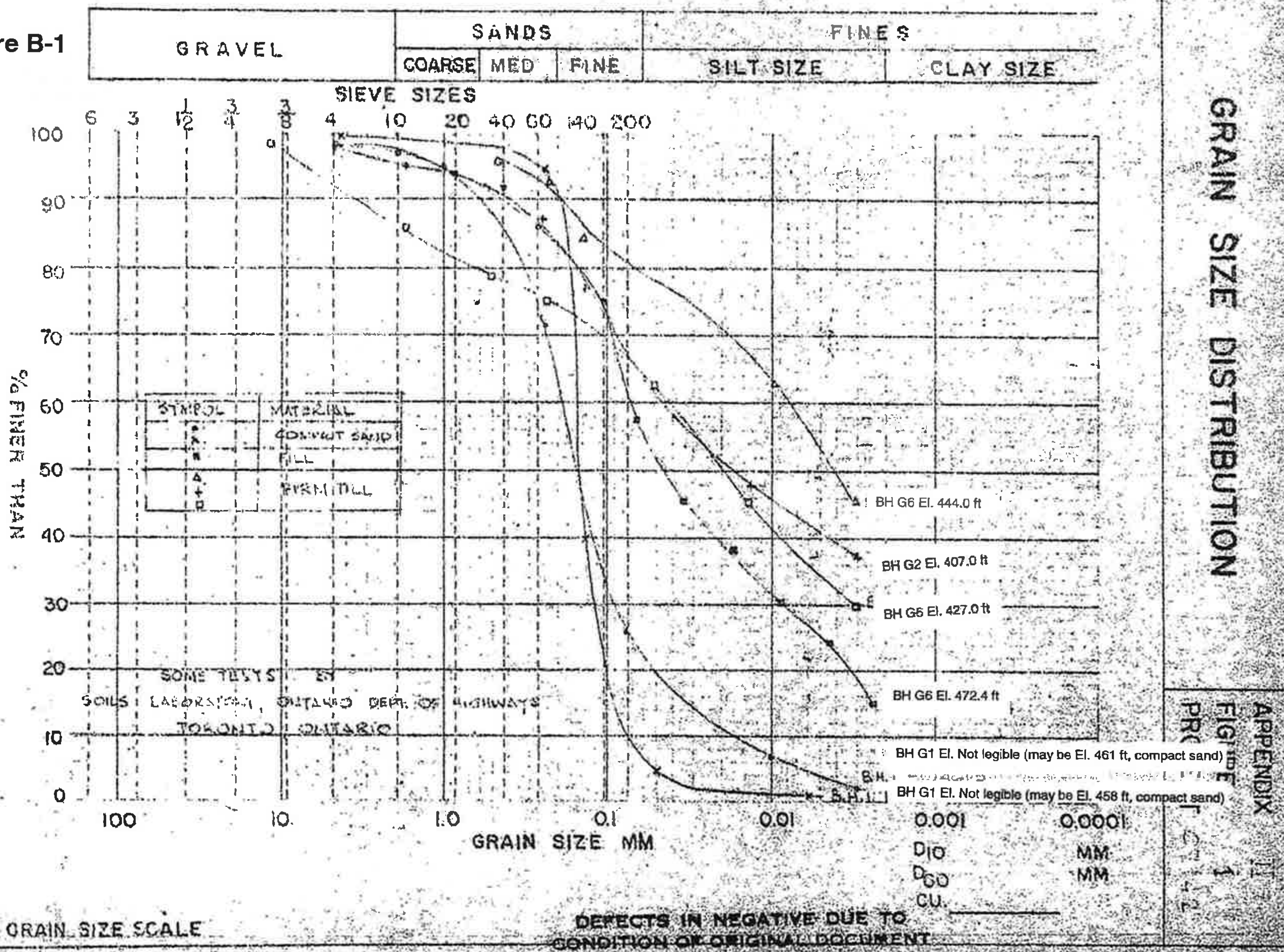
+³, ×³: Numbers refer to Sensitivity

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

Appendix B

Test Results

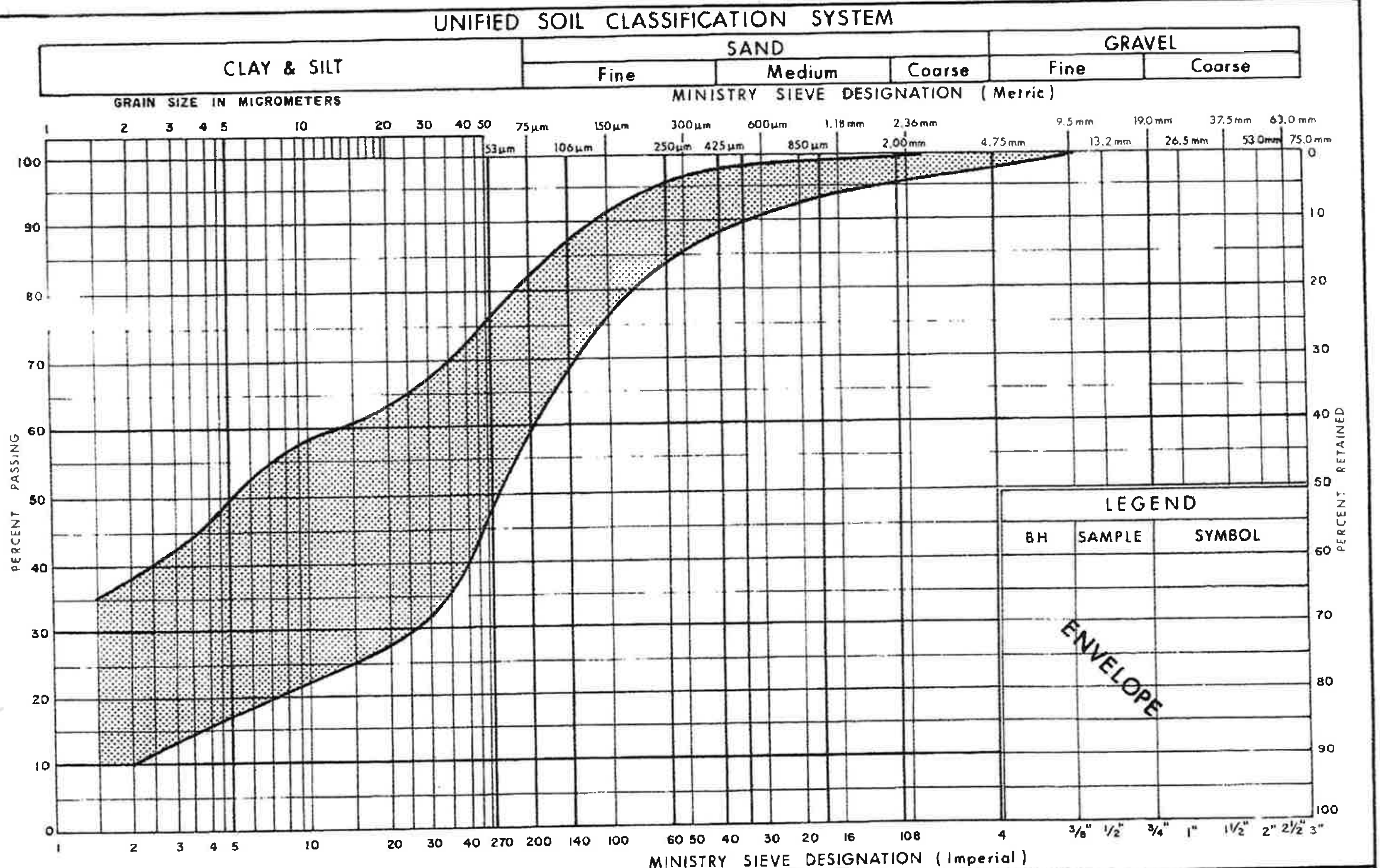
Figure B-1



GRAIN SIZE DISTRIBUTION
APPENDIX
FIGURE 1
GRAVEL
SANDS
COARSE
MED
FINE
FINES
SILT SIZE
CLAY SIZE
SIEVE SIZES
% FINER THAN
GRAIN SIZE MM
SYMBOL
MATERIAL
COMPACT SAND
FILL
FIRM FILL
SOME TESTS BY
SOILS LABORATORY, ONTARIO DEPT. OF HIGHWAYS
TORONTO, ONTARIO
BH G6 El. 444.0 ft
BH G2 El. 407.0 ft
BH G6 El. 427.0 ft
BH G6 El. 472.4 ft
BH G1 El. Not legible (may be El. 461 ft, compact sand)
BH G1 El. Not legible (may be El. 458 ft, compact sand)
D10 MM
D60 MM
CU.
MIT. GRAIN SIZE SCALE
DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

Figure B-2

78 12 M



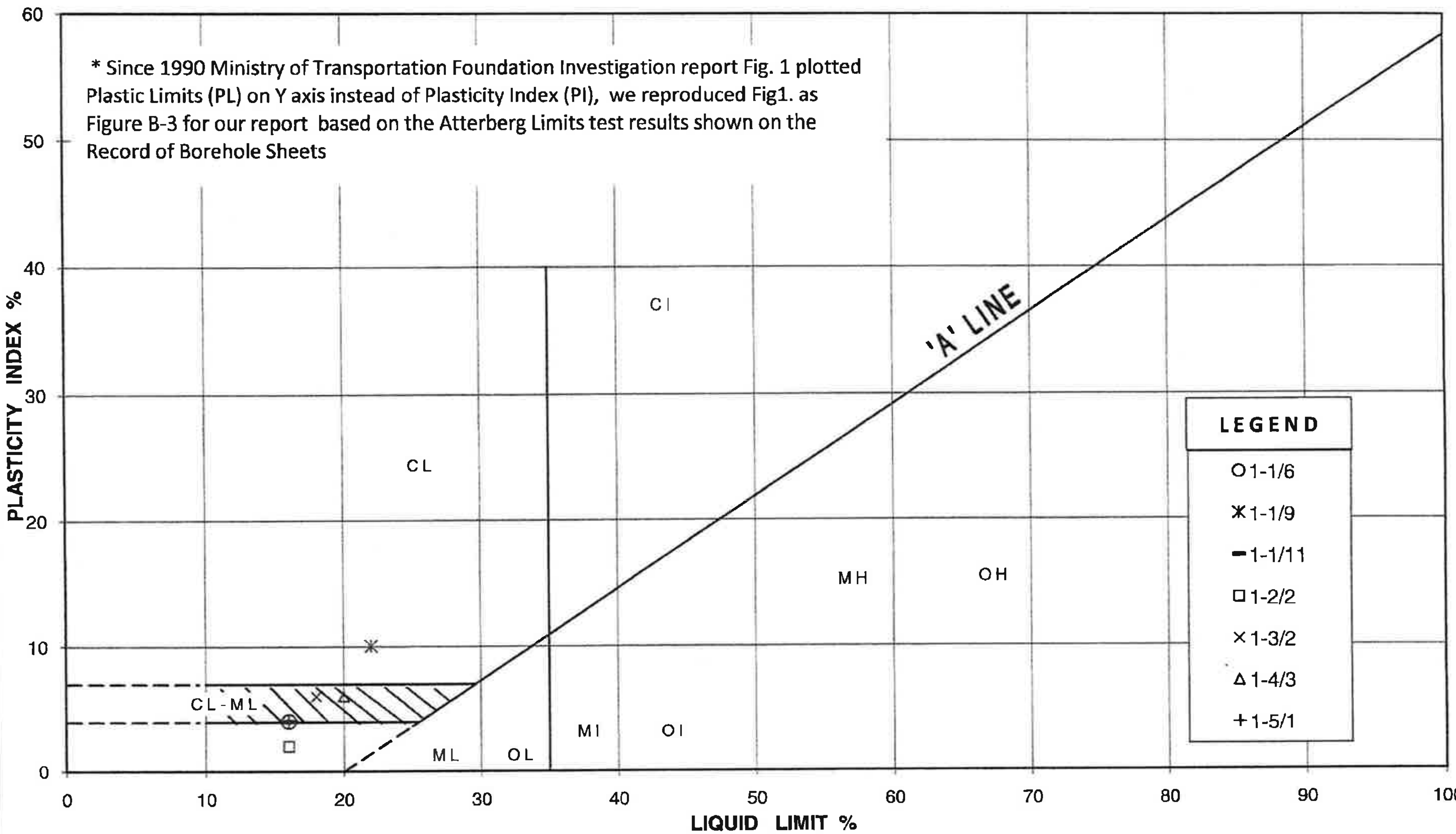
Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION
SANDY SILT (FILL)

FIG No 2

W P 260-86-01/A



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

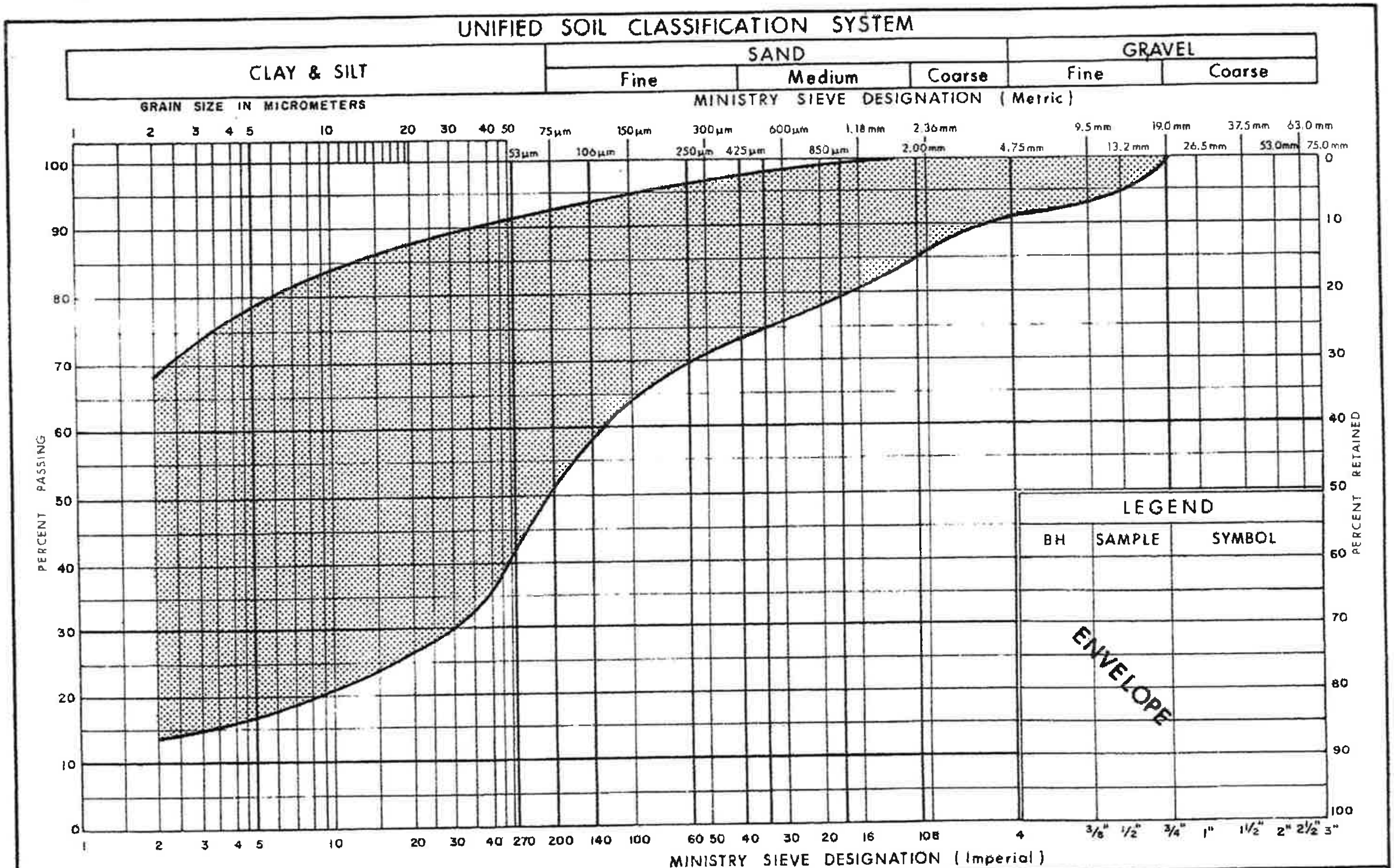
PLASTICITY CHART

SANDY SILT FILL (ONLY FINE PORTION TESTED)

FIGURE No.	B-3
REF. No.	TRANETOB01245AA
DATE	

Figure B-4

78 12 M



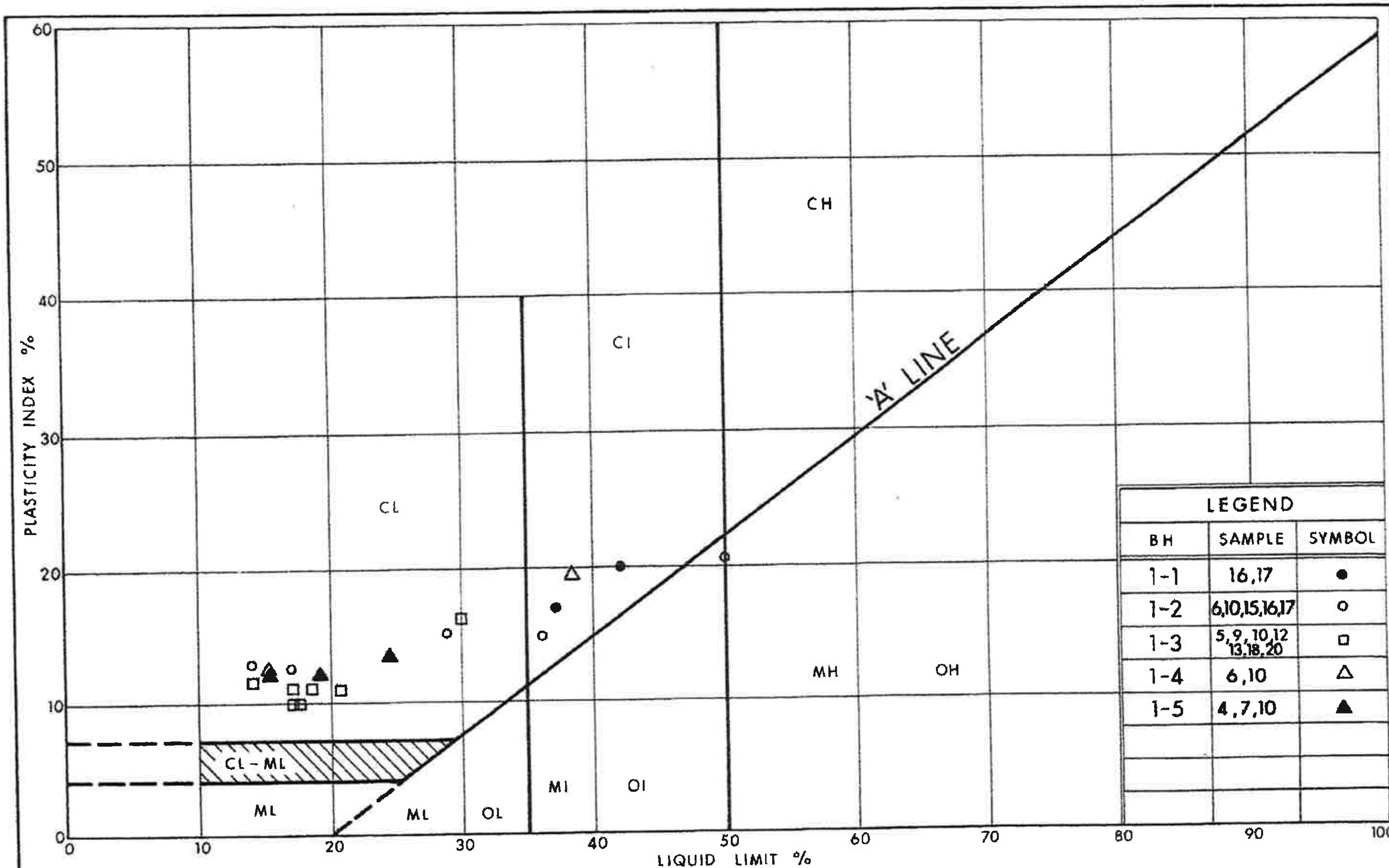
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY
(SOME SAND, TRACE GRAVEL)

FIG No 4
W P 260-86-01/A

Figure B-5

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

CLAYEY SILT TO SILTY CLAY

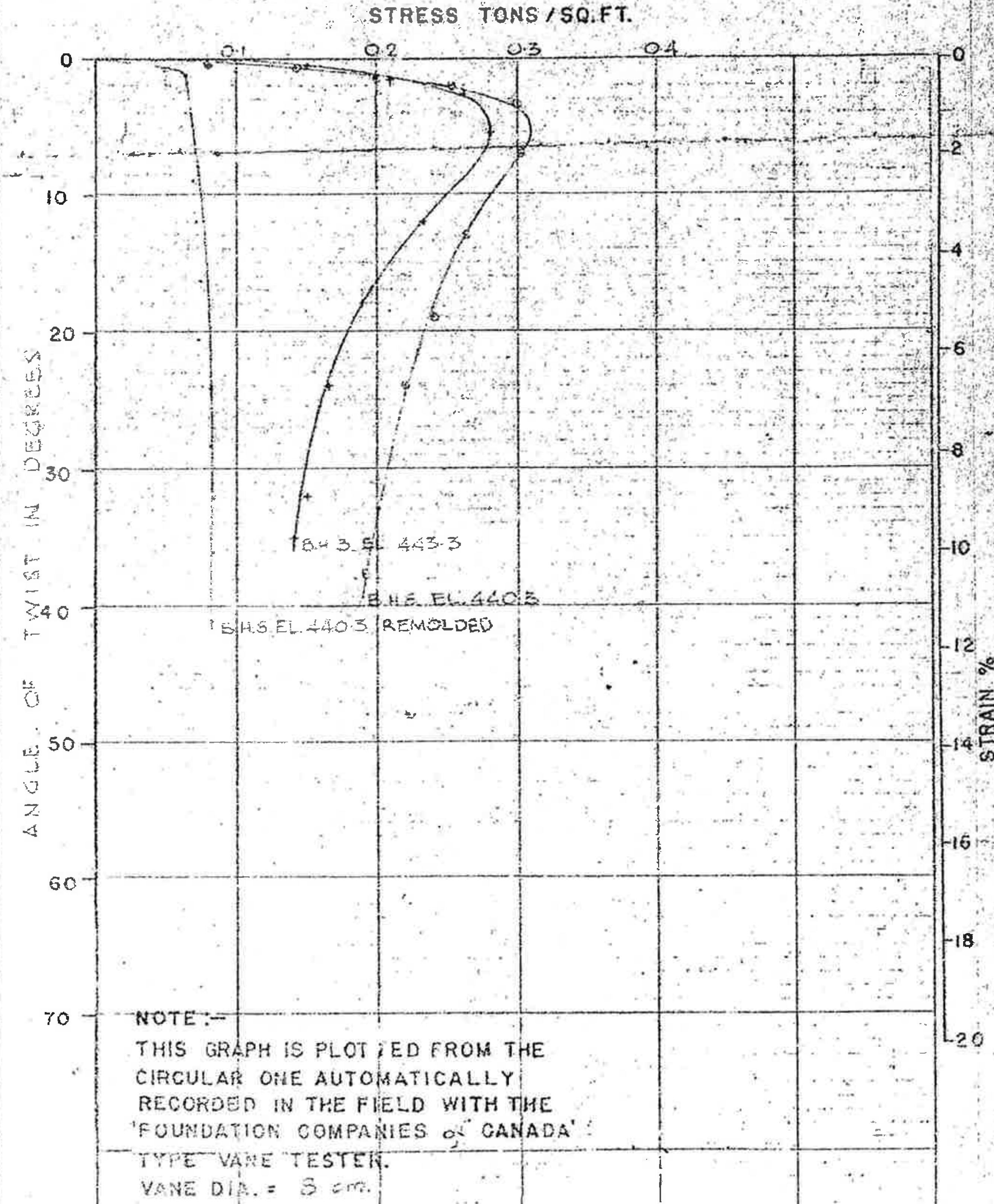
FIG No 3

W P 260-86-01/A

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

APPENDIX II
FIGURE 2
PROJECT C7142

Figure B-6

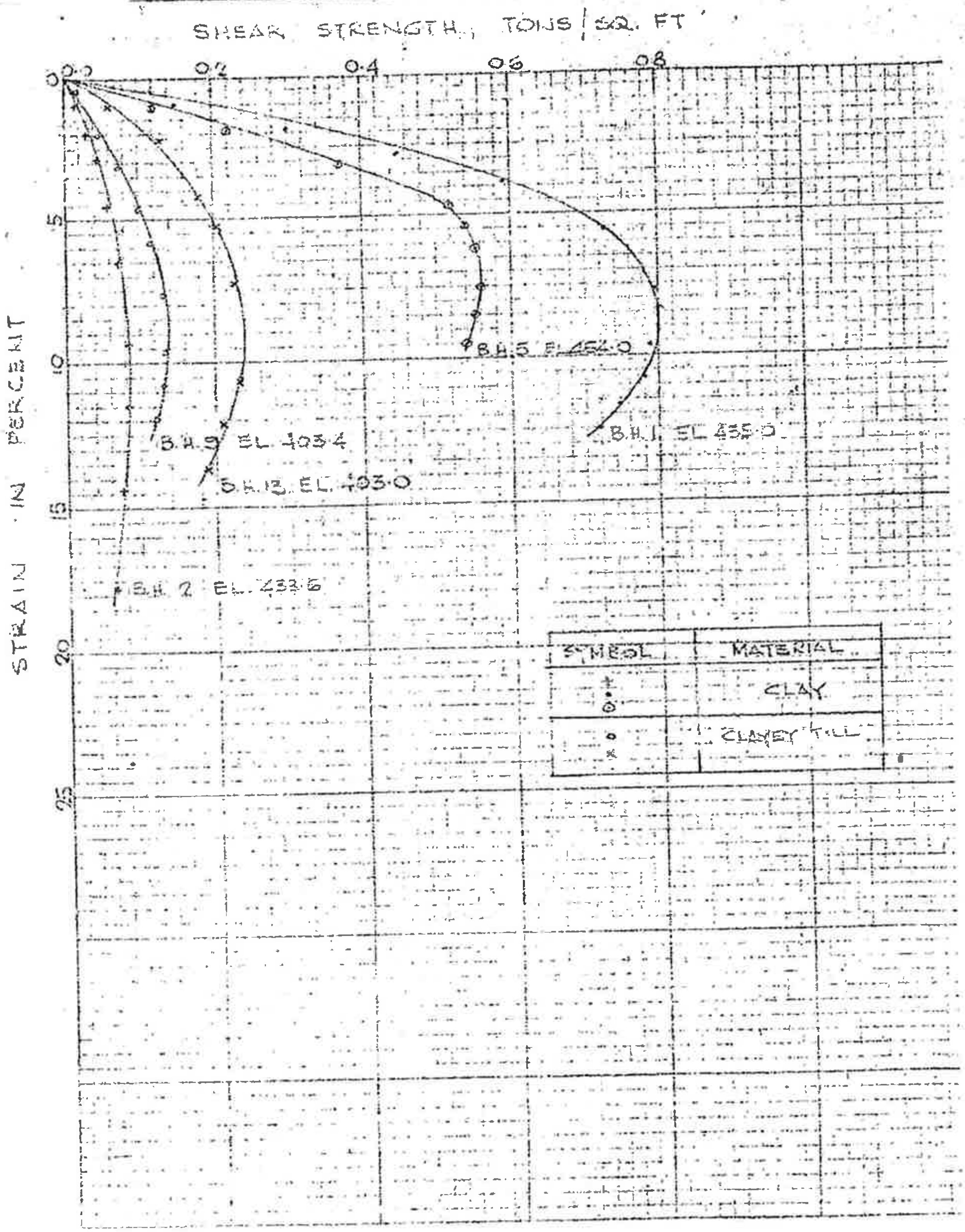


UNCONFINED COMPRESSION TESTS

TYPICAL STRESS-STRAIN CURVES

APPENDIX II
FIGURE 3
PROJECT C7142

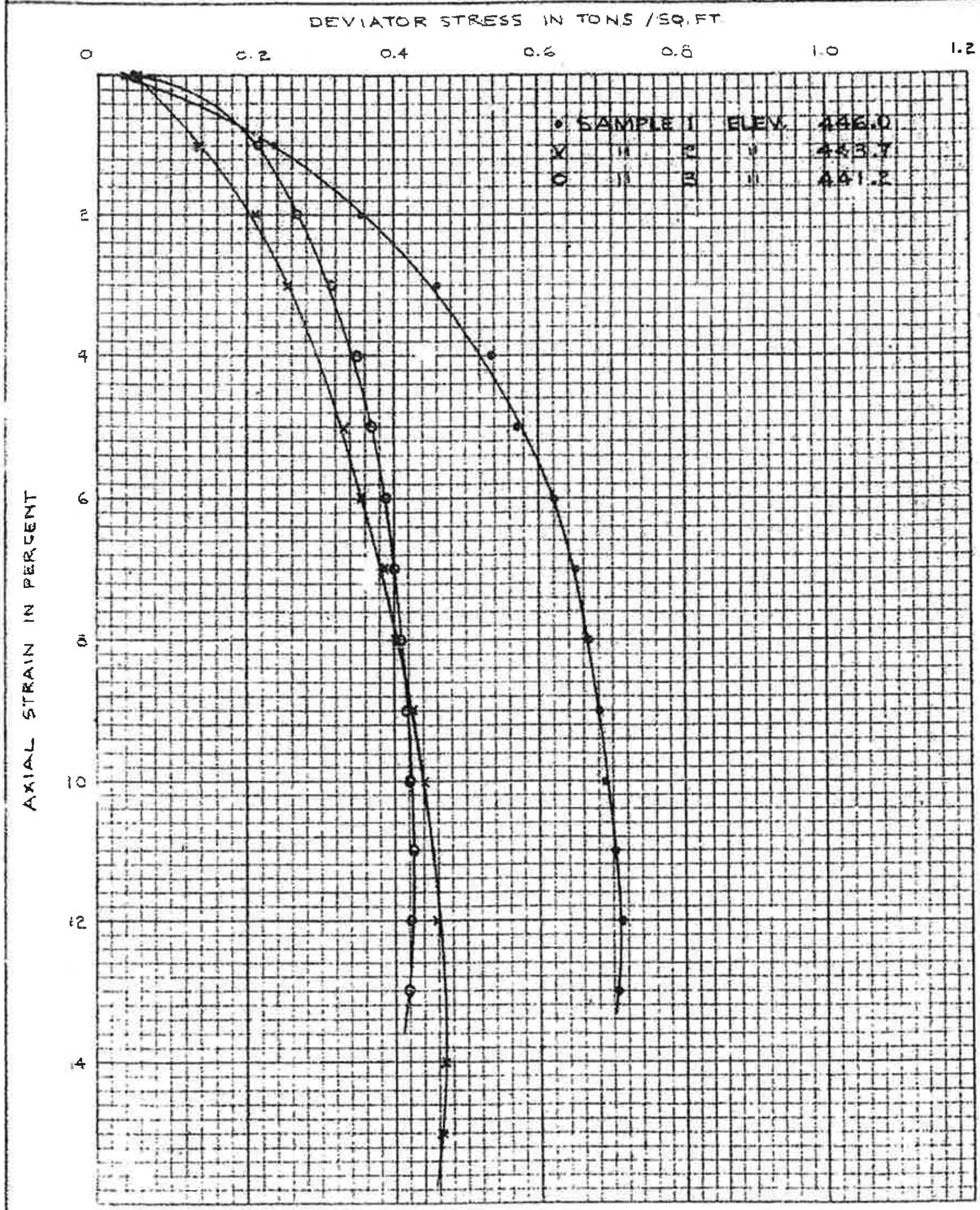
Figure B-7



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

UNDRAINED TRIAXIAL TESTS STRESS - STRAIN CURVES SOFT SILTY CLAY BOREHOLE 3 & 3A	APPENDIX II FIGURE I PROJECT S7002

Figure B-8



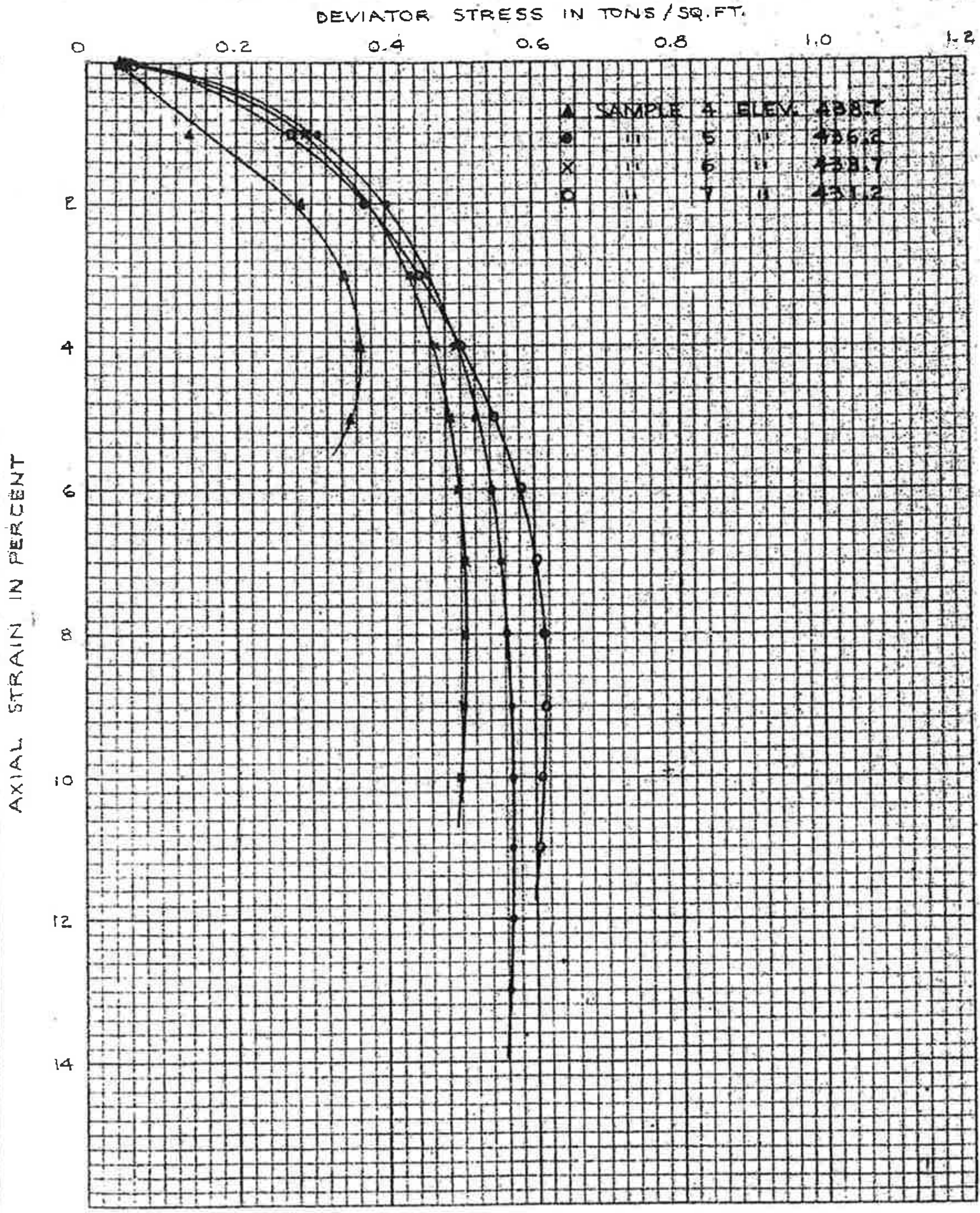
GEOCON

UNDRAINED TRIAXIAL TESTS
STRESS - STRAIN CURVES

Figure B-9

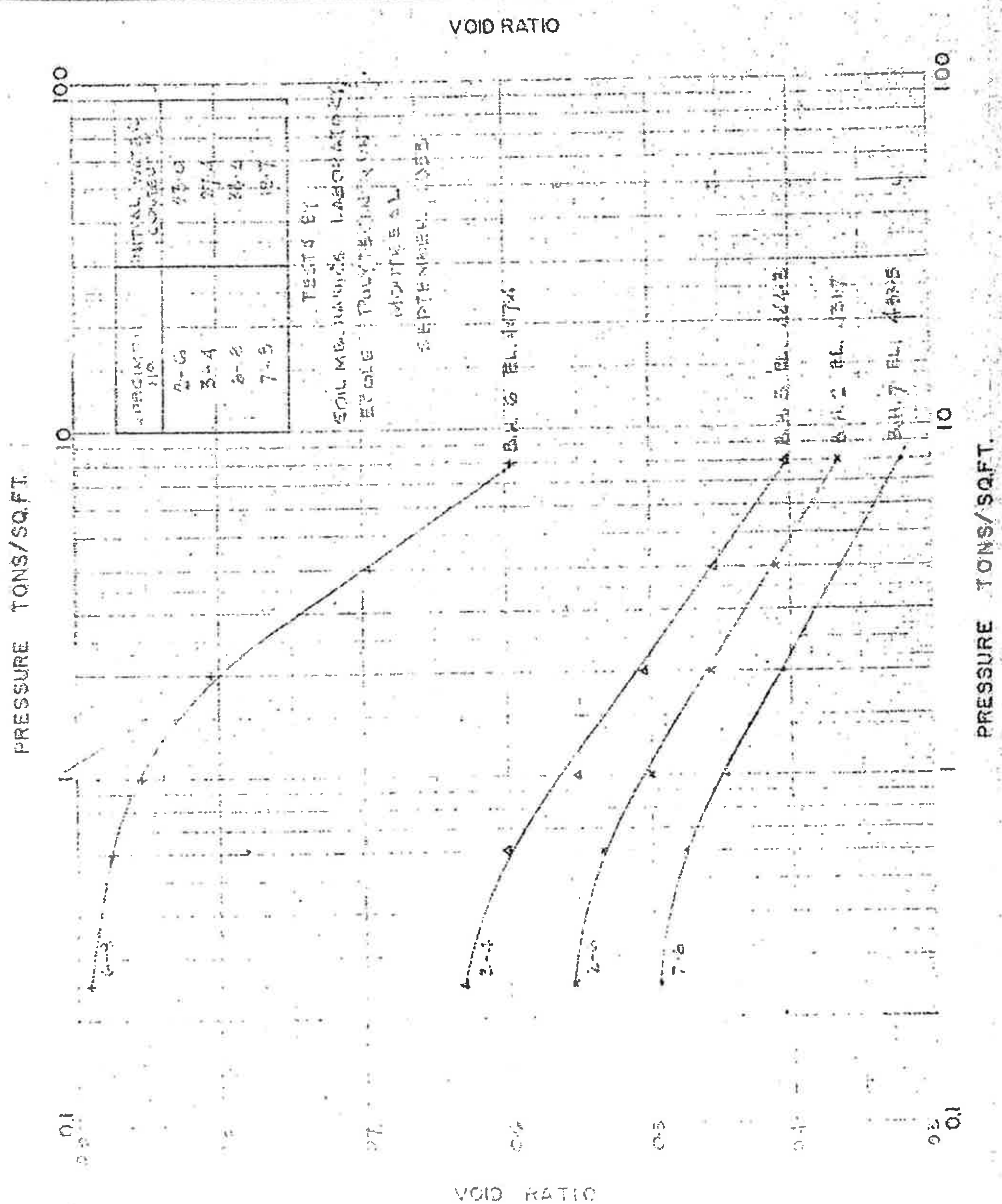
SOFT SILTY CLAY
BOREHOLE 3 & 3A

APPENDIX	II
FIGURE	2
PROJECT	S7002



GEOCON

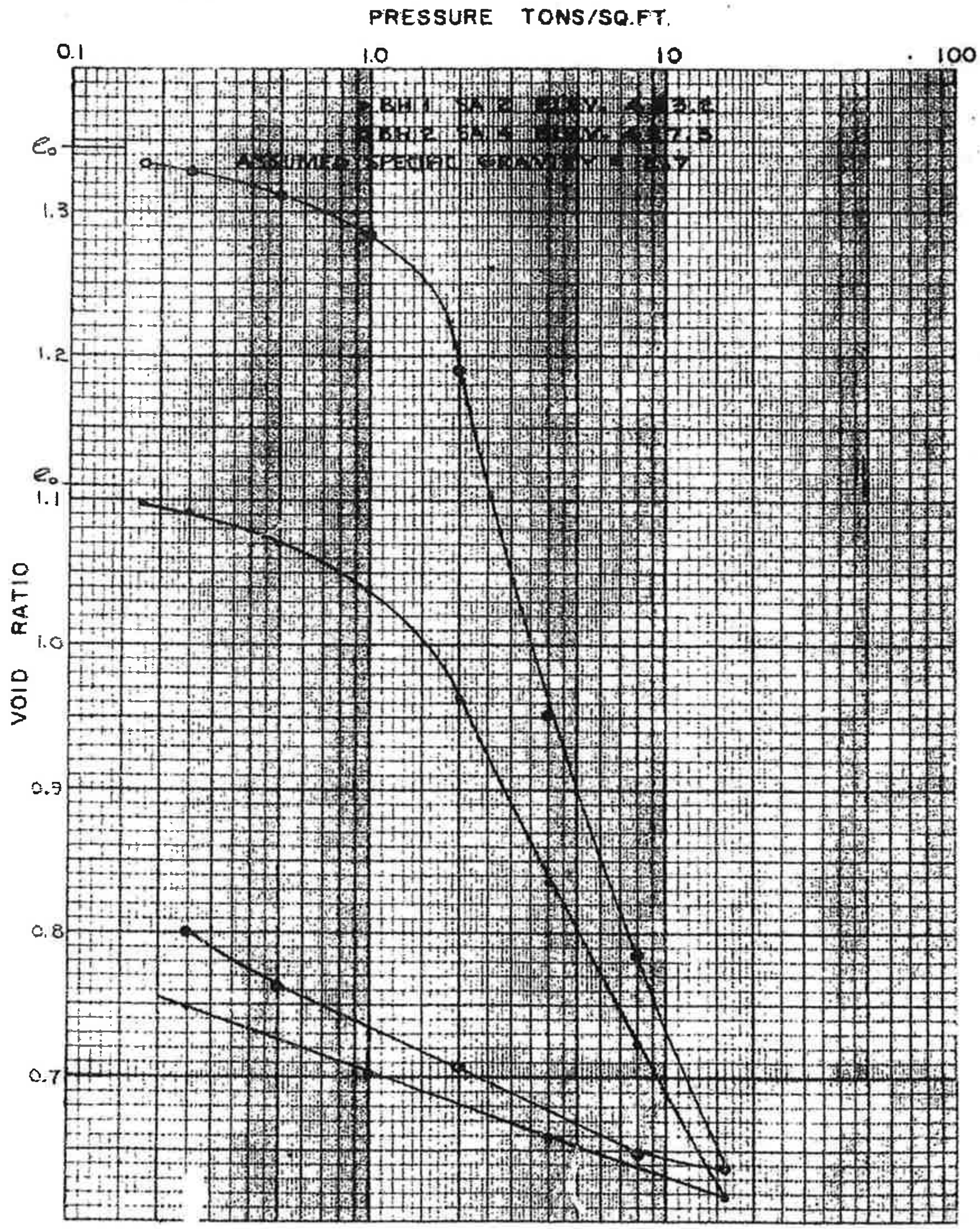
VOID RATIO-PRESSURE DIAGRAM CONSOLIDATION TEST	APPENDIX II
Figure B-10 ON THE SOFT CLAY	FIGURE 4
	PROJECT C714



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST	APPENDIX II FIGURE 3 PROJECT 57002

Figure B-11



GEOCON

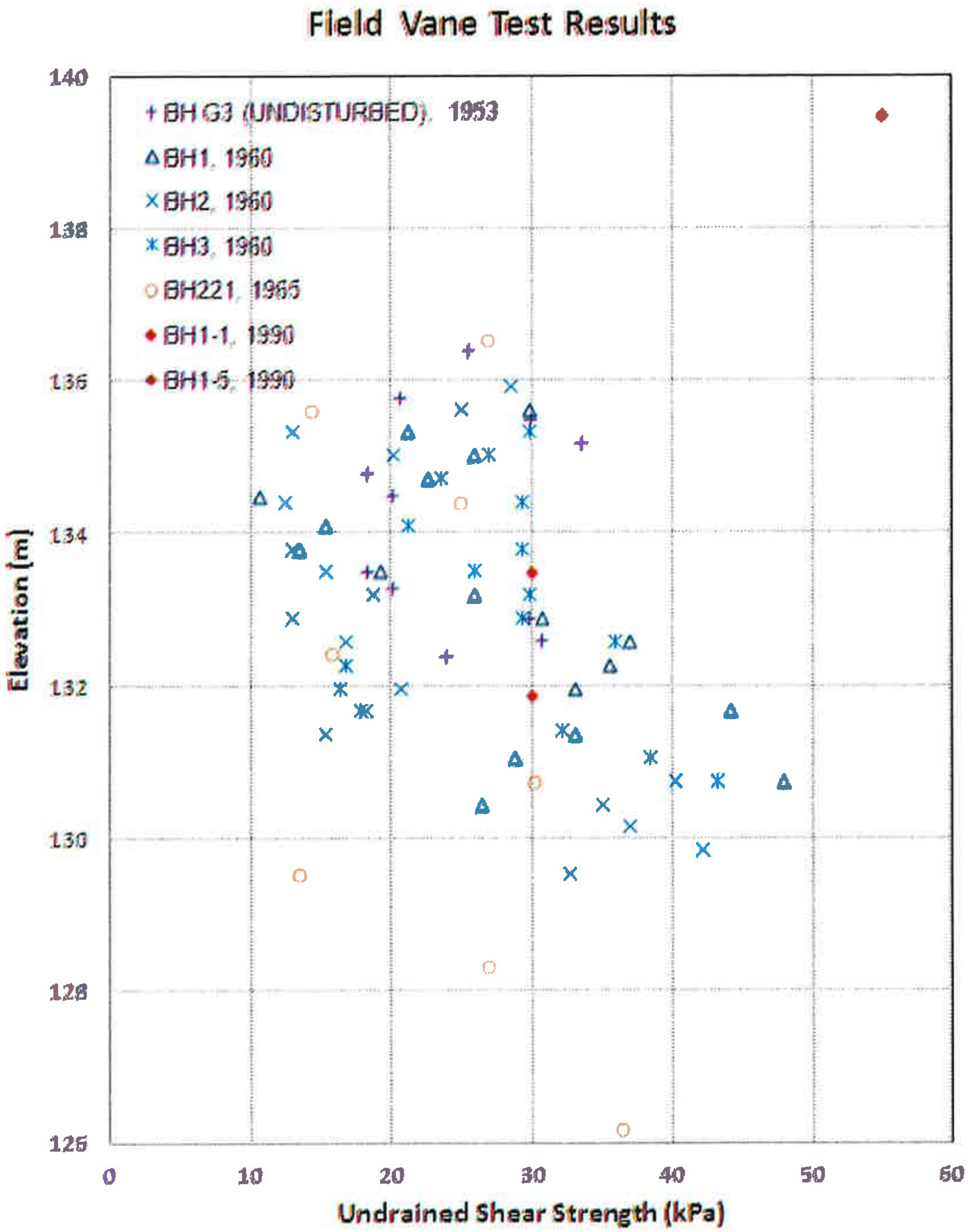
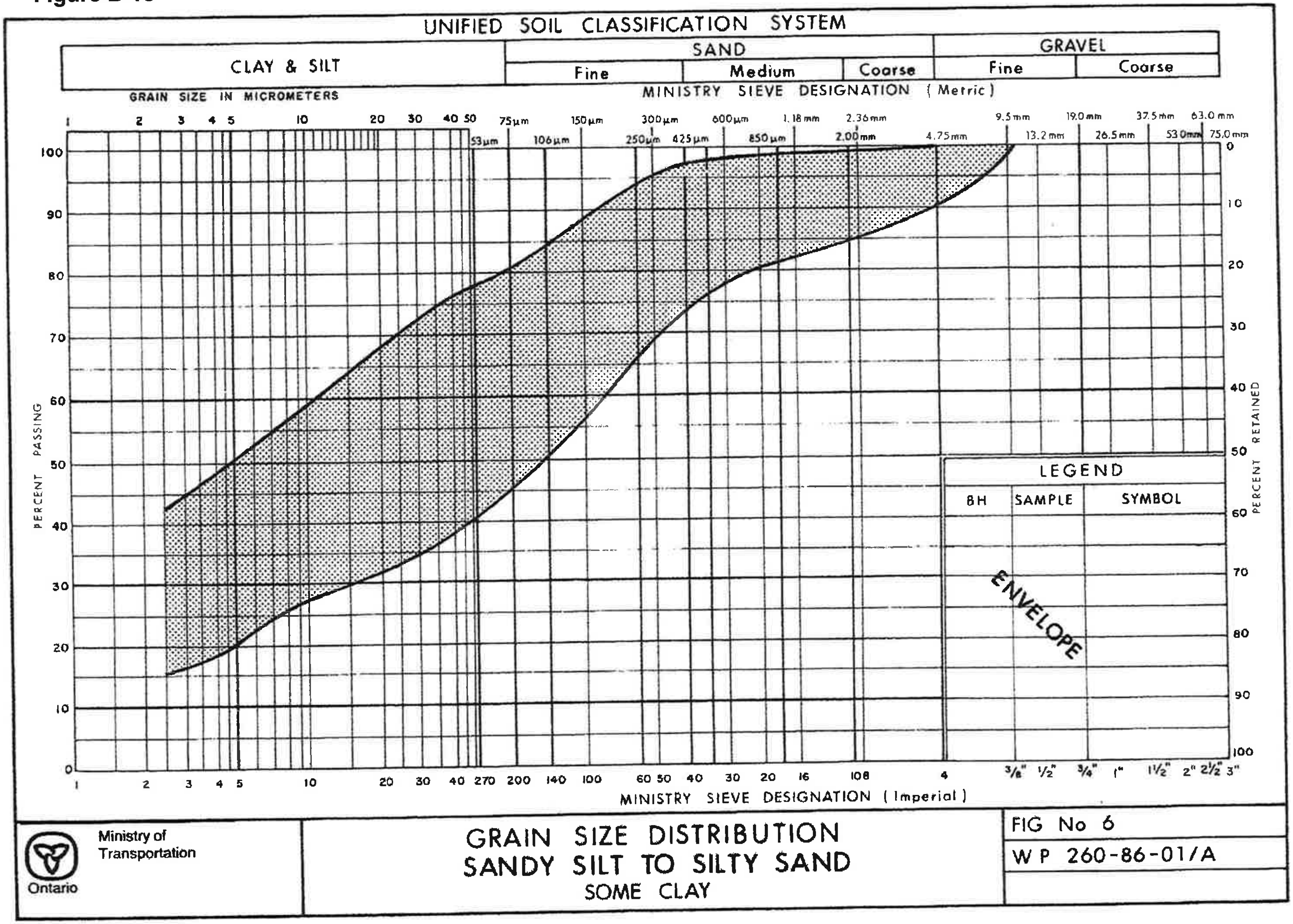


Figure B-12 Undrained Shear Strengths vs Elevations

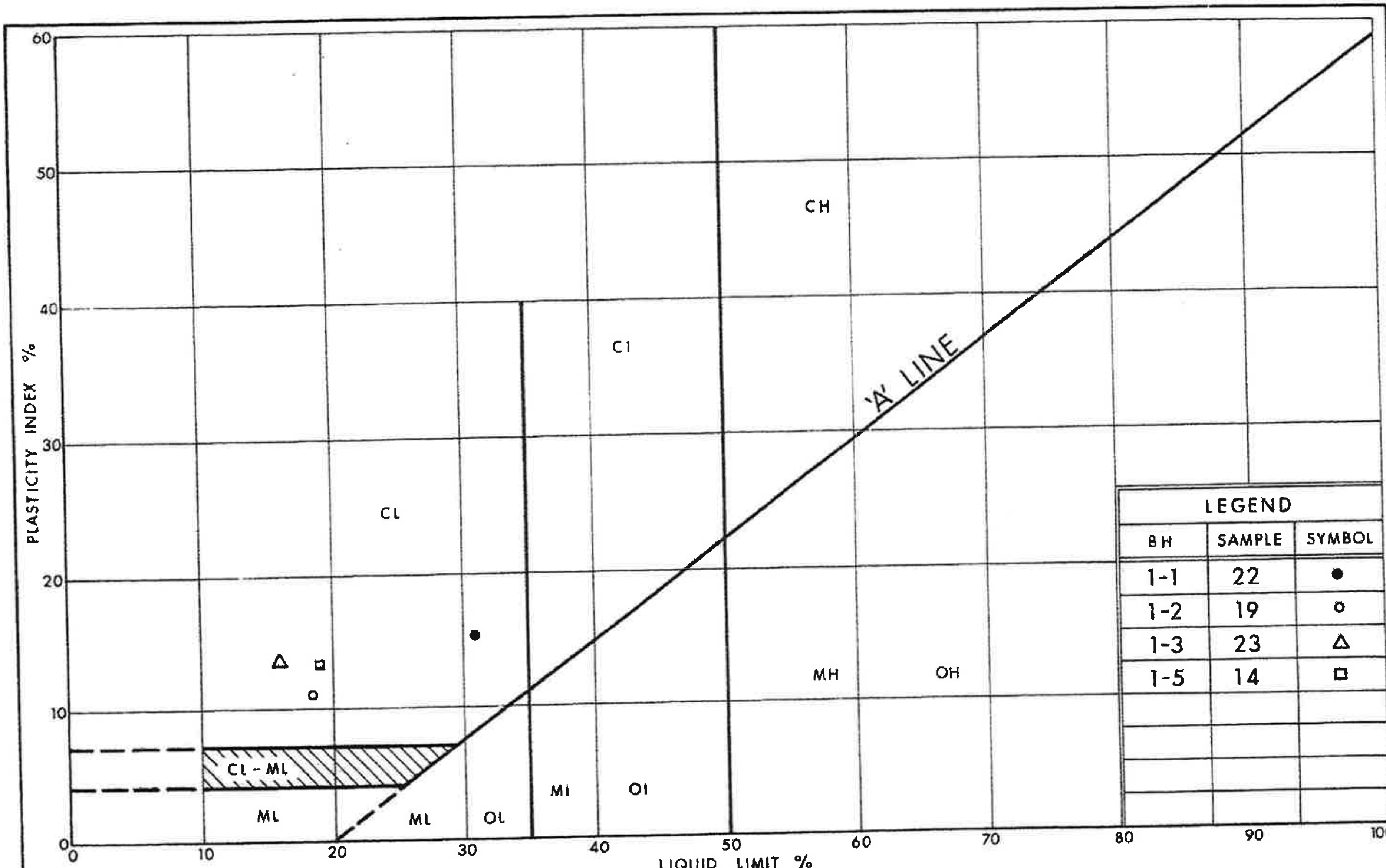
Figure B-13



78 12 M
UNIFIED SOIL CLASSIFICATION SYSTEM
CLAY & SILT
SAND
Fine
Medium
Coarse
GRAVEL
Fine
Coarse
GRAIN SIZE IN MICROMETERS
MINISTRY SIEVE DESIGNATION (Metric)
75µm 150µm 300µm 600µm 1.18 mm 2.36 mm 9.5 mm 19.0 mm 37.5 mm 63.0 mm
53µm 106µm 250µm 425µm 850µm 2.00 mm 4.75 mm 13.2 mm 26.5 mm 53.0 mm 75.0 mm
PERCENT PASSING
PERCENT RETAINED
LEGEND
BH
SAMPLE
SYMBOL
ENVELOPE
MINISTRY SIEVE DESIGNATION (Imperial)
270 200 140 100 60 50 40 30 20 16 10 8 4
3/8" 1/2" 3/4" 1" 1 1/2" 2" 2 1/2" 3"
Ministry of Transportation
Ontario
GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILTY SAND
SOME CLAY
FIG No 6
W P 260-86-01/A

Figure B-14

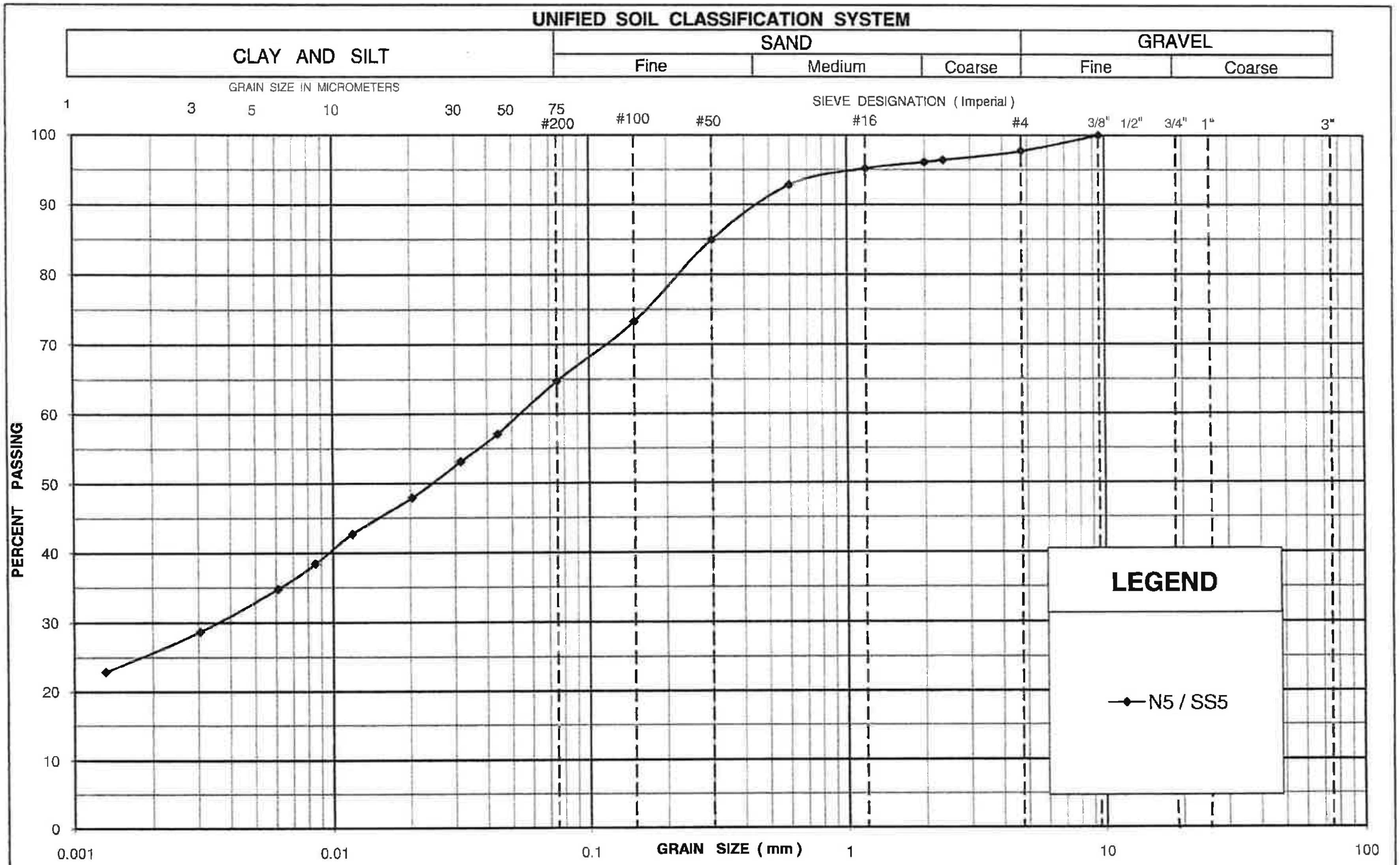
Oct 75, FF-5-21



Ministry of Transportation
Ontario

PLASTICITY CHART
SANDY SILT TO SILTY SAND
SOME CLAY

FIG No 5
W P 260-86-01/A



coffey geotechnics SPECIALISTS MANAGING THE EARTH

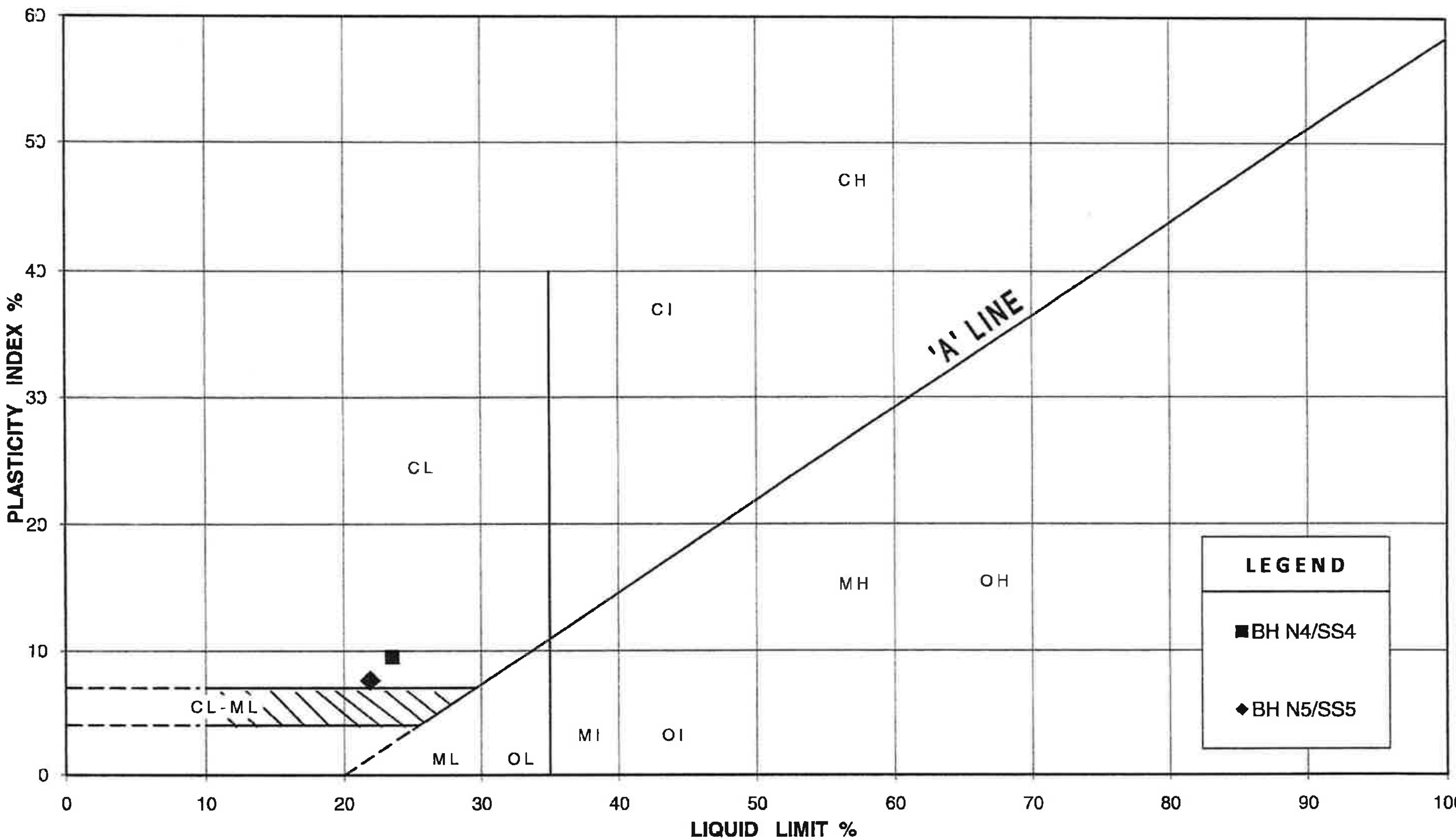
GRAIN SIZE DISTRIBUTION

FILL: Cohesive Zone within the Fill

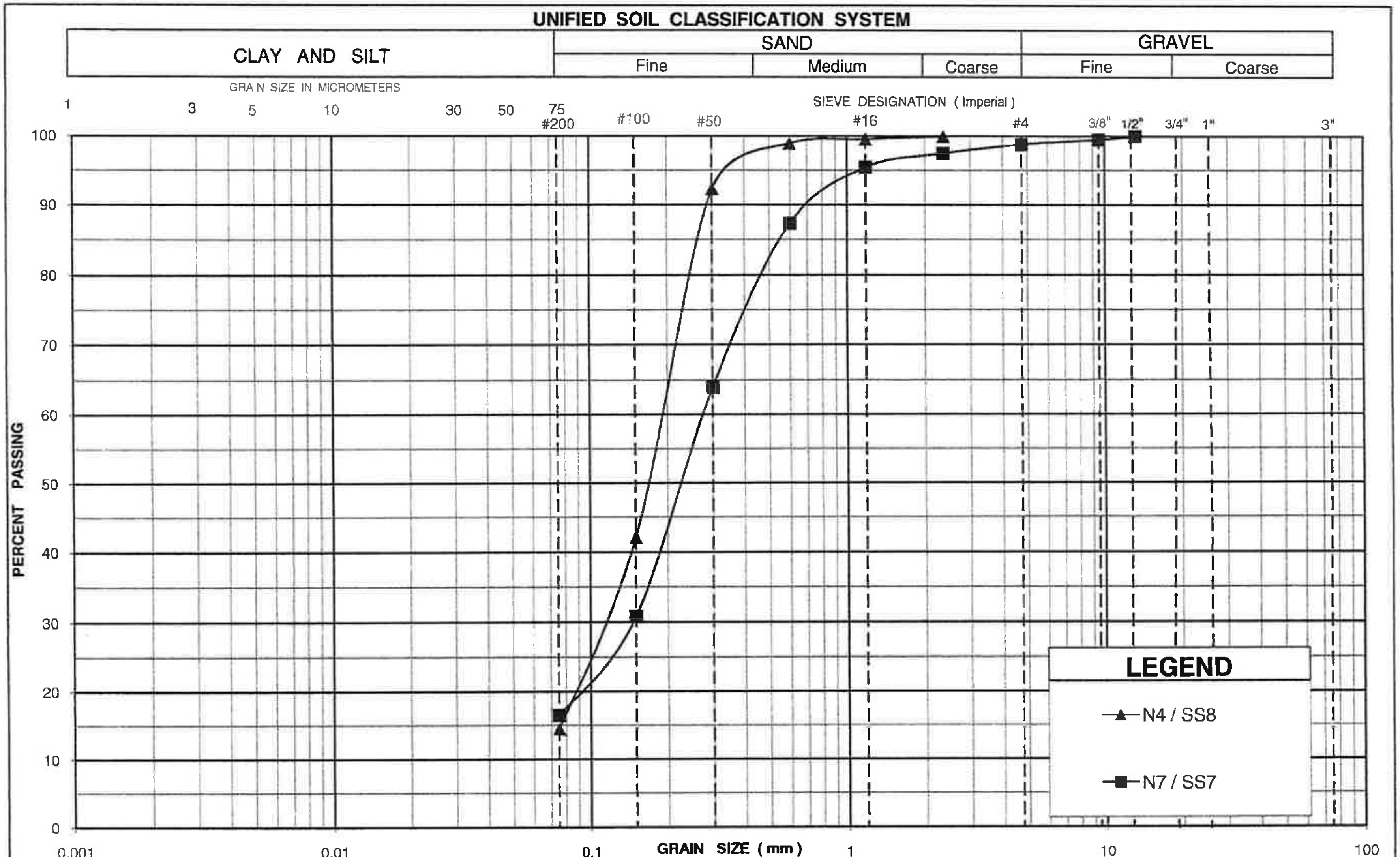
FIGURE NO.: B-15

PROJECT NO: TRANETOB01245AA

DATE: March, 2010



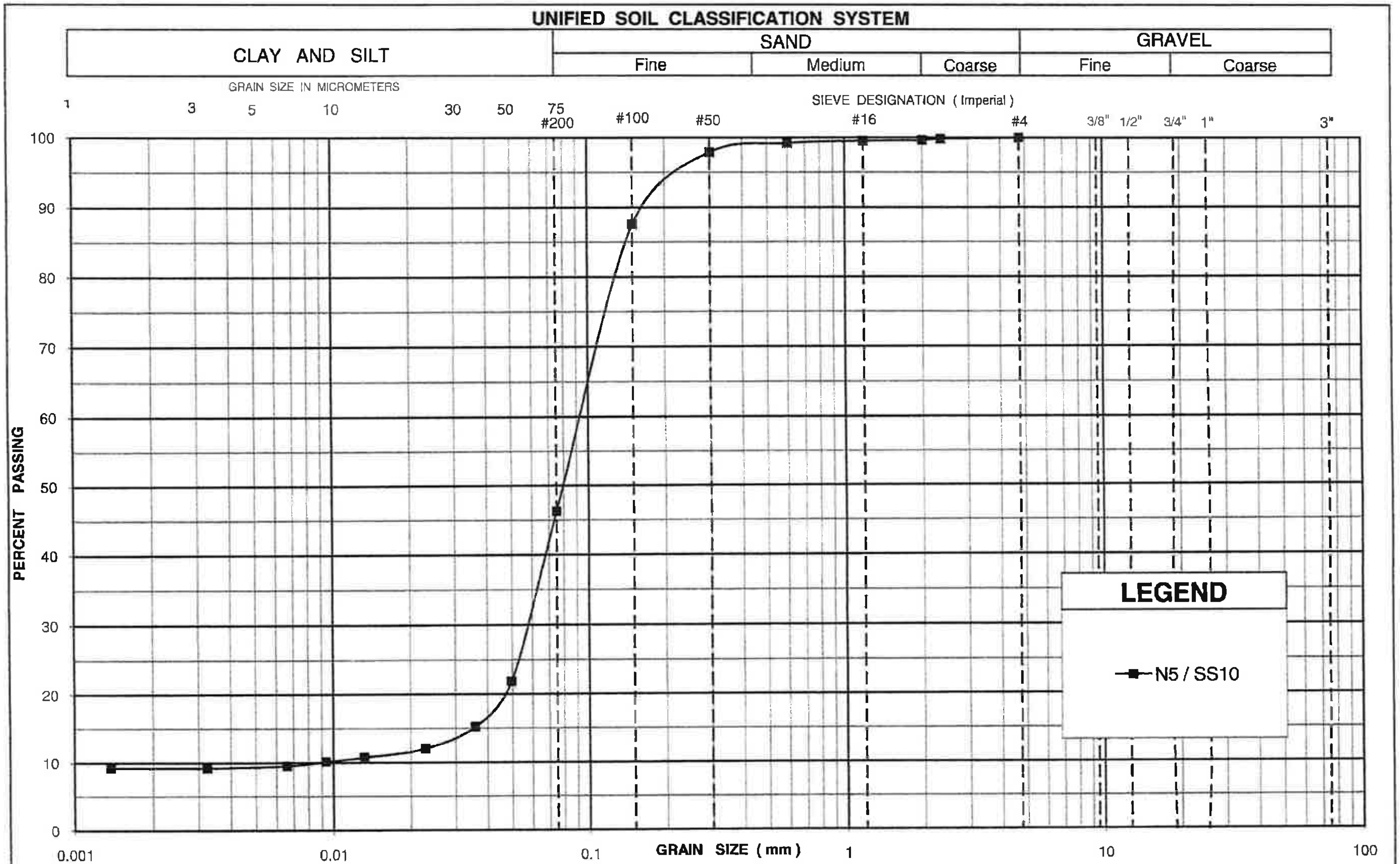
coffey geotechnics SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART FILL : Sandy Silt to Silty Sand with Clay	FIGURE No. B-16 REF. No. TRANETOB01245AA DATE MARCH, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SAND

FIGURE NO.: B-17
PROJECT NO: TRANETOB01245AA
DATE: MAR. 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

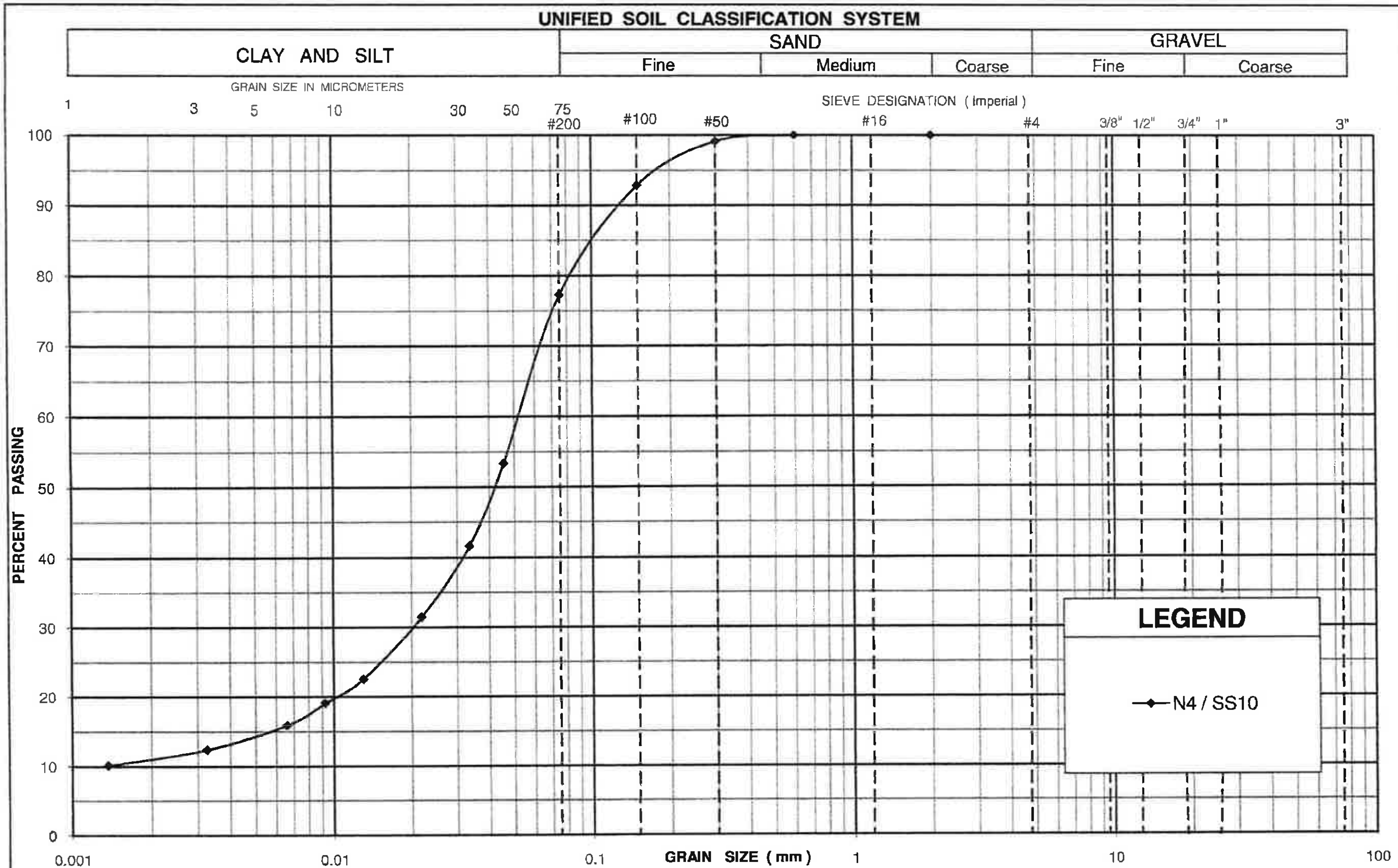
GRAIN SIZE DISTRIBUTION

SILTY SAND TO SANDY SILT

FIGURE NO.: B-18

PROJECT NO: TRANETOB01245AA

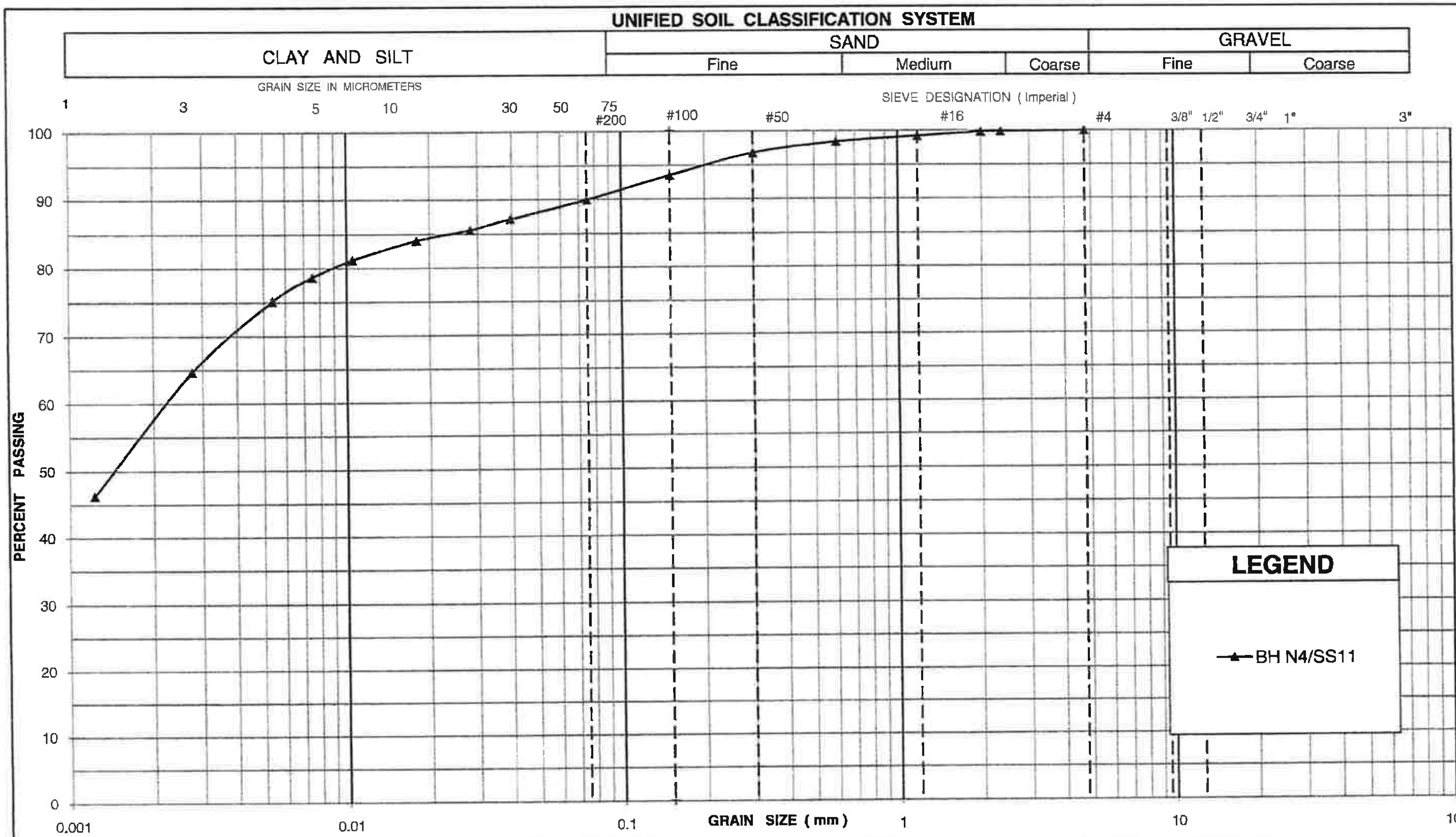
DATE: MARCH, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SILT

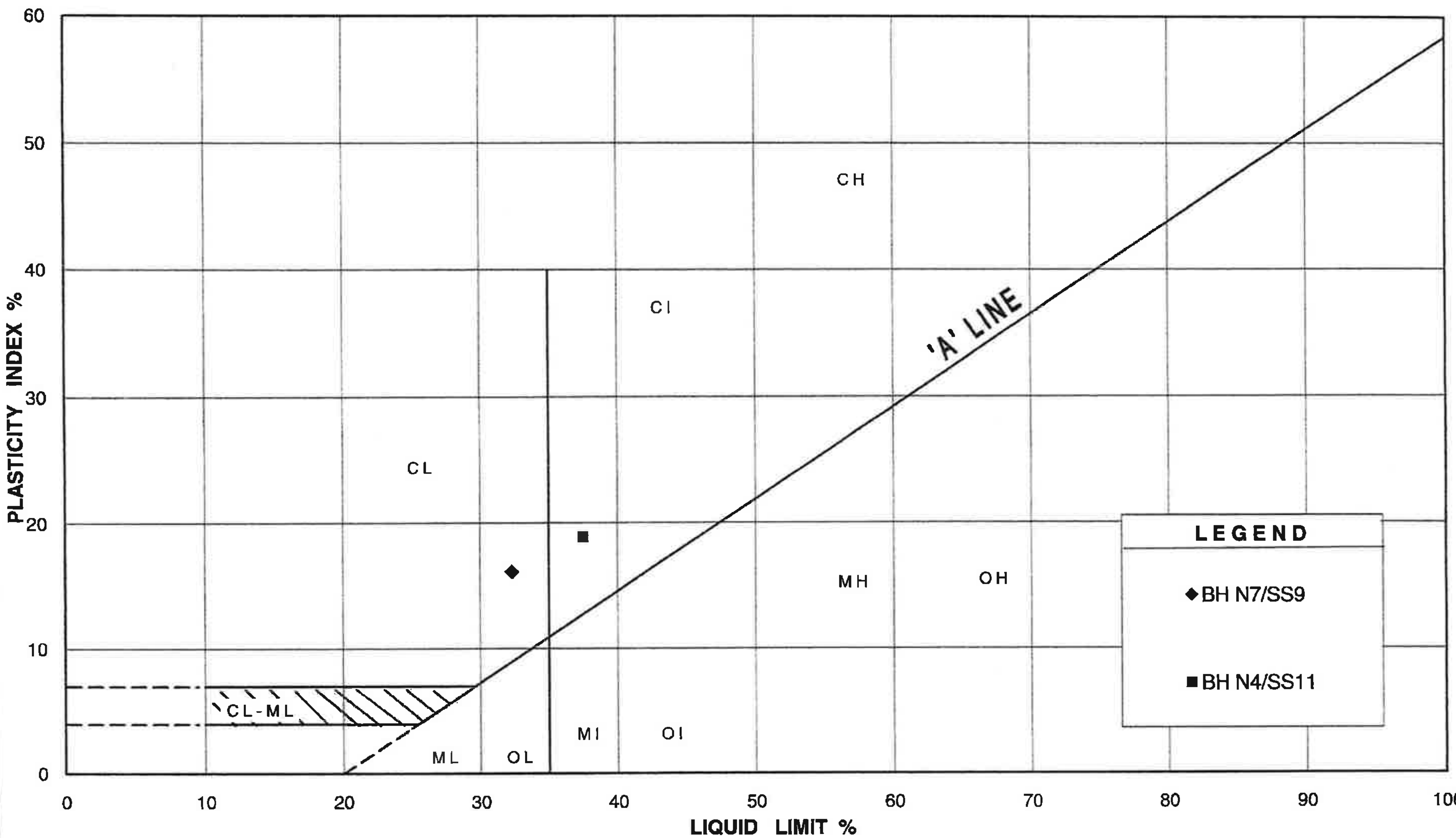
FIGURE NO.: B-19
PROJECT NO: TRANETOB01245AA
DATE: MARCH, 2010



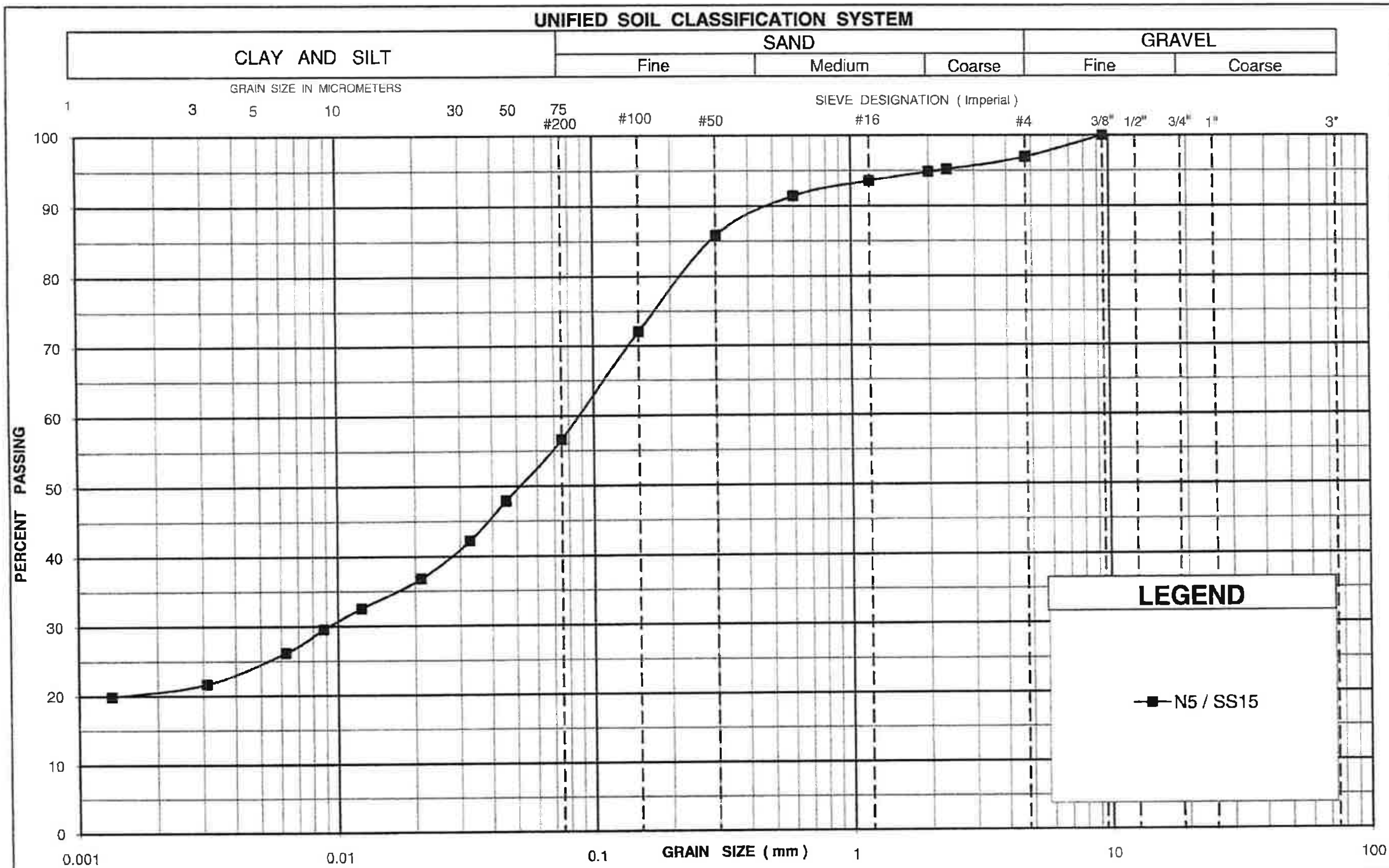
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SILTY CLAY

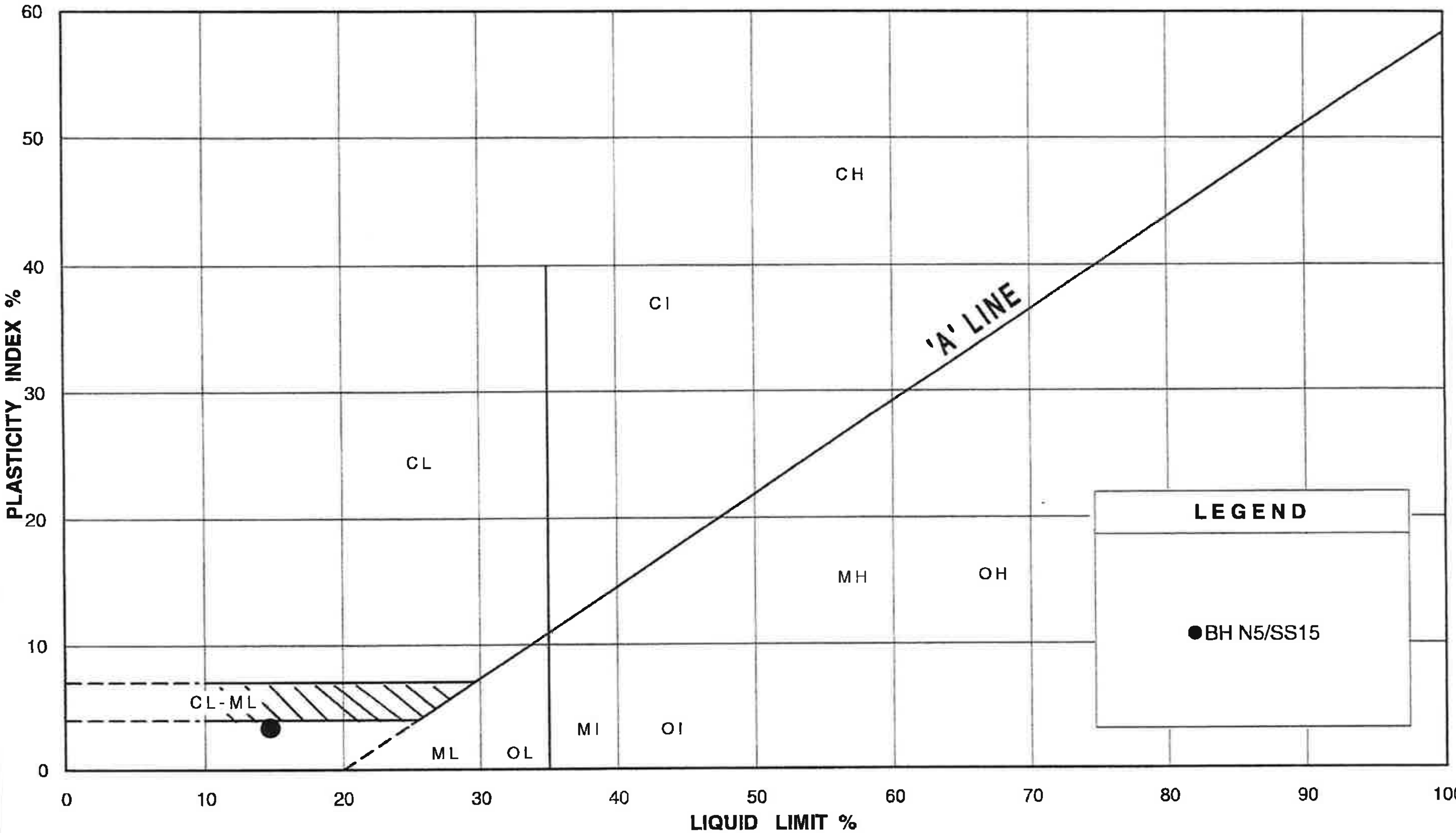
FIGURE NO.: B-20
PROJECT NO: TRANETOB01245AA
DATE: MARCH, 2010



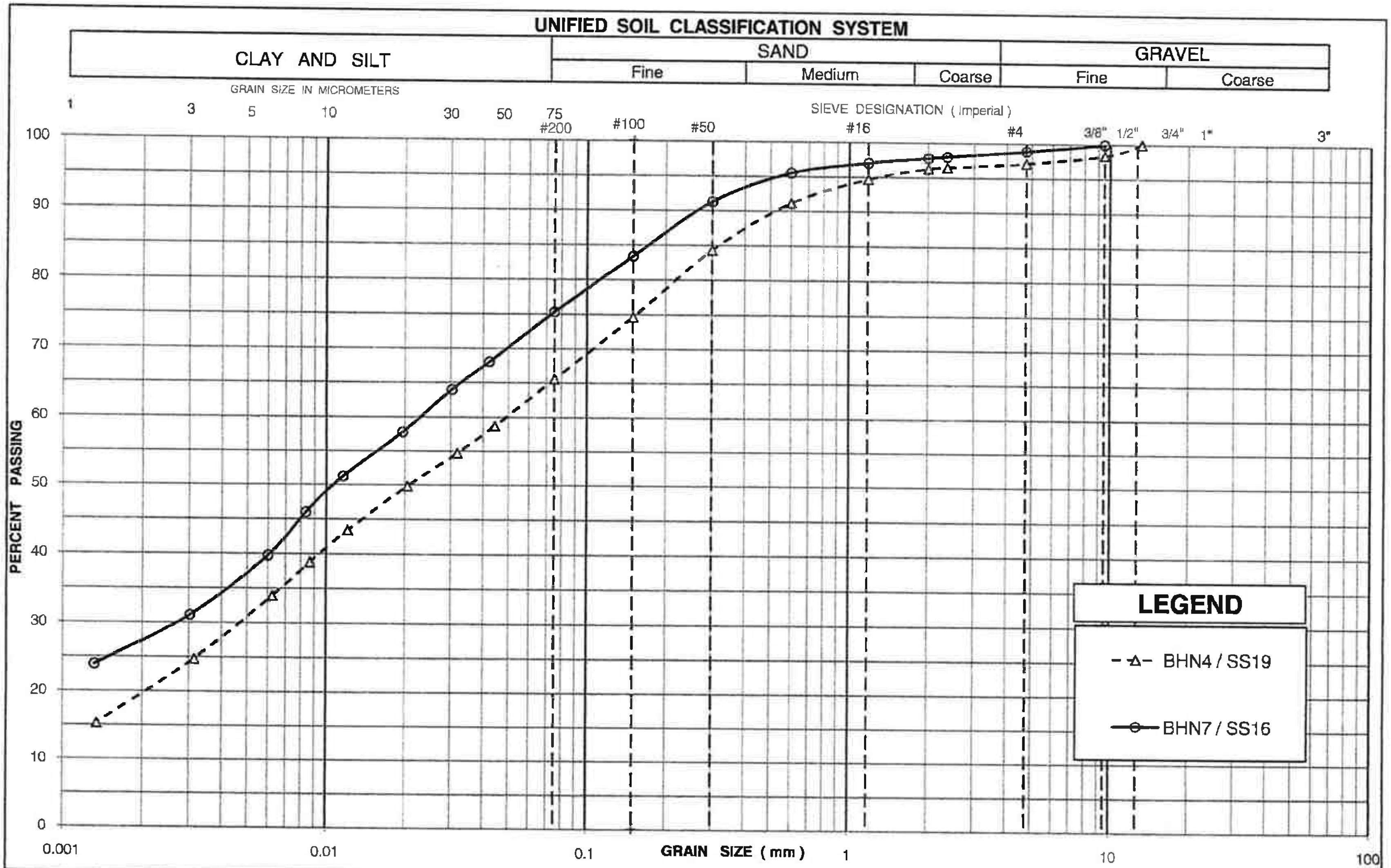
coffey geotechnics SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART SILTY CLAY	FIGURE No. B-21 REF. No. TRANETOB01245AA DATE MARCH, 2010



coffey geotechnics SPECIALISTS MANAGING THE EARTH	GRAIN SIZE DISTRIBUTION LESS PLASTIC ZONE WITHIN SILTY CLAY DEPOSIT	FIGURE NO.: B-22 PROJECT NO: TRANETOB01245AA DATE: March, 2010



coffey geotechnics SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART LESS PLASTIC ZONE WITHIN SILTY CLAY	FIGURE No. B-23 REF. No. TRANETOB01245AA DATE MARCH, 2010

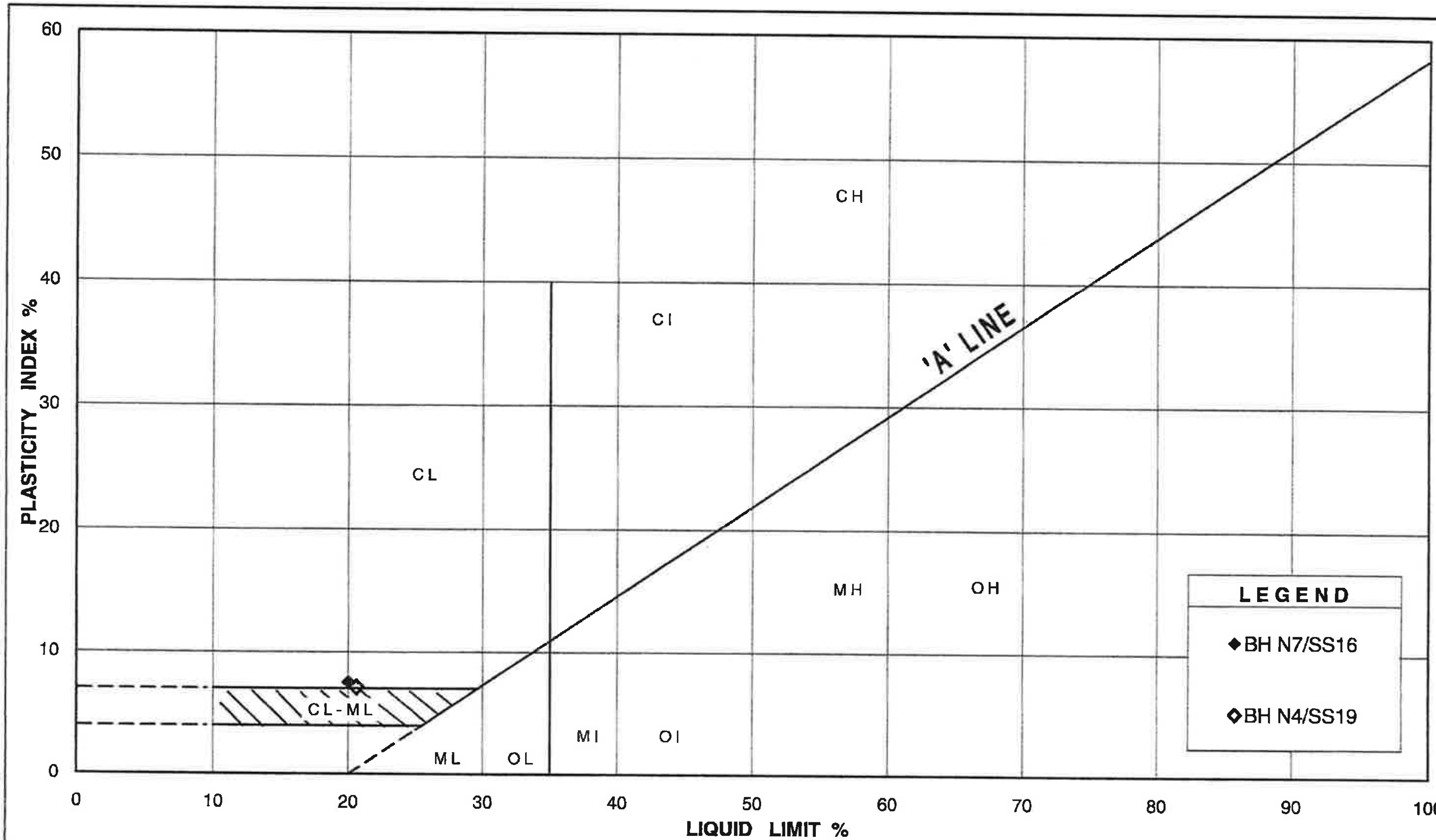


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

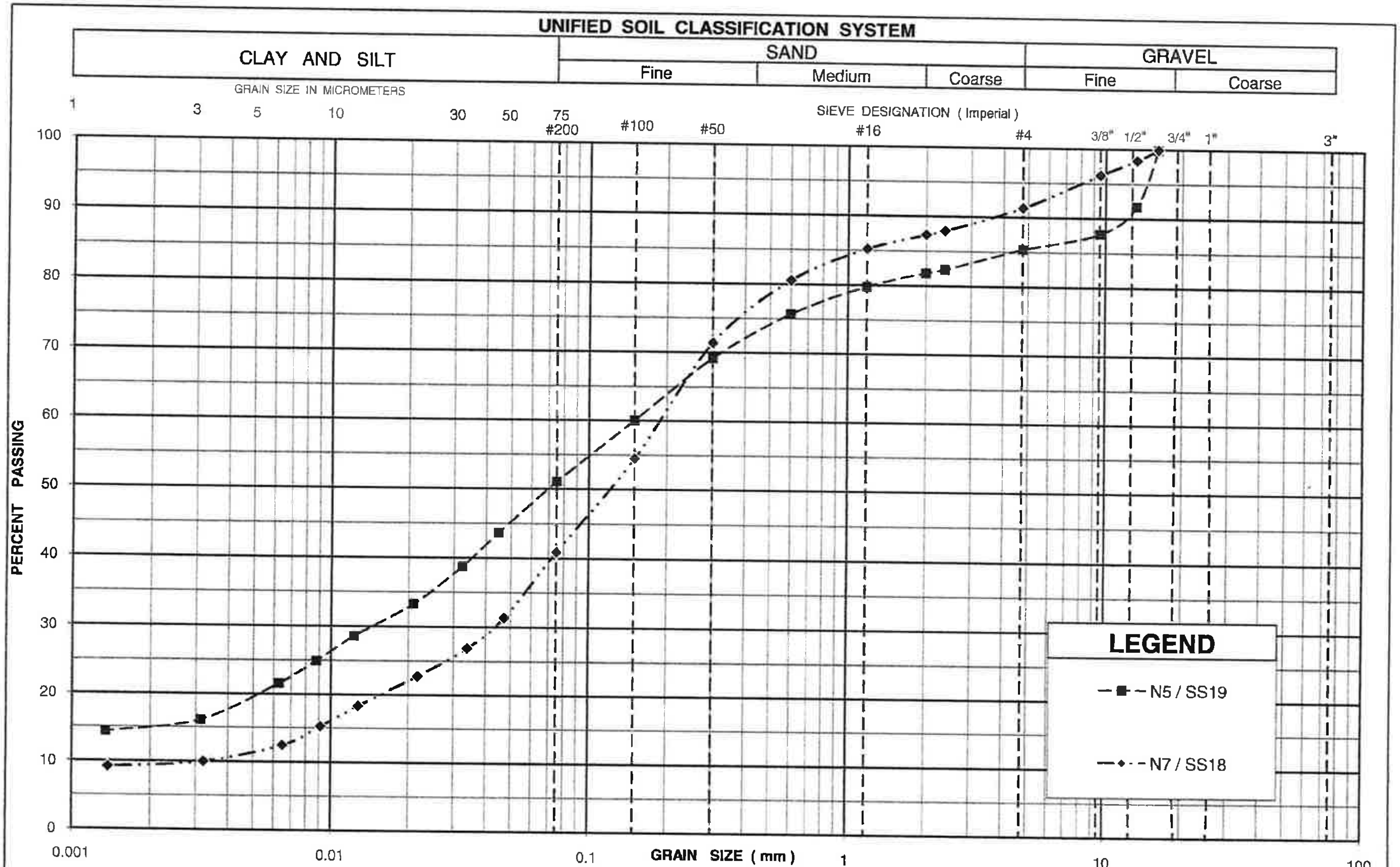
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL, trace to some sand, trace gravel

FIGURE NO.:	B-24
PROJECT NO:	TRANETOB01245AA
DATE:	MAR, 2010



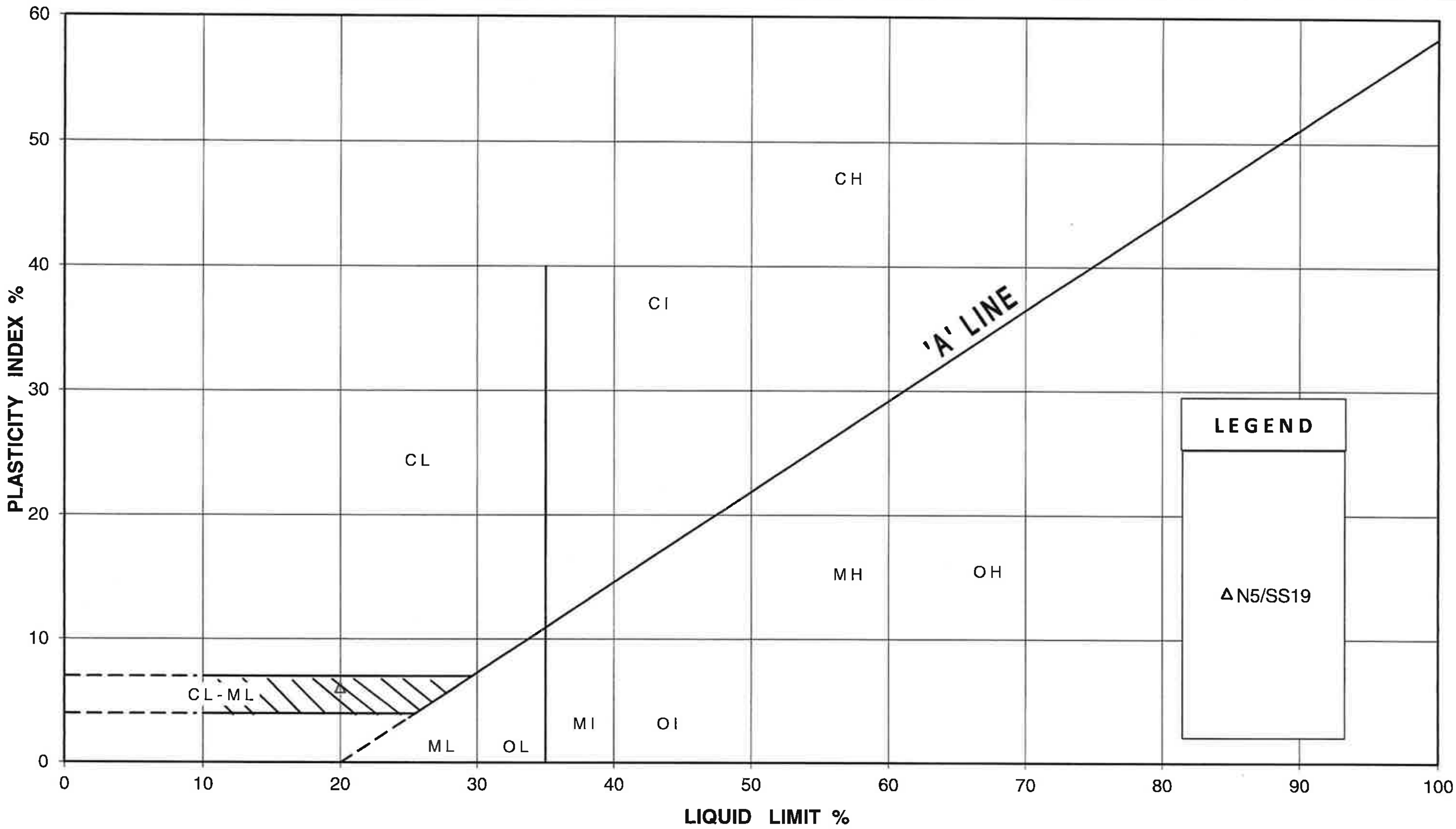
coffey geotechnics SPECIALISTS MANAGING THE EARTH	PLASTICITY CHART CLAYEY SILT TILL	FIGURE No. B-25 REF. No. TRANETOB01245AA DATE MARCH, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILTY SAND TILL

FIGURE NO.: B-26
PROJECT NO: TRANETOB01245AA
DATE: MAR, 2010



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PLASTICITY CHART

SANDY SILT TO SILTY SAND TILL

FIGURE No.	B-27
REF. No.	TRANETOB01245AA
DATE	

Appendix C

Stratigraphic Contacts - Highway 401 Viaduct Site

(Past Investigations)

Project Number: TRANETOB01245AA-AD **VIADUCT**

Table C-1

References	Borehole Designat	Ground Surface (ft)	Top of Sand (ft)	Top of Clay (ft)	Top of Till (ft)	Top of Lower Sand (ft)	Water Table (ft)	*Ground Surface* (m)	*Top of Sand* (m)	*Top of Clay* (m)	*Top of Till* (m)	*Top of Lower Sand* (m)	*Water Table* (m)	*Depth to Till* (m)	*Depth to Sand* (m)	Thickness: Fill or Clayey Silt, Organic (m)	Thickness: Sand (m)	Thickness: Clay (m)	Thickness: Till (m)	Thickness: Fill or Clayey Silt, Organic (ft)	Thickness: Sand (ft)	Thickness: Clay (ft)	Thickness: Till (ft)
	Minimum	461.0	454.9	443.2	401.7	0.0	447.0	140.5	138.7	134.7	122.4	0.0	136.2	10.9	0.0	0.0	0.3	8.5	0.0	0.0	0.0	28.0	0.0
	Maximum	487.9	470.0	479.0	451.0	0.0	460.6	152.3	143.3	148.3	137.5	0.0	149.6	25.3	0.0	11.6	5.2	139.1	0.0	33.0	17.0	456.3	0.0
	Averages	474.5	462.7	455.5	417.4		456.6	145.8	141.0	139.7	127.0		141.0	18.3		4.3	2.6	20.7		12.1	5.8	74.3	
The Foundation Companies of Canada, 1953	G1	466.1	464.6	456.3			460.6	142.1	141.6	139.1			140.4			3.0		139.1		9.8		456.3	
	G2	462.2	460.2	443.2	401.7		455.3	140.9	140.2	135.0	122.4		138.8	18.4		5.9		12.6		19.2		41.3	
	G3	466.3	466.0	447.4			459.1	142.1	142.0	136.4			139.9			5.8		136.4		18.9		447.4	
	G6	487.9	454.9	448.4	410.9			148.7	138.7	136.7	125.0			23.7		10.1	2.0	11.7		33.0	6.5	38.4	
	G7	461.0	459.0	449.5	410.0		457.8	140.5	139.9	137.0	125.0		139.5	15.5		0.6	2.9	12.0		2.0	9.5	39.5	
Geocon Limited, 1960	1	475.5	461.0	448.5	407.5		456.5	144.9	140.5	136.7	124.2		139.1	20.7		4.4	3.8	12.5		14.5	12.5	41.0	
	2	461.8	461.3	447.3	406.8			140.8	140.6	136.3	124.0			16.8		0.2	4.3	12.3		0.5	14.0	40.5	
	3	475.0	459.0	449.5			457.4	144.8	139.9	137.0			139.4			4.9	2.9			16.0	9.5		
DHO*, 1964	B18	461.5	461.5	444.5	403.0		455.0	140.7	140.7	135.5	122.8		138.7	17.8		0.0	5.2	12.6		0.0	17.0	41.5	
DHO*, 1965	201	475.2	461	446	406		447	144.8	140.5	135.9	123.7		136.2	21.1		4.3	4.6	12.2		14.2	15.0	40.0	
	202	474.8	470	455	412		458	144.7	143.3	138.7	125.6		139.6	19.1		1.5	4.6	13.1		4.8	15.0	43.0	
	203	475.7	466	456	412			145.0	142.0	139.0	125.6			19.4		3.0	3.0	13.4		9.7	10.0	44.0	
	204	477.8	464	460	416		458	145.6	141.4	140.2	126.8		139.6	18.8		4.2	1.2	13.4		13.8	4.0	44.0	
	204A	477.8	462	461	420		458	145.6	140.8	140.5	128.0		139.6	17.6		4.8	0.3	12.5		15.8	1.0	41.0	
	204B	477.8	463	460	423			145.6	141.1	140.2	128.9			16.7		4.5	0.9	11.3		14.8	3.0	37.0	
	204C	477.8	465	460	415			145.6	141.7	140.2	126.5			19.1		3.9	1.5	13.7		12.8	5.0	45.0	
	204D	477.8	463	460	413			145.6	141.1	140.2	125.9			19.8		4.5	0.9	14.3		14.8	3.0	47.0	
	204E	477.8	463	458	420			145.6	141.1	139.6	128.0			17.6		4.5	1.5	11.6		14.8	5.0	38.0	
	205	478.5	465	459	414			145.8	141.7	139.9	126.2			19.7		4.1	1.8	13.7		13.5	6.0	45.0	
	206	481	466	465	423			146.6	142.0	141.7	128.9			17.7		4.6	0.3	12.8		15.0	1.0	42.0	
	207	481.4		472	441			146.7		143.9	134.4			12.3		2.9		9.4		9.4	0.0	31.0	
	208	484.9		475	443			147.8		144.8	135.0			12.8		3.0		9.8		9.9	0.0	32.0	
	209	486.9		479	451			148.4		146.0	137.5			10.9		2.4		8.5		7.9	0.0	28.0	
	210	478	463	460	420			145.7	141.1	140.2	128.0			17.7		4.6	0.9	12.2		15.0	3.0	40.0	
	211	462.5	461.3	452.5	413		457	141.0	140.6	137.9	125.9		139.3	15.1		0.4	3.1	12.0		1.2	10.2	39.5	
Ministry of Transportation, 1990	1-1							150.8	139.2	134.7	125.5		147.5	25.3		11.6	4.5	9.2					
	1-2							151.3	142.5	138.2	128.7		139.1	22.6		8.8	4.3	9.5					
	1-3							151.7		144.4	128.8		149.6	22.9		7.3		15.6					
	1-4							152.0		145.9	133.7		143.5	18.3		6.1		12.2					
	1-5							152.3		148.3	137.1		147.0	15.2		4.0		11.2					

Appendix D

Site Photographs

Photograph 1 Viaduct site (looking west)

Photograph 2 Viaduct site (looking west)

Appendix E

Explanation of Terms Used in the Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_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 = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_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
P	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
P_d	kg/m^3	DENSITY OF DRY SOIL	W_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_P)/ I_P$	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L - W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	$e_{,max}$	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PRELIMINARY FOUNDATION DESIGN REPORT, EXISTING VIADUCT WEST OF HIGHWAY 401 AND LESLIE STREET, HIGHWAY 401 REHABILITATION FROM LESLIE STREET TO WARDEN AVENUE, MTO CENTRAL REGION, G.W.P. 2130-01-00 GEOCRES 30M14-335

Delcan Corporation
Project: TRANETOB01245AA-AE
December 19, 2011

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

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4 DISCUSSION AND RECOMMENDATIONS

As part of the proposed rehabilitation of Highway 401 from Leslie Street to Warden Avenue, three highway ramp structures were originally identified within the scope of Foundation Engineering, for field investigation and preliminary foundation recommendations. On that basis, Coffey Geotechnics Inc. (Coffey) was retained by Delcan Corporation (Delcan) to carry out a preliminary foundation investigation at the site of the proposed rehabilitation of the following existing highway ramp structures.

Structure Name	MTO Structure Number
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 Foundation Engineering scope of work was changed in late 2010, based on the overall preliminary design recommendations. This happened after the completion of Coffey's foundation investigation program. The new scope was defined based on the following:

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

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

Based on the information provided to us by Delcan, the existing N-W ramp and the Highway 401 westbound Collector structures, carrying highway traffic entering from Leslie Street and Highway 401 collector traffic, will be replaced with a CNR overpass structure and embankments to the east and west of CNR, conditions permitting. This treatment will eliminate the existing viaduct and will place the the traffic on a raised embankment. Preliminary construction staging drawings of the proposed works (prepared by Delcan), including a high permanent retaining wall on the north side, were provided to Coffey. It is our understanding that further details of the new embankment will be developed during detail design phase. .It should be pointed out that the borehole coverage is not consistent with normal MTO convention due to this fact. This report provides preliminary foundation recommendations for the proposed embankment and retaining wall at the existing viaduct, west of CN track(s). Discussion and recommendations for the retaining wall located east of CN track(s) are presented in the C.N.R. Overpass Structure Preliminary Foundation Investigation and Design Report, dated September 30, 2011 by Coffey.

It is our understanding that two options were considered; the first option was to maintain (rehabilitate) the existing Highway 401 westbound collector structure and also to rehabilitate/widen N-W ramp structure and the other option was to fill-in the void underneath the existing Highway 401 at the viaduct to accommodate westbound collector and N-W ramp lanes on top of the newly constructed embankment.

Since maintaining the existing structure option would not change the geotechnical conditions, only the filling-in option will be discussed in this report.

Based on the information provided to us by Delcan, the proposed new embankment to replace the existing viaduct will be located west of the existing CN track(s) and its total length will be about 350 m. Preliminary construction staging drawings provided to us by Delcan are included in Appendix G. A retaining wall is proposed along the north edge of the proposed embankment due in part to property restrictions.

Based on the staging drawings provided to us by Delcan, the construction will start with the demolition of the existing structure(s) from the north side and the remaining Highway 401 structures (lanes) will be retained for the Highway 401 traffic during the construction. The northern half of the embankment will be constructed with a permanent reinforced concrete retaining wall on the north side, along with a temporary shoring system on the south side. After the construction of the northern portion, the construction will move to the southern portion (in between the newly built northern embankment, supported by permanent retaining walls, and the existing highway westbound express lanes) and the newly built embankment and the remaining Highway 401 structure(s) will support the Highway 401 traffic. The new retaining wall is necessary due to property restrictions.

Typical height of the existing embankment (berm, above o.g.) is 5 m on the east side, decreasing westerly to about 3 m. The grade will be raised by about 6 m, thus bringing the total fill height to about 9 to 11 m. The height of the new permanent retaining wall, supporting the embankment along the north side will be about 1 m higher than the proposed embankment top (i.e. about 10 to 12 mm above the o.g. level), as presently proposed by Delcan.

Based on the available MTO GEOCRES information, the existing structure at the proposed viaduct site appears to be supported by caisson foundations. Available information of the existing foundations is summarized in Table H-1 in Appendix H.

In general, the sub-surface stratigraphy comprises topsoil, pavement, pavement fill, other fill and native surficial granular soils overlying silty clay, which is in turn underlain by cohesive and non-cohesive glacial till deposits (See Drawings 2 and 3 in the Foundation Investigation Report). The previous and present investigations indicate similar overall subsurface conditions at the site.

4.1 Past Geotechnical Events

The following is a sequential summary of past geotechnical events at the existing viaduct site.

❖ ***Embankment Failure During Construction (1953)***

In 1953, the construction for the original Highway 401 interchange for the C.N.R. and Leslie Street overpass (present express lanes) commenced. The maximum required embankment height was 10.7 m (35 feet) including pavement. When the fill attained the maximum height of 9.8 m (32 feet), a failure of the northern slope of the western approach embankment occurred. The Foundation Companies Canada investigated the embankment instability by advancing sixteen (16) boreholes at the site to provide remedial measures (1953). The proposed remedial measure was to decrease the shearing stress in the clay by adding berms. No berm was suggested for embankment heights less than 6.4 m (21 feet). For a 10.7 m high embankment with 2H:1V side slopes, a berm 4.3 m (14 feet) high and 15 m wide (50 feet) was proposed.

❖ ***Embankment Construction (1954-1959)***

Construction proceeded without further incident until late fall of 1954 when another slide took place on the east approach with the embankment height at 9.1 m (30 feet). The appropriate revision in berm layout having been made, the construction of the interchange was then successfully completed.

❖ ***Additional investigation for the proposed service road (1960)***

Geocon Ltd was retained by Department of Highways, Ontario and carried out an investigation by advancing three boreholes at the site to provide information required for the design of the proposed service roads to Highway 401. Specifically, it was required to determine if the shear strength of the soft clay underlying the existing embankment has been affected by the weight of the embankment since its construction in 1954.

No substantial gain in the strength of the weak clay was noted. However, results of two oedometer tests indicated that the soft clay had been pre-consolidated by about 50 kPa (1000 psf) in excess of the overburden pressures existing before the berm construction.

❖ ***Investigation for the proposed structure (1990)***

Ministry of Transportation Ontario carried out a foundation investigation at the site by advancing five (5) boreholes for the proposed about 270 m long, 17 span Highway 401 westbound collector lanes.

The selection of a bridge viaduct structure instead of embankment for the construction of westbound collector lanes was generally due to concerns with slope stability and specifically due to the slope failure at this site which took place in 1953.

References

- *The Foundation Company of Canada Limited, Toronto Bypass Highway #401, Soil Conditions – C.N.R. & Leslie St. Overpass, C7142, September 30, 1953.*
- *Geocon Limited, Soil Conditions and Stability, Proposed Embankment, Leslie St. & Hwy. 401, S7002, April 8, 1960.*
- *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.*

- *Golder investigation report (slope stability, 1962) was mentioned in MTO report (1990) but is not available in MTO GEOCRES information system.*

4.2 Current Site Setting

It is our understanding that at the existing Highway 401 interchange was last performed in early 1990's and no additional major construction was carried out at this interchange location after that time. Based on the available information (i.e. MTO GEOCRES), the existing Highway 401 westbound collector and N-W ramp structures are supported on deep foundations. Details of the existing foundations are summarized in Tables H-1 in Appendix H. Side slopes of the existing embankment berm under the structure appear to be about 2H:1V based on the section drawings provided to us by Delcan and available past reports drawings (See Appendices F and G).

According to the Ministry of Transportation, Engineering Material Office investigation report (1990), this site was described as follows at the time of investigation; *"The northern side of the embankment which supports Hwy. 401 WB core lanes is exposed under the bridge. The exposed embankment (under the bridge) slopes at 1.5H to 1V. The ground surface under the bridge, extending from the toe of the embankment towards north is almost flat. The ground then slopes down at 2H to 1V towards the north into a drainage ditch and then rises up."*

The existing site setting, including highway embankments, berm, drainage ditch and ground beyond the drainage ditch on the north side of the existing highway (see cross-sectional and construction staging drawings in Appendix G), appears to be very similar to the previously described topography at the time of previous investigation (1990). It should be also noted that the original berm width was about 15 m and it may have been widened in 1990's by about 5 m to the north (based on the cross-sectional drawings, see Appendix G) to facilitate the construction of the piers of the existing highway Westbound Collector and N-W Ramp Lanes. This existing berm under the existing Westbound Collector viaduct is therefore the original 15 m wide and 4.3 m high berm placed to stabilize the 10.7 m high embankment placed in 1953 for the central (express) lanes. However, it appears that in 1990's this berm was widened by another 5 m. Since deep foundations were used to support the existing Highway 401 collector and ramp structures on the north side of the Highway, alteration of topography due to the construction in early 1990's, except for the above mentioned widening, can be assumed to be minimal. As was mentioned earlier, it is our understanding that no major construction was carried out in the vicinity of the existing viaduct site after early 1990's.

4.3 New Embankment Construction Options

The existing void underneath the highway at the viaduct site was created by the construction of the existing Highway 401 westbound collector and N-W ramp structures in 1990. The existing viaduct at this site was built with a berm placed in 1953 as a remedial measure to the previously observed slip. For the proposed embankment (replacing the existing viaduct), the existing void underneath the existing Highway 401 Westbound Collector and the N-W Ramp Structures will be filled with acceptable / applicable engineered fill materials to accommodate Highway 401 westbound collector and N-W ramp lanes on top of the proposed new embankment. The height difference between the existing highway express lane embankment and the top of the existing berm appears to be up to 6 m. The height of the existing highway express lane embankment over the original ground is believed to be up to 10.7 m and the width of the existing berm appears to be about 20 m, as discussed above. This void will be filled up with acceptable / applicable engineered fill material and up to 7 m high (above the existing grade and up to about 11 m high with respect to original grades) vertical face retaining wall structure will be installed on the north side of the highway due to the closely located northern limit of right of way (ROW). The wall will also enclose and protect the engineered fill material.

Since two failures have occurred in the vicinity of the site during the embankment construction, the use of normal earth fill may not be applicable for this project. If normal earth fill must be used for the proposed embankment, a structural support of the earth fill may be required. This option is briefly discussed in Section 4.3.1 of this report.

Soft ground improvement technologies, such as preloading with wick drains, deep soil mixing, stone column and electro-osmotic treatment, can be also considered for this project in conjunction with normal earth fill (in case, ground can be improved enough to support normal earth fill). Details of these options are discussed in Section 4.3.2.

Light-weight fill materials such as expanded polystyrene (EPS), slag and tire derived aggregates (TDA) can also be considered for the new embankment construction. Filling material selection will depend on the stability of the new embankment as well as the anticipated serviceability (i.e. long-term settlement). Details of these options are discussed in Section 4.3.3.

Following are some details of the above mentioned options.

4.3.1 Structural Support for Earth Fill

At grade level reinforced concrete slab, founded on deep foundations taken to competent soil, may be used to support the entire embankment thereby eliminating additional stresses on the soft clay layer.

The embankment support slab should extend over the full length of the filling area and should also extend laterally out under embankment side slopes. The edge of the slab can also support the proposed retaining walls for this project. Available foundation options are as follows;

- Driven Steel H piles
- Driven Steel tube piles
- Cast-in-place concrete piles
- Continuous flight auger piles
- Micropiles
- Re-use of the existing foundations

Founding depth/elevation of new foundations needs to be decided when details of structural requirements are available.

Unless existing foundations (i.e. caisson) can be re-used, structural slab option is anticipated to be more expensive than constructing a new embankment. In this instance consideration can be given to utilizing hollow, light weight concrete rather than regular fill. This would be advantageous if the structure was to be utilized as a parking facility.

If the existing foundations are to be re-used to support the newly proposed embankment, residual capacity and integrity of the existing piles should be carefully evaluated / verified in the detail design stage. Detailed information about the existing foundations must be available to provide confidence in their re-use.

If new piles are to be installed to supplement the capacity of the existing piles, the spacing between the existing and the new piles needs to be taken into account to evaluate the new capacity of the resulting hybrid foundation system. Pile spacing in between the existing and new piles should be provided in accordance with CHBDC S6-06 Section 6.8. Interference between the existing foundations and the new foundations should be carefully investigated prior to laying out the new foundations. Differential settlement between the existing and new foundations also needs to be considered.

4.3.2 Ground Improvement Techniques

Ground improvement techniques can be considered as feasible options for this project. Two main objectives of ground improvement are reducing the compressibility of founding soil and strengthening the soil at the site. Through soil improvement, both the stability and serviceability concerns would be eliminated. Based on the present data, it appears that the existing (berm) fill may not have received proper quality and compaction control when it was first placed. This means that it too may present problems due to settlements especially due to differential (i.e. uneven settlements). This aspect should be considered in the evaluation of options. Although the methods discussed in the ensuing paragraphs relate to native soils, some may also help improve, to a certain extent, the existing berm fill.

Available ground improvement methods for native soils are as follows;

4.3.2.1 Preloading / Surcharging / Wick Drains

Pre-loading/surcharging with/without wick drains options are the most common soft ground improvement technique. Where site conditions are appropriate, these options offer an innovative, cost effective solution to embankment settlement/stability concerns. Another aspect of this method is its influence in improving the existing fill settlements. However, this option would be impractical for this project due to the existing site conditions (low overhead condition) and the required relatively longer surcharge or preload period (i.e. time constraints), after the removal of the existing structure.

4.3.2.2 Deep Soil Mixing Method

Deep Mixing Method (DMM) is an in situ soil treatment technology whereby the soil is blended with cementitious and/or other materials. These materials are widely referred to as "binders" and can be introduced in dry and slurry (wet) form. Dry mixing method is common for normal soft soil with higher natural moisture contents. Binders are injected through hollow, rotated mixing shafts tipped with cutting tool. The shaft above the tool may be further equipped with discontinuous auger flights and/or mixing blades or paddles.

These shafts are mounted vertically on a suitable carrier.

The resulting cemented soil that ensues the process generally has a higher strength, lower permeability, and lower compressibility than the native soil, although its total unit weight may be less. The exact properties obtained reflect the characteristics of the native soil, the construction variables (principally the mixing method), the operational parameters and the binder characteristics.

The original concept appears to have been developed more than 60 years ago in the United States, although contemporary deep mixing technology reflects mainly Japanese and Scandinavian effort over the last four to five decades. It was introduced in North America in the late 1980's and has been widely used since.

The main applications of DMM are as follows;

- Hydraulic cut off walls
- Excavation support walls
- Ground treatment
- Liquefaction mitigation
- In-situ reinforcement, piles and gravity walls
- Environmental remediation

The various DMM techniques can be used to produce a wide range of treated structures as follows;

- Single elements
- Rows of overlapping elements (walls or panels)
- Grids or lattices
- Block

The particular geometry chosen is, of course, dictated by the purpose of the DMM application and reflects the mechanical capabilities and characteristics of the particular method used.

The design strength and other details regarding the design of deep soil mixing should be discussed with a specialist contractor and we will be pleased to expand on this further should you wish to pursue this option. Careful field control and experienced contractor is required for this process.

Due to the limited available head-room for equipment operation and time frame to facilitate the implementation of this method, however, it is unlikely, to be a feasible solution. Consideration can however also be given to improving the soil beyond the toe of the berm (for slope stability purposes), if permission can be obtained from the owners of the adjoining property to the north.

4.3.2.3 Stone Columns

Stone column construction involves the partial replacement of unsuitable soils with a compacted vertical column of stone that usually completely penetrates the weak strata. When jetting water is used, the process is named vibro-replacement (or the wet method). When used without jetting water in partially saturated soils, the process is known as vibro-displacment (or the dry method). Both wet and dry methods have been used in Europe and Canada since late 1950's.

The stone is densified by the use of a vibrating probe (frequently called vibroflot or poker) originally developed in 1930's for the compaction of granular, non-cohesive soils. In the wet process, the vibroflot opens a hole by jetting using large quantities of water under high pressure. In the dry process, which may utilize air, the probe displaces the native soil laterally as it is advanced into the ground. In both methods, the weight of follower tubes attached above the probe and the vibration of the probe aid in advancing hole.

The probe typically varies in diameter from 300 to 460 mm. Due to soil erosion and lateral compaction, the excavated hole is slightly larger than the probe. To construct the column, the hole is backfilled in 0.3 to 1.2 m lifts with the probe usually being left in the hole. Stone is poured from the ground surface and allowed to fall through the annular space provided between the probe and the side of the enlarged hole. Each lift is re-penetrated several times with the vibrating probe to densify the stone and force it into the surrounding soil. The vibrating probe may also be momentarily left in a stationary position to densify the stone. Successive lifts are placed and densified until a column of stone has been formed up to the ground surface.

In stone column construction, usually 15 to 35 percent of the weak soil volume is replaced by stone. The presence of the column creates a composite material of lower compressibility and higher shear strength than the native soil. Confinement, and thus stiffness of the stone is provided by the lateral stress within the weak soil.

The main applications of stone column construction are as follows;

- Embankment fill support
- Railroad and Wharf structure support
- Liquefaction mitigation
- Slope stability

The design bearing capacity and other details regarding the stone column should be discussed with a specialist contractor and we will be pleased to expand on this further should you wish to pursue this option. Careful field control and experienced contractor is required for this process.

This process will unlikely be suitable for this project due to lack of sufficient overhead under the existing viaduct.

4.3.2.4 Electro-Osmotic Treatment

Electro-osmotic technique for soil treatment utilizes a direct electric current (DC) to induce water flow toward electrodes which are inserted through the compressible soil layers. The soil pore water will be attracted towards the direction of the negative terminal (cathode) due to the interaction of the electric field, the ion in the pore water and the soil particles. If drainage is provided at the cathode and prohibited at the anode, consolidation will be induced by electro-osmosis, resulting in the lower soil water content, higher shear strength and lower compressibility. In addition, electrochemical reactions associated with an electro-osmotic process alter the physical and chemical properties of the soil and lead to a further increase in shear strength. Preliminary laboratory and in-situ testing may be required to assess the extent, duration and effectiveness of the treatment. According to many research papers, electro-osmotic treatments can induce few decades' settlements prior to construction depending on site conditions and treatment plan.

The details regarding the electro-osmotic treatment should also be discussed with a specialist contractor. However, this process is likely to suffer from a lack of time available to allow its implementation.

4.3.3 Light Weight Fill Options

The use of light weight fill option is considered to be the most favourable option for this project because the anticipated construction period is likely to be shorter than a conventional embankment construction and traffic disruption will be minimized in comparison with other construction options mentioned above. In addition, the anticipated short term stability of the new embankment will improve with a light weight fill in comparison with a conventional earth fill. Anticipated long term settlement of embankment will be minimized or eliminated by using light weight fill as well.

Available light weight fill materials are summarized in Table 4.3.3.1.

Table 4.3.3.1

Materials	Unit Weight	Material Cost	Remarks (Pros/Cons)
Extruded Polystyrene (XPS)	0.5-1.0 kN/m^3	More expensive than EPS	Higher stiffness and less thermal conductivity than EPS. Improved surface roughness Most expensive option
Expanded Polystyrene (EPS)	0.5-1.0 kN/m^3	Expensive	Excellent thermal performance, high compressive strength, outstanding impact absorption, imperviousness to moisture, recyclable Expensive option
Blast Furnace Slag	Typically 11.5 kN/m^3** to 14.2 kN/m^3***	Relatively lower cost	Lightweight compacted density, high angle of internal friction angle and economical. A general approval was given by MTO and MOE but the product used for the site will need to be checked by MOE to make sure that it falls within acceptable range.
Tire derived aggregate (TDA)	Typically 8 kN/m^3	Relatively lower cost	Light weight, low lateral pressures on walls, good thermal insulation, high permeability and absorb vibration, economical Relatively newer technology. Will need to be formally approved by MTO and MOE
Expanded clay	Typically 8 kN/m^3	Relatively lower cost	Light weight, low lateral pressures on walls, good thermal insulation, high permeability and absorb vibration, economical Relatively newer technology. Will need to be formally approved by MTO and MOE

*actual material cost and specifications should be confirmed with local supplier
**ultra light weight slag
***light weight slag

4.4 Slope Stability Analyses

For this study, slope stability analyses were carried out using a limit equilibrium approach. The analyses were carried out using the commercial two-dimensional slope stability computer program Slope/W (Geostudio 2007) and the Mogerstern-Price method of analysis for both short term (undrained) and long term (drained) analysis calculations.

The short term stability analysis represents the embankment and founding ground stability immediately after the embankment construction. For fine-grained soils, undrained shear strengths are used (total stress approach), while for soils that drain freely, drained strengths are used (effective stress approach). Within fine-grained soils, which are sufficiently impermeable, excessive pore pressures are induced by additional loading (embankment construction) and these will be dissipated with time.

The long term stability analysis represents the embankment and founding ground stability long time after the construction. This analysis is performed to reflect the condition after swelling or consolidation has occurred. Shear strengths are expressed in terms of effective stresses and the pore water pressures are estimated from the stabilized or adverse groundwater conditions anticipated.

As was mentioned earlier, embankment instability occurred in 1953 during the construction of embankment at the viaduct site. When fill attained the maximum height of 9.8 m, failure occurred on the north side of the Highway 401. The Foundation Companies Canada (1953) carried out the site investigation immediately after the slip and then embankment construction was completed with the placement of a 4.3 m high berm to reduce the shear stress in the underlying soft clay.

A significant aspect of this previous failure is the presence of the former failure zone present at this site. From the previous correspondence it is believed that the failure extended from near the centreline of the embankment (i.e. present express lanes) to about 17 m (55 ft) beyond the north toe of the embankment under construction at that time. This could possibly form a zone of weakness when the existing berm is further loaded. Geotechnical literature suggests that such failure zones are typically between 10 and 15 cm thick and that the clay particles re-orient themselves in the direction of the failure. In this case, however, as the clay is only very lightly pre-consolidated, there may be not be a significant difference in the stress-strain characteristics between the peak and post-peak (i.e. residual) undrained shear strengths of the material. In any event, however, the presence of this failure zone needs to be taken into consideration.

For this preliminary study, back stability analyses of the previous embankment failure and of the rectified embankment with berm placement were carried out using the same slope geometry that the Foundation Companies Canada used for their analysis (1953, See Appendix F), to get a better understanding of soil strength parameters. In other words, a back analysis was done using the known failure points beyond the north toe and the centreline of the embankment. The parameters chosen with the help of this analysis was used for further loading analysis of the existing berm.

The shear strengths of the soft clay used for the short-term stability analysis were estimated based on the field vane test results of the past investigations (The Foundation Companies Canada, 1953, Geocon Limited, 1960 and Ministry of Transportation, 1990). The in-situ undrained shear strengths, as measured by field vane tests, along with the design values used for this present analysis are presented in Figure I-1 in Appendix I. Long-term stability analyses of the existing embankment with berm were also carried out to evaluate the stability of the existing embankment using drained soil parameters.

The effects of the existing embankment and berm weight on the undrained shear strength of the underlying clay for more than five decades were also evaluated by comparing the field vane test results (in-situ undrained shear strength) of the past and recent investigations (see Figure I-2 in Appendix I). Recently measured in-situ undrained shear strengths of the clay appear to plot near the estimated upper bound of the previously measured shear strengths as shown in Figure I-2. Consequently, there appears to be a gain in undrained shear strength under the berm, but this should be verified during the detailed investigation and design.

The updated design undrained shear strengths, shown Figure I-2, as well as the design undrained shear strengths shown Figure I-1 were used to evaluate the short-term stability of new embankment constructed with various different fill materials (i.e. earth fill, EPS etc).

Long-term and short-term stability analyses were carried out using the listed soil parameters in Tables 4.4.1 and 4.4.2.

<u>Previous embankment stability analyses</u>

For the previously failed embankment stability analyses (back analysis to confirm the soil parameters), we made the following assumptions;

- Assume that the geometry of previously failed embankment is as shown on 1953 Foundation Companies Canada and 1960 Geocon reports (instability started from centerline of embankment and terminated 17 m (55 ft) front of the embankment toe)
- The original proposed maximum height of embankment was about 10.7 m (35 ft)
- Slope failed at 9.8 m high (32 ft) in 1953
- Placement of berm, wherever slope is higher than 6.4 m (21 ft), was implemented as remedial measure
- Originally planned berm height and width were 4.3 m (14 ft) and 15.2 m (50 ft)
- The start and end points of the slip circles (i.e. entry and exit slip surface option in Slope/w) were specified for the analyses.

<u>New embankment stability analyses</u>

- For our analyses only deep seated failures were considered. In other words, the slip circles were not allowed to intersect the face of the retaining wall that would be constructed for this project
- The effects of existing deep foundations as well as those that may be used to support the retaining wall were ignored. This however would be on the safe side.

Table 4.4.1 Soil Parameters (for previous embankment failure and existing condition, See Figure I-1)

Soil Type	Unit Weight (kN/m^3)	Shear Strength Parameters			
		Undrained		Drained	
		Undrained shear strength (cohesion, kPa)	Angle of internal friction (deg)	Cohesion (kPa)	Effective angle of internal friction (degrees)
Embankment Fill	21	-	32	-	32
Surficial Granular Soils	20	-	30	-	30
Cohesive Soil (upper)	18	15-40*	-	-	27
Sand	20.5	-	33	-	33
Cohesive Soil (lower)	18.5	60	-	-	28
Glacial Till	22	-	36	-	36

*Shear strength linearly increases with depth (increasing rate = 3.5 kPa/m and maximum shear strength is 40 kPa)

Table 4.4.2 Soil Parameters (for new construction options, See Figure I-2)

Soil Type	Unit Weight (kN/m^3)	Shear Strength Parameters			
		Undrained		Drained	
		Undrained shear strength (cohesion, kPa)	Angle of internal friction (deg)	Cohesion (kPa)	Effective angle of internal friction (degrees)
Pavement structure	22	-	33	-	33
Embankment Fill	21	-	32	-	32
Surficial Granular Soils	20	-	30	-	30
Cohesive Soil (upper)	18	30-50*	-	-	-
Sand	20.5	-	33	-	33
Cohesive Soil (lower)	18.5	70	-	-	-
Glacial Till	22	-	36	-	36
Earth Fill	21	-	32	-	32
EPS**	0.5-1.0	5***	-	5***	-
Slag**	11.5	-	38	-	38
TDA**	8	-	20	-	20

*Shear strength linearly increases with depth (increasing rate = 3 kPa/m and maximum shear strength is 50 kPa)

**Materials' shear strength parameters are assumed for stability analysis (see each figure in Appendix J), ultra light weight slag is listed.

***This value is an assumed very low value. However, it does not materially affect the factor of safety results

A factor of safety close to unity was calculated for the previously failed embankment (see Figure J-1 in Appendix J) using the undrained shear strength parameters given in Table 4.4.1. Factor of safety versus embankment height is also presented in Figure J-2 in Appendix J. This indicates that the above tabulated soil parameters are reasonable to simulate the previously failed slope at the site, since a factor of safety of 1.04 was obtained for the 1953 failure, using these parameters. As well, the same parameters indicate a factor of safety of 1.31 with the stabilizing berm in place and a factor of safety of 1.3 was quoted by the Foundation Companies Canada (1953 report), who designed the berm.

The following table presents the calculated factors of safety for both short-term and long-term stability of the existing embankment and various filling-in options.

Table 4.4.3

Construction Stage	Type of Analysis	Figure No***	Factor of Safety	Remark
Past and existing embankment construction				
Embankment Construction (about 10 m high)+	Short term stability♦+	J-1	1.04	Stability vs embankment height, See Figure J-2
Embankment with berm (10 m high embankment with 4.5 m high berm)+	Short term stability♦+	J -3	1.31	
Embankment with berm (10 m high embankment with 4.5 m high berm)+	Long term stability+		2.91	
Proposed embankment construction				
Conventional earth fill option	Short term stability♦	J-4	1.07	
	Short term stability♦♦		1.26	
	Long term stability*		1.96	
EPS fill option** Unit weight of EPS=0.5-1.0 kN/m^3	Short term stability♦	J-5	1.34	
	Short term stability♦♦		1.63	
	Long term stability*		2.50	
Slag fill option** Unit weight of slag=11.5 kN/m^3	Short term stability♦	J-6	1.21	
	Short term stability♦♦		1.47	
	Long term stability*		2.14	
Tire derived aggregate (TDA)** Unit weight of TDA=8 kN/m^3	Short term stability♦	J-7	1.29	
	Short term stability♦♦		1.56	
	Long term stability*		2.25	

+Stability analyses were carried out using entry and exit option (based on the description of previous failure)
♦soil properties listed in Table 4.4.1 were used
♦♦ soil properties listed in Table 4.4.2 were used (with consideration of strength increase)
*Long term stability analysis includes surcharge of 12 kPa on top of the embankment to simulate the anticipated traffic load on the highway
** material shear strength parameters of EPS, ultra light weight Slag and TDA are assumed
Note: Proper drainage system needs to be installed to avoid an unexpected water pressure build up behind the wall
Figures are in Appendix J

Typically minimum factor of safety 1.3 for embankment is generally acceptable for MTO projects but for significant structures, the minimum factor of safety frequently increased to 1.5 for long term analyses. The factor of safety figures in the table indicate that with the assumed parameters and undrained shear strength value using historical data, only the EPS option is acceptable, while with the recent vane test results the ultra light weight slag and TDA are also acceptable, in addition to EPS option.

4.5 Settlements

In addition to slope stability, settlements play a major role in the selection of fill materials that can be used to raise the grade.

As the weak silty clay deposit is known to be a normally or slightly pre-consolidated material, stresses induced by any additional filling can be expected to cause significant settlements in this deposit. From the past investigations a number of consolidation test results are available in addition to one from the present investigation. While majority of these are not specifically from the viaduct location, they can be and were utilized for this preliminary analysis. Based on these there are two scenarios;

- The silty clay is pre-consolidated by about 1000 psf (48 kPa) as mentioned in the 1960 report by Geocon*
- The silty clay is normally consolidated.

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* The report states that "no definite conclusion regarding the reason for the negligible strength increase under the berm can be made on the bases of the available data. However, the results of two consolidation tests carried out in boreholes 1 and 2 and shown on Figure 3 of Appendix II suggests that the clay had been pre-consolidated by about 1,000 pounds per square foot in excess of the overburden pressure existing before construction of the berm"

Based on these, the magnitude of the anticipated settlements due to 6 m grade raise is as follows.

- Settlement of existing embankment fill and the underlying surficial fine gained granular natural soils: 60 mm
- Consolidation settlement of the underlying silty clay: 270 mm

The settlement of the existing fill and the underlying fine grained granular soils can be expected to be substantially completed within a period of several weeks, while the consolidation settlement of the underlying silty clay would take many months. In addition to this, the new embankment fill would undergo settlements which can be expected to be of the order of 30 mm, for a 6 m high fill constructed to MTO standards.

Settlements of this order of magnitude are considered excessive which will not only adversely influence the performance of the Highway 401 pavement (leading to cracking, etc., especially differential settlements) but will also cause down-drag forces on the retaining wall and its foundations. These aspects must be taken into consideration in the detail design phase. It is recommended that the design consider a maximum allowable settlement, which will provide an economical solution while minimizing these adverse effects. It is tentatively suggested that the settlement criterion be about 30 mm on top of the pavement (i.e. top of asphalt) for this preliminary analysis, which should however be carefully re-considered with discussions with other designers such as structural engineers, pavement engineers etc, on the design team.

Based on our preliminary settlement calculations with the available data and using the C_c and C_r values suggested in the Geocon 1960 report, the increase in the stresses be limited to about 20 kPa. This assumes that the weak clay is pre-consolidated by at least 20 kPa.

This criterion would dictate the use of EPS for this project.

The use of EPS typically requires a pavement thickness (i.e. asphaltic concrete, base and sub-base courses) of 1.4 m plus 125 mm thick concrete cover immediately overlying the EPS. A typical MTO convention in this regard is given in Appendix L.

If the EPS were to be placed directly on the existing (berm) grade, the increase in stress would be about 1.5 m x 23 kN/m^3 = 34.5 kPa, assuming an average bulk unit weight of 23 kN/m^3 for the asphalt, granular and the concrete cover, combined.

This means that some of the existing berm fill may need to be removed and replaced with EPS. In this case since the stresses by the pavement fill will be about 14.5 kPa in excess of 20 kPa criterion (i.e. 34.5-20 = 14.5), ignoring the weight of EPS, the soil replacement depth will be as follows. Assuming a bulk unit weight of 20 kN/m^3 for the existing berm fill materials at the surface, 14.5 ÷ 20 = 0.725 m or approximately 0.75 m of berm fill should be replaced with EPS.

The following installation procedure is anticipated. The existing berm fill would be removed to the required EPS design elevation (in the example above, 0.75 m below existing grade) plus 0.5 m. The existing pier would be cut-off at this elevation. The expected subgrade would then be carefully inspected by geotechnical personnel whereby any exposed unsuitable soils would be removed and replaced with suitable fill. The subgrade would then be compacted from the surface using a suitable heavy compactor. The grade would then be raised about 0.35 m using a suitable fill, preferably free draining and frost free. This layer would be compacted to not less than 95 % of its Standard Proctor Maximum Dry Density (SPMDD). The final lift would consist of sand with no gravel size particles to avoid damage to the EPS which would be placed in it.

Another option would be to have the existing foundations carry some weight. For example, instead of replacing the 0.75 m fill (as given by the above example) by EPS, this 0.75 m of the existing fill can be supported by of the existing foundations. This will probably involve the placement of a properly compacted granular fill platform with one or two layers of bi-axial geo-grid to distribute the load to the existing foundations. The analysis of proper load distribution is a rather indeterminate problem but would be worth while looking into to reduce EPS costs.

4.6 Preferred Option

Based on the staging drawings provided to us by Delcan, the embankment will be built, with a staging plan. According to this plan, the construction will be carried out in stages and we understand that the construction will start with the demolition of the existing bridge structure on the north side and the remaining Highway 401 structures (lanes) will be retained for the Highway 401 traffic during the construction. The existing embankment side slope (north side) of Highway 401 will be shored and excavated and a permanent retaining wall will be constructed. The new embankment will be placed with the newly constructed retaining wall on the north side and a shoring system on the south side. After the construction of the northern segment, construction will subsequently move to the southern segments (towards the south end) and the newly built embankment and the remaining Highway 401 structures will support the Highway 401 traffic.

From geotechnical engineering point of view (i.e. results of stability analyses and settlement considerations), the use of EPS is, in our opinion, the most favourable option for the proposed embankment construction for following reasons:

- Provides favourable conditions for foundation / slope stability
- Minimizes settlement

- Reduces lateral pressures on the retaining wall (in fact without the use of EPS, the design of a high wall may present difficulties, i.e. very costly).

Stability of EPS under anticipated heavy traffic loads from Highway 401 during and after construction should be verified by the EPS supplier, along with the minimum granular pavement fill thickness requirement.

Even though stability of the proposed embankment using light weight fill materials other than EPS, such as slag and TDA, may be acceptable from the stability point of view, anticipated settlements, as well as differential settlement between the existing and new embankment, are unlikely to be acceptable due to the additional stresses imposed on the founding soils (i.e. weak silty clay) induced by the weight of slag and TDA.

The use of EPS will minimize or eliminate potential settlement problems because EPS itself is almost weightless material (0.5-1.0 kN/m^3) and EPS is typically designed for no additional load to the existing conditions. For this design aspect, some sub-excavation may be required to counter-balance the additional weight of the proposed pavement structure, which will be placed on top of the proposed EPS, as discussed before.

In principle, the EPS thickness would reflect the fill thickness (i.e. where the proposed fill thickness is larger, the EPS would be thicker). The presently proposed sections and construction staging drawings are given in Appendix G. The following design criteria with the EPS option are recommended.

- The recommended thickness of the pavement structure over the EPS is about 1.5 m including the concrete cover over the EPS (present, MTO design requirements include a 125 mm thick concrete cover over the EPS). The design and construction of the EPS should be in accordance with MTO Special Provision entitled "Expanded Polystyrene Embankment" included in Appendix L.
- Sub-excavation of the existing grade may be required to counter-balance the weight increase due to the proposed pavement structure on top of the EPS.
- EPS bottom elevation should be higher than the existing ditch elevation and granular levelling pad should be sloped about 3% toward the existing ditch on the north side of the existing highway or towards similar drainage points for drainage purpose.
- EPS should be checked against uplift, including provisions for permanent drainage.
- EPS needs to be protected from ultra-violet light exposure and contamination.
- The soil underlying the EPS should be well compacted and the top 0.15 m of the soil should consist of sand with no gravel to prevent damage to the EPS.

4.7 Retaining Walls

We understand that the project includes the construction of retaining walls on the north side of the proposed embankment. 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 3 to 7 m above the existing grades and about 11 m above o.g. (original grade). The walls will need to be high performance and high appearance type of retaining structure (i.e. vertical concrete wall). Preliminary construction staging drawings in Appendix G present the proposed retaining wall.

Typical retaining wall options are as follows;

- Conventional Cast-in-place Reinforced Concrete Retaining Walls
- Retained Soil System (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.

4.7.1 Conventional Cast-in-place Reinforced Concrete Retaining Walls

Historical boreholes, located in the vicinity of the possible retaining wall locations, show that the possible retaining wall locations are underlain by an about 4.3 m thick (average) fill deposit. The fill is underlain by an about 5 m (average) surficial granular soil, bringing the combined thickness of these surface or near surface deposits generally about 9.3 m. MTO (1990) investigation report also give the indication that the fill generally consists of fine grained granular soils (including irregular layer of clayey silt and sand) with typical N-values ranging from about 6 to 37 blows/0.3 m (i.e. generally loose to dense but typically compact to dense, except for Borehole 1-1).

As was mentioned before, these conditions are unfavorable for the foundation support of structures using normal spread footing foundations. Unless the stresses can be substantially distributed fill and in the underlying surficial granular 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 date, in general, conventional cast-in-place reinforced concrete retaining walls extending to about 3 to 7 m height above existing grades (or about 11 m above o.g. levels) can not be supported on conventional spread footing foundations with the prevailing subsurface conditions. Consequently, deep foundations will need to be used.

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

- Cast-in-place concrete piles (caissons)
- Driven Steel H piles
- Driven Steel tube 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 piles may be required to resist the lateral loads on the retaining structure.

4.7.1.1 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) can be a feasible foundation option. It is noted that some of the existing Highway 401 structures in the vicinity of the proposed embankment as well as the existing viaduct structure are supported on drilled caisson foundations. Since the site is located in the Greater Toronto Area (i.e. close to residential areas and hospitals), drilled caisson is typically considered to be a favourable deep foundation option because of less noise and vibration generated during the construction in comparison with driven piles. The existing structure need to be removed prior to the installation of the caissons with a staging plan.

Based on the local design practice, caissons are socketed into the very dense/ hard till (N > 50 blows/0.3 m) a sufficient distance, whereby the caissons are designed for a combination of shaft friction/adhesion and end bearing resistance in competent till. For this particular project, for sake of simplicity, we recommend that the caissons be designed for an end bearing resistance of 2000 and 3000 kPa at SLS and ULS, respectively, for a minimum of 2.0 m embedment (socket) in the very dense / hard till, plus a friction/adhesion factor. For this project however we recommend that the total penetration be limited to no more than 2.0 m.

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

Table 4.7.1.1.1 Recommended Caisson Resistances

Caisson Diameter	Recommended SLS value using base area only* (kPa)	Recommended Factored ULS value using base area only* (kPa)	Recommended Friction/Adhesion SLS value using circumferential area (kPa)	Recommended Factored Friction/Adhesion value at ULS using circumferential area (kPa)	Corresponding Caisson Resistance kN/Pile
0.91 m (36-inch)	2000	3000	75	120	1730(SLS) 2650 (ULS)
1.07 m (42-inch)	2000	3000	75	120	2300 (SLS) 3500 (ULS)
1.22 m (48-inch)	2000	3000	75	120	2900 (SLS) 4400 (ULS)

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

These caisson sizes are recommended for efficiency in installation in consideration of the prevailing subsurface conditions. As well as was mentioned before it is assumed that the caissons will be socketed at least 2.0 m into the very dense / hard till (i.e. N > 50 blows/0.3 m). Higher caisson resistances would be available for greater penetration into the competent till but this is not recommended with due consideration of the excess hydrostatics 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 $(1.07/2)^2 \times \pi = 0.90\ m^2$. When designed for a value of 2000 kPa, a resistance of $2000\ kN/m^2 \times 0.90\ m^2 = 1800$ kN is obtained at SLS plus a resistance of 75 kPa for the additional 2.0 m penetration, giving $75\ kN/m^2 \times 1.07\ m \times \pi \times 2.0\ m = 500$ kN. When added, the resulting resistance at SLS is 1800 kN + 500 kN = 2300 kN/pile.

The recommended distance between any two adjacent caissons is not less than 2.5 diameters (centre to centre).

Table 4.7.1.1.2 presents the anticipated caisson depths/elevations at Boreholes N4, N5 and N7.

Table 4.7.1.1.2 Anticipated Caisson Depths/Elevations

Borehole No.	Existing Ground Elevation (m)	Anticipated Caisson Depth (m)	Anticipated Caisson Bottom Elevation (m)	Anticipated Socket and Base Subgrade Type
N4	144.2	23.0	121.2	Hard Clayey Silt Till
N5	144.5	22.5	122.0	V.Dense Sandy Silt Till
N7	144.9	21.4	123.5	V.Dense Silty Sand Till

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

Lateral loads can be expected to play a significant role in the design of the retaining wall.

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 level ground surface (i.e. not sloping) equal to six times the pile/caisson diameter, and a width which equals to three times the pile/caisson diameter. The pile/caisson diameter is the diameter of round pile/caisson or the average face to face distance of octagonal, hexagonal and square piles. For a pile/caisson in cohesive soils, the passive earth resistance should be limited to $2c_u$ (c_u=undrained shear strength) at the ground surface and increase linearly to $9c_u$ at a depth of three pile/caisson diameters and beyond. This pressure should be converted into a passive resistance by using a bearing width which equals to the pile/caisson diameter. In accordance with CHBDC, a resistance factor 0.5 is to be applied in calculating factored ULS resistance. The ULS lateral resistance of a pile/caisson group may be estimated as the sum of the single pile/caisson resistance across the face of the pile/caisson group, perpendicular to the direction of the applied lateral load. We will be pleased to give you preliminary soil parameters regarding this aspect, if they are needed at this preliminary stage of foundation design.

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

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

Difficulties may be encountered during the installation of the caissons due to the presence of surficial granular overburden below groundwater table and anticipated cobbles and boulders in the transition zone of silty clay to glacial till deposit as well as within the glacial till deposits. This can be discussed with a specialist contractor in relation to cost vs. caisson diameter. Dewatering will be required during the installation of the caissons due to the observed high groundwater table which will likely cause disturbance of the subgrade at the base of the caissons. These aspects will need to be red flagged in the contract documents to minimize construction claims. An NSSP should be issued to alert the presence of cobbles and boulders and potential basal and sidewall instability during the caisson installation. Temporary steel casing would be required to be installed during the construction of the caisson holes to prevent caving. The casing would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking.' If a permanent casing is to be adopted, above mentioned geotechnical resistances need to be revised. To prevent the disturbance of the base of the caisson, the concrete must be poured without delay after cleaning the base and its inspection and approval. Dewatering process may be required to successfully install caissons in this respect. This may range from dewatering the upper perched water table to pressure relief measures to reduce the hydrostatic uplift condition at the base of the caisson. Tremie concreting of the caisson can also be considered to reduce dewatering requirements for the installation of the caissons. Based on the available subsurface information, tremie concreting may be a favourable option for this project.

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

Additional deeper boreholes with piezometric instrumentation are recommended to reduce risks in caisson installation, including the definition of the interface between the till deposits and the underlying water bearing granular deposits.

4.7.1.2 Driven Steel Piles

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

If piles are to be used, the existing immediate adjacent bridge structures will need to be removed prior to driving the piles. Steel H-piles (HP310 x 110) driven to practical refusal in the competent glacial till materials can be designed for MTO's standard values of 1700 kN/pile for factored U.L.S. and 1250 kN/pile for S.L.S (for 25 mm settlement). These values can be increased by 50 kN/pile for HP 310 x 125 piles, due to the increased steel area. Normally, somewhat higher resistances are available for the pile sizes recommended. However, in view of the possible upward gradients, artesian pressures and known past problems experienced at this interchange, the use of higher capacities is not recommended. The anticipated pile tip (refusal) elevations at Boreholes N4, N5 and N7 are given in Table 4.7.1.2.1.1.

Table 4.7.1.2.1.1 Anticipated Pile Tip Depths/Elevations

Borehole No.	Existing Ground Elevation (m)	Anticipated Pile Tip Depth (m) below the existing ground	Anticipated Pile Tip Elevation (m)
N4	144.2	22.7	121.5
N5	144.5	22.0	122.5
N7	144.9	21.6	123.3

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 would normally 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. 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 and as such the use of a heavier section (such as HP 310x125) without flange reinforcement would be better suited for this project. Alternatively consideration can be given to the use of pile tip reinforcement (such as Titus point) which do not protrude beyond the pile perimeter. 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. This aspect presents an advantage for steel piles in comparison with caissons, as steel piles can be driven inclined (battered), where necessary. 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.

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

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

4.7.1.2.2 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 vibration monitoring programme should be carried out during pile driving. Special Provisions for vibration monitoring was included in Appendix L.

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

Steel tube piles of 300 mm nominal diameter with a suitable steel thickness (e.g. 324 mm x 9.4 mm), driven at least 1 m to 2 m into the very dense/hard soil, can be expected to provide a Factored Axial Resistance at U.L.S. of 1050 kN and an Axial Resistance at S.L.S. equal to 700 kN at about the tip elevations (approximately 0.6 m higher) quoted for steel H-piles given in in Section 4.7.1.2.1.

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 L.

4.7.1.3 Micropiles

Another alternative which may be considered is the use of micropiles to support the retaining wall structure.

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

Based on the observed subsurface conditions at the site, geotechnical resistance of micropile primarily depends on the bond length within the competent glacial till deposit and the type of micropile/installation method. For a preliminary design purposes, axial resistances of up to about 500 kN/micropile are available at ULS and 350 kN/micropile at SLS (for 260 mm diameter micropile) with a penetration not less than 5 m into the competent glacial till. The lateral resistances would depend on the diameter and reinforcement of micropile and needs to be checked with a specialist contractor.

As mentioned before, the use of micropiles may be less economical than other conventional deep foundations due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install conventional deep foundations. However, use of micropiles may shorten the construction period because micropiles can be installed under the existing structure without traffic disruption (removal of the existing structure may only be mandatory for micropiles at a later stage where they interfere with the existing foundations)

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

4.7.1.4 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 while the use of CFA piles will unlikely be economical/suitable for this project. We will be pleased to expand on this further should MTO wish to further pursue this option.

4.7.2 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 and is not a practical option, but this should be confirmed when details are known.

4.8 Lateral Pressures on Retaining Walls

Backfill behind the retaining structures should consist of non-frost susceptible, free-draining granular material 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. S6-06. For design purposes, the following static parameters (unfactored) can be used.

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

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

Compacted Granular 'B' Type I

Angle of Internal Friction, ϕ = 32° (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 retaining structure is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C. S6-06. 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. S6-06. When selecting the parameters vibrations from the train traffic should also be considered.

For unrestrained 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. S6-06 commentary can be consulted. We understand, however, that the present design of the rigid frame structure does not incorporate any walls.

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

Where EPS is used, a Ka value of between 0.05 and 0.1 and unit weight of 1 kN/m^3 can be adopted for EPS, for preliminary design purposes. The actual values to be utilized should however be confirmed depending on the conditions, for the final design. Proper drainage is required for the EPS (embankment) facing the wall to avoid hydrostatic uplift and damage due to continuous exposure to water, as well as to prevent hydrostatic forces on the wall. Drainage for retaining 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.

4.8.1 Seismic Design Consideration

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of C.H.B.D.C. S6-06). 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 C.H.B.D.C. S6-06 provides that Toronto has a Zonal Acceleration Ratio of 0.05 and Velocity Related Seismic Zone (Zv) 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 viaduct 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 C.H.B.D.C. S6-06 indicate that seismic analysis is not required 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.

4.9 Construction Comments

It is anticipated that sub-excavation for the proposed embankment construction will take place mostly in fill materials. The observed groundwater levels are lower than the expected excavation level, therefore, no major problems are foreseen during earthworks due to groundwater. After, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a suitable compactor. It should be noted that top 0.15 m of the soil should consist of sand with no gravel to prevent damage to the EPS.

For the retaining walls, the proposed pile cap (bottom) shown on the preliminary construction staging Drawings (see Appendix G) should be installed at least 1.2 m below the final grade for frost protection. Deep foundations should be installed as per OPSS 903.

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 mostly fill materials, occasionally into the surficial granular soil deposits. These soils can be classified as follows:

Fill	Type 3 soil above water level
	Type 4 soil below water level
Surficial Granular Soils	Type 3 soil above water level
	Type 4 soil below water level

Temporary (may be permanent) shoring is required to support the excavation based on Delcan's construction staging drawings, due to space limitations at the site. In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing / rakers). If the shoring systems components (i.e. lagging system) are to be remaining in place, the use of timber lagging may not be suitable as timber could be subject to rotting in the long run. Tight interlocking sheeting is also frequently 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 embankment adjacent to it. As well, major problems due to groundwater within the fill are not expected to present problems to justify the use of sheeting enclosures. Based on the construction staging drawing provided to us by Delcan, soldier pile and lagging system will be used for removal of the existing embankment side slopes as well as new EPS embankment construction with staging plan. Some dewatering may be required due to the possible perched groundwater condition within the fill.

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 4.9.1 Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m^3)
Embankment Fill	0.31	0.47	3.2	20.0
Silty Sand to Sandy Silt / Silt	0.33	0.50	3.0	19.5
Sand	0.31	0.47	3.2	20.5
Silty Clay	0.36	0.53	2.7	18.0
Glacial Till (upper 1 to 2 m)	0.29	0.46	3.4	21.0
Glacial Till	0.26	0.41	3.8	22.0

It should be pointed out that the presence of gravel and cobbles can possibly occur within 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 embankment. Special provision for vibration monitoring is given in the Appendix L. An NSSP may need to be issued in this respect.

Typically, we recommend that an NSSP be issued specifying that shoring piles will be cut off approximately 1.2 m below grade, however, for this project soldier piles and lagging system should be left in place for the stability of EPS embankment as well as for a construction convenience. However, the lagging should not be prone to deterioration. In this case, shoring piles can be cut off at the bottom level of concrete slab which will be placed on top of the EPS fill.

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

4.10 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 for the retaining walls.

5 RECOMMENDATIONS FOR DETAILED FOUNDATION INVESTIGATION

Some of the salient features for the detailed foundation investigation are as follows;

- If possible two boreholes should be drilled from near the toe of the existing berm, on the o.g., to compare shear strength of the soil beneath the embankment and beyond.
- Consolidation characteristics of the weak silty clay stratum should be carefully investigated. Determination of the pre-consolidation pressure will play a significant role in the design of the embankment. It should however be pointed out that in our experience obtaining sufficiently undisturbed samples in the field for oedometer testing as well as the preparation of the samples from this deposit for oedometer testing are difficult to achieve. Sampling and testing should be performed with this in mind, for the test results to be meaningful.

- Consideration should be given to the use of CPT (Cone Penetration Test), especially in the weak silty clay
- Historical boreholes show the presence of a sand layer sandwiched between upper weak silty clay and lower firm silty clay, while this was not found in the more recent boreholes (i.e. MTO, 1990 and Coffey, 2010). This aspect should be verified, as it would have an effect on the settlement analyse, including time rate of settlement.
- Our stability analyses were performed on geometry derived from historical data (especially geometry underneath the existing viaduct). During detailed design, actual geometry of existing ground surface underneath the existing viaduct should be surveyed and used for detailed stability analyses.
- The 5 m widening of the original berm in 1990's should be confirmed.
- The available boreholes do not characterize the berm material characteristics. It would be useful to find this out. However, this may be difficult to achieve, as it may required hand drilling , due to low overhead clearance under the existing viaduct
- Consideration should be given to the presence of an existing failure plan and its implications for stability analysis

6 CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.

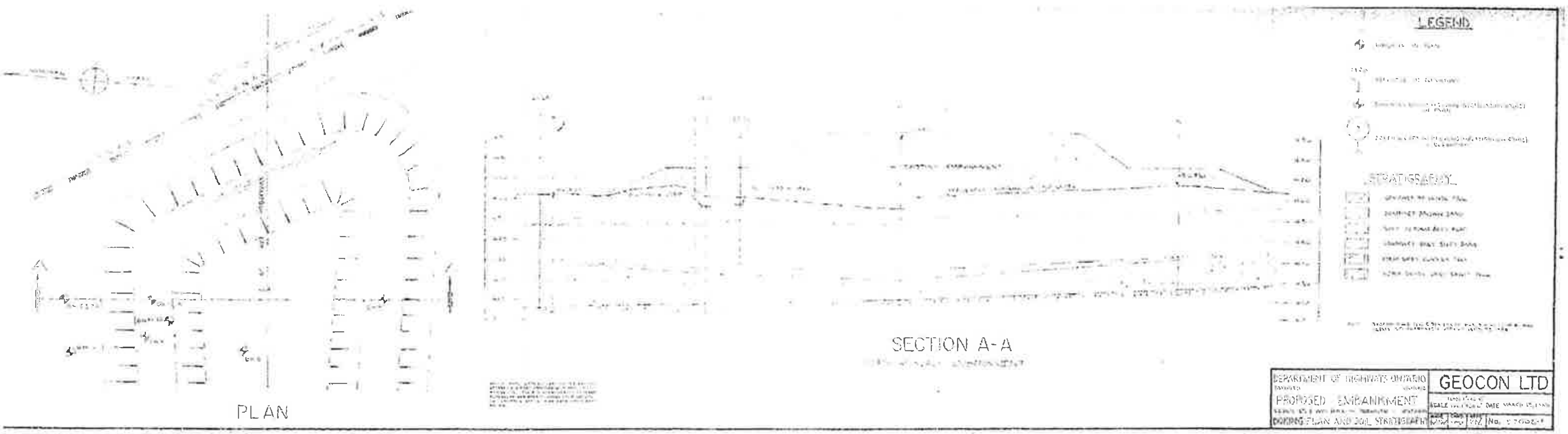
Gwangha Roh, Ph.D.

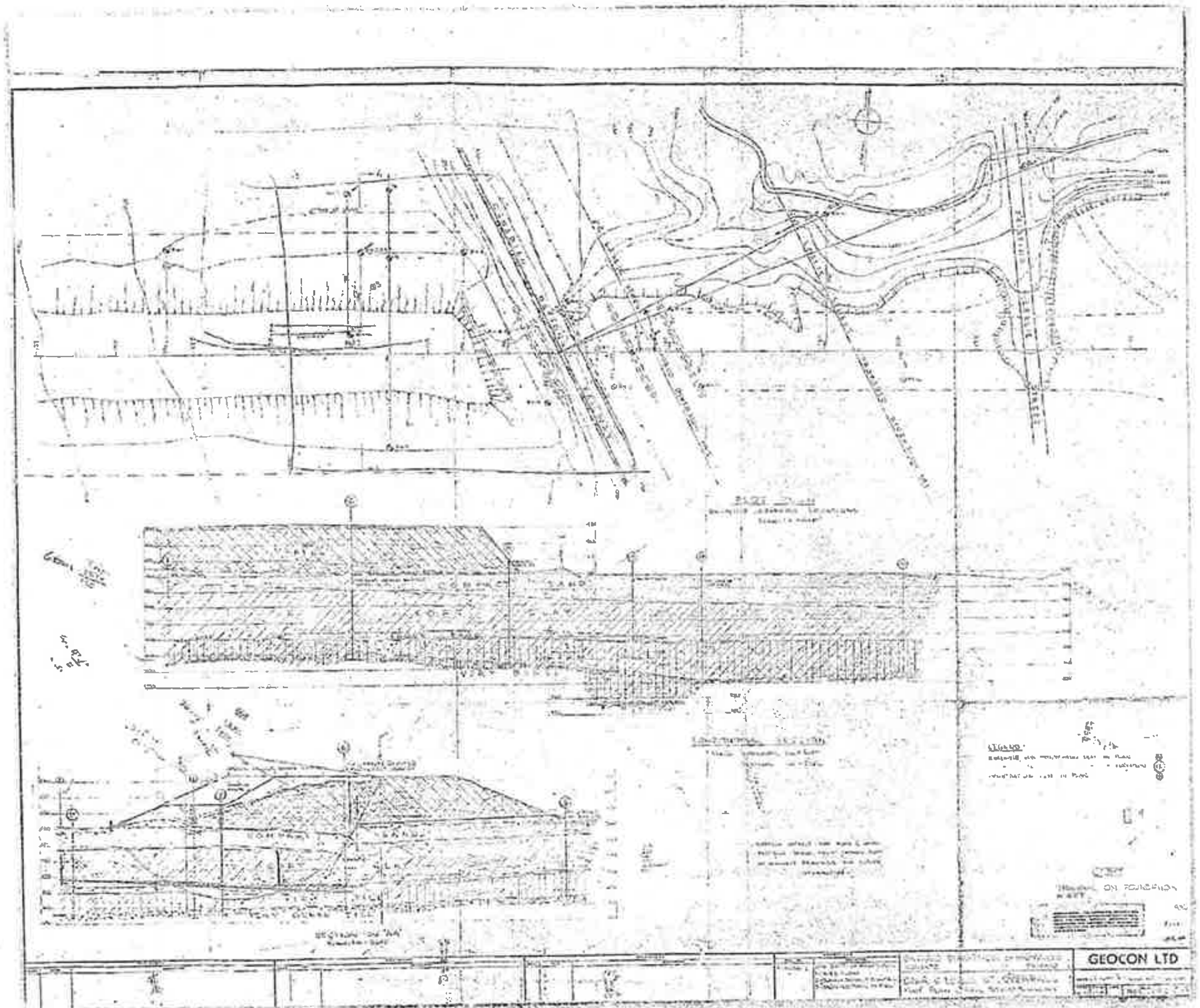
Ramon Miranda, P.Eng.

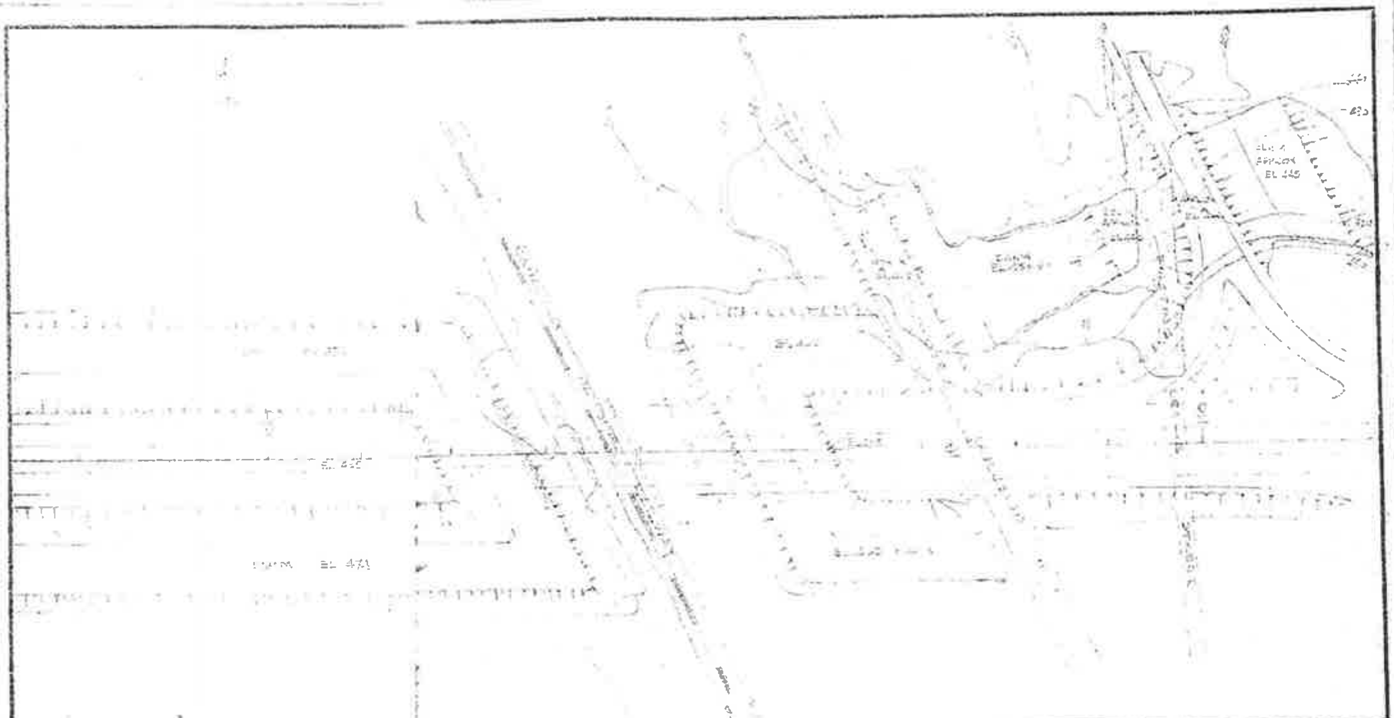
Zuhtu Ozden, P.Eng.

Appendix F

Drawings from Past Reports


LEGEND
STRATIGRAPHY
PLAN
SECTION A-A
DEPARTMENT OF HIGHWAYS ONTARIO
PROPOSED EMBANKMENT
BORING PLAN AND SOIL STRATIGRAPHY
GEOCON LTD
SCALE
DATE
No.


GEOCON LTD



REFERENCE: ABOVE [illegible] ONTARIO
DEPT. OF HIGHWAYS [illegible] 3255-1

REVISIONS		
MARK	DATE	DESCRIPTION

FOUNDATION COMPANIES CANADA

ONTARIO DEPARTMENT OF HIGHWAYS
TORONTO, ONTARIO

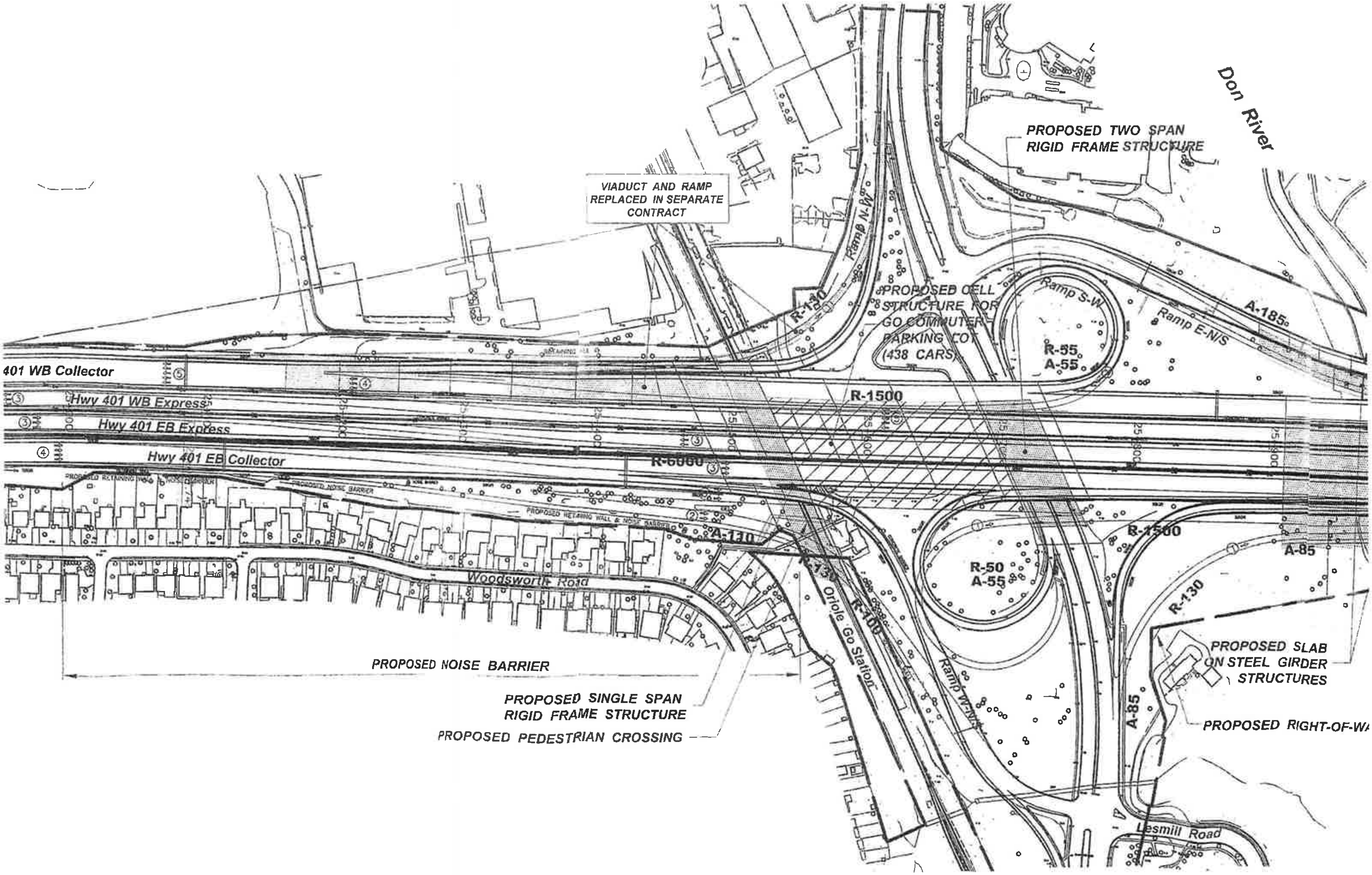
CNR & LESLIE ST OVERPASS
PLAN OF EMBANKMENTS & BERMS

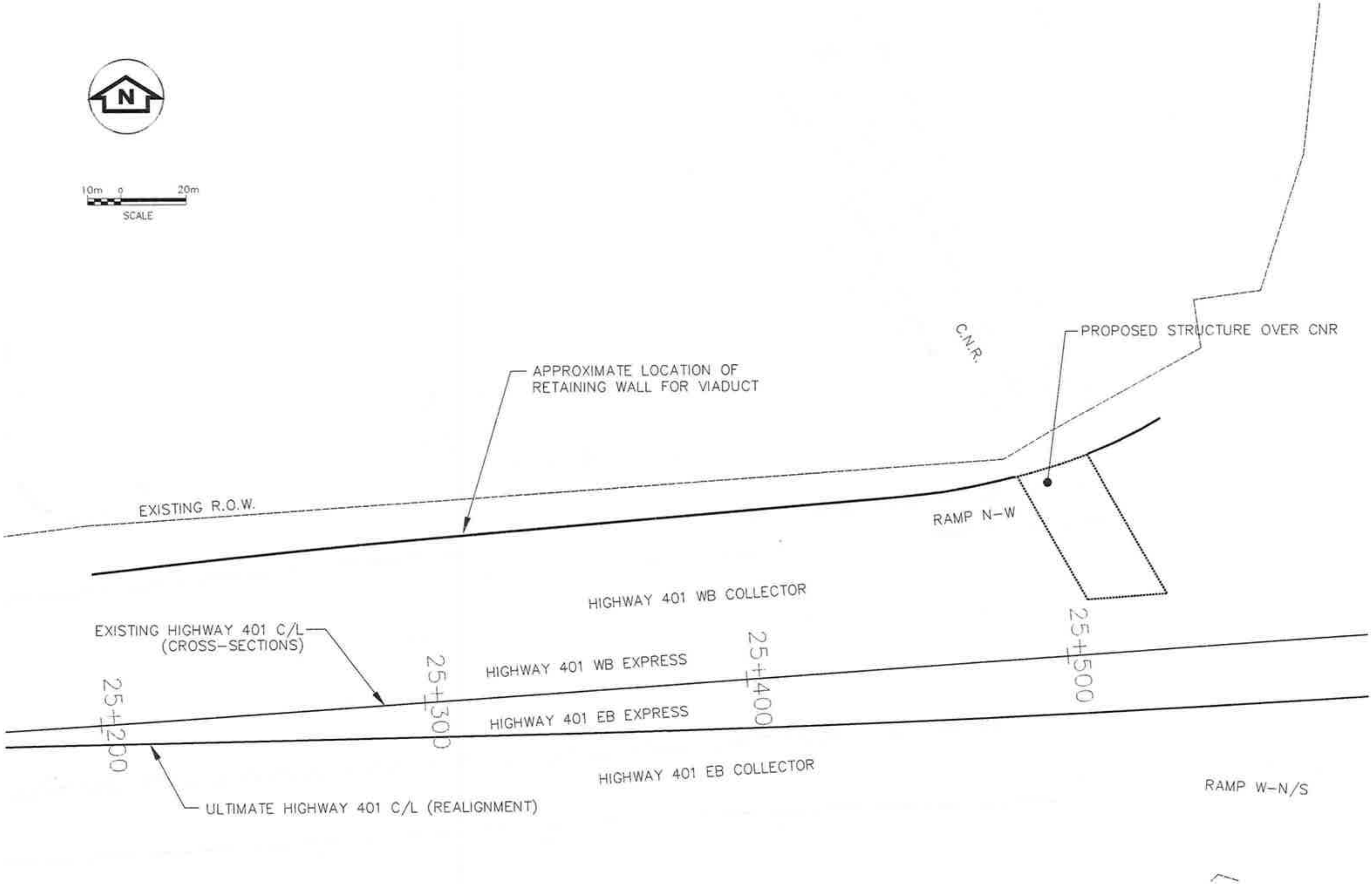
DATE 2 OCT 1955 SCALE 1 IN = 100 FT.

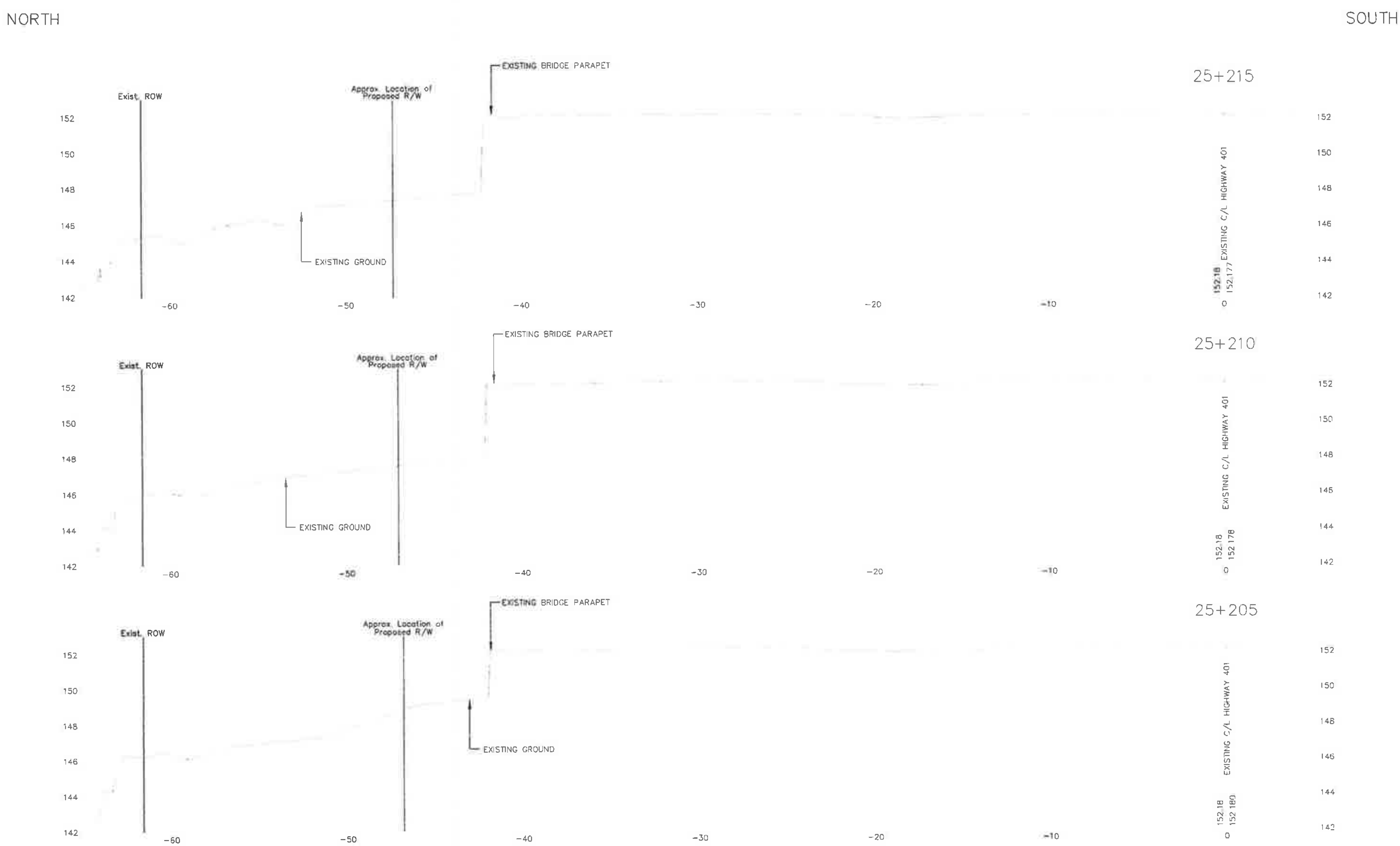
MADE W.A.M. | CHKD. | APPD. | NO. APPENDIX II FIG. 5

Appendix G

Construction Plan, Section and Staging Drawings


Don River
PROPOSED TWO SPAN RIGID FRAME STRUCTURE
VIADUCT AND RAMP REPLACED IN SEPARATE CONTRACT
PROPOSED CELL STRUCTURE FOR GO COMMUTER PARKING LOT (438 CARS)
Ramp N-W
Ramp S-W
Ramp E-N/S
Ramp W-N/S
R-120
R-55
A-55
R-50
A-55
A-185
A-85
R-130
R-1500
R-6000
R-100
A-130
401 WB Collector
Hwy 401 WB Express
Hwy 401 EB Express
Hwy 401 EB Collector
PROPOSED NOISE BARRIER
PROPOSED RETAINING WALL & NOISE BARRIER
Woodsworth Road
Oriole Go Station
PROPOSED SINGLE SPAN RIGID FRAME STRUCTURE
PROPOSED PEDESTRIAN CROSSING
PROPOSED SLAB ON STEEL GIRDER STRUCTURES
PROPOSED RIGHT-OF-WA
Lesmill Road


N
10m
0
20m
SCALE
C.N.R.
PROPOSED STRUCTURE OVER CNR
APPROXIMATE LOCATION OF
RETAINING WALL FOR VIADUCT
EXISTING R.O.W.
RAMP N-W
HIGHWAY 401 WB COLLECTOR
EXISTING HIGHWAY 401 C/L
(CROSS-SECTIONS)
HIGHWAY 401 WB EXPRESS
HIGHWAY 401 EB EXPRESS
25+200
25+300
25+400
25+500
HIGHWAY 401 EB COLLECTOR
RAMP W-N/S
ULTIMATE HIGHWAY 401 C/L (REALIGNMENT)



NORTH
SOUTH
25+215
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.18
152.177
25+210
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.18
152.178
25+205
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.18
152.180
152
150
148
146
144
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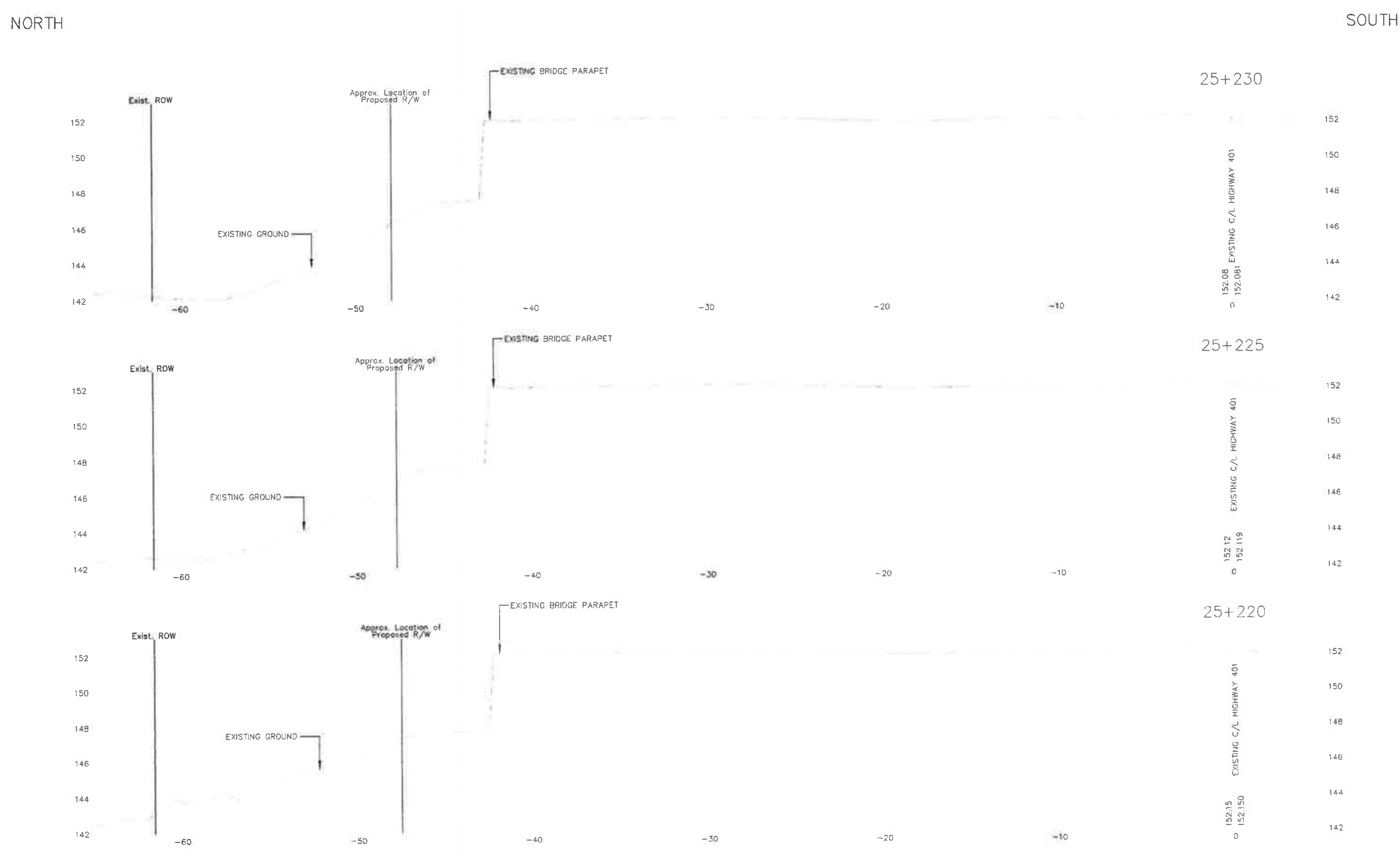
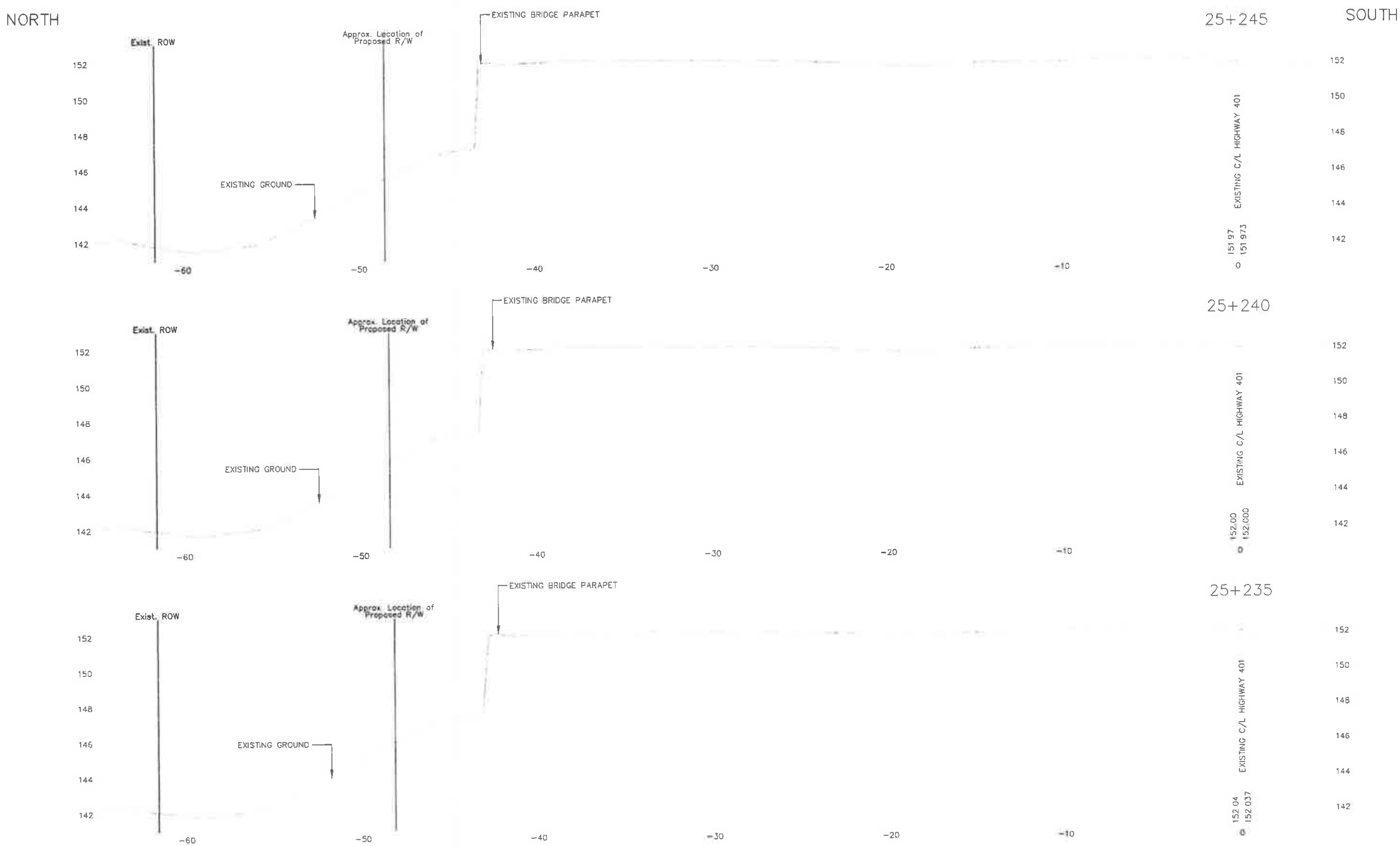
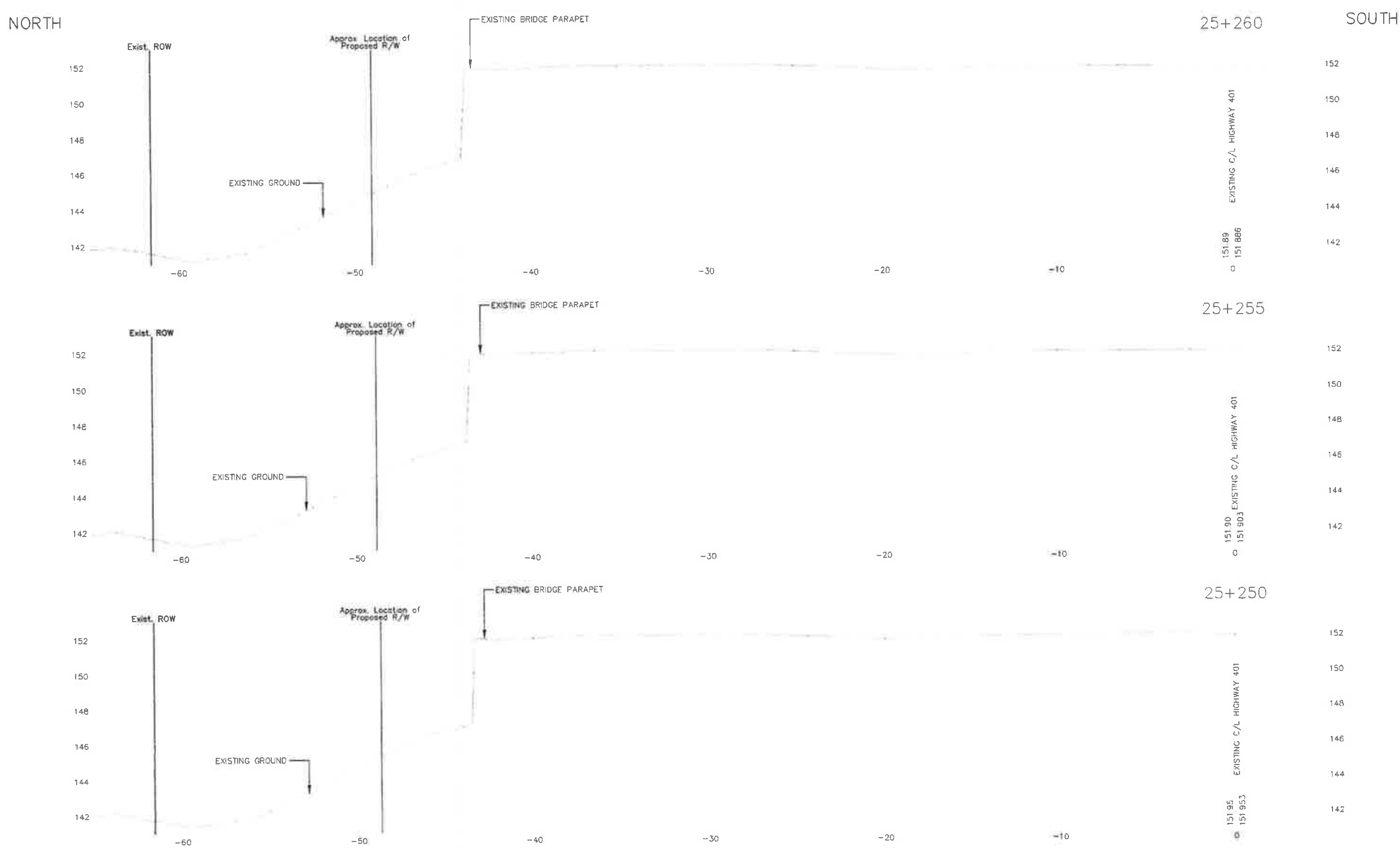
-50
-40
-30
-20
-10
0

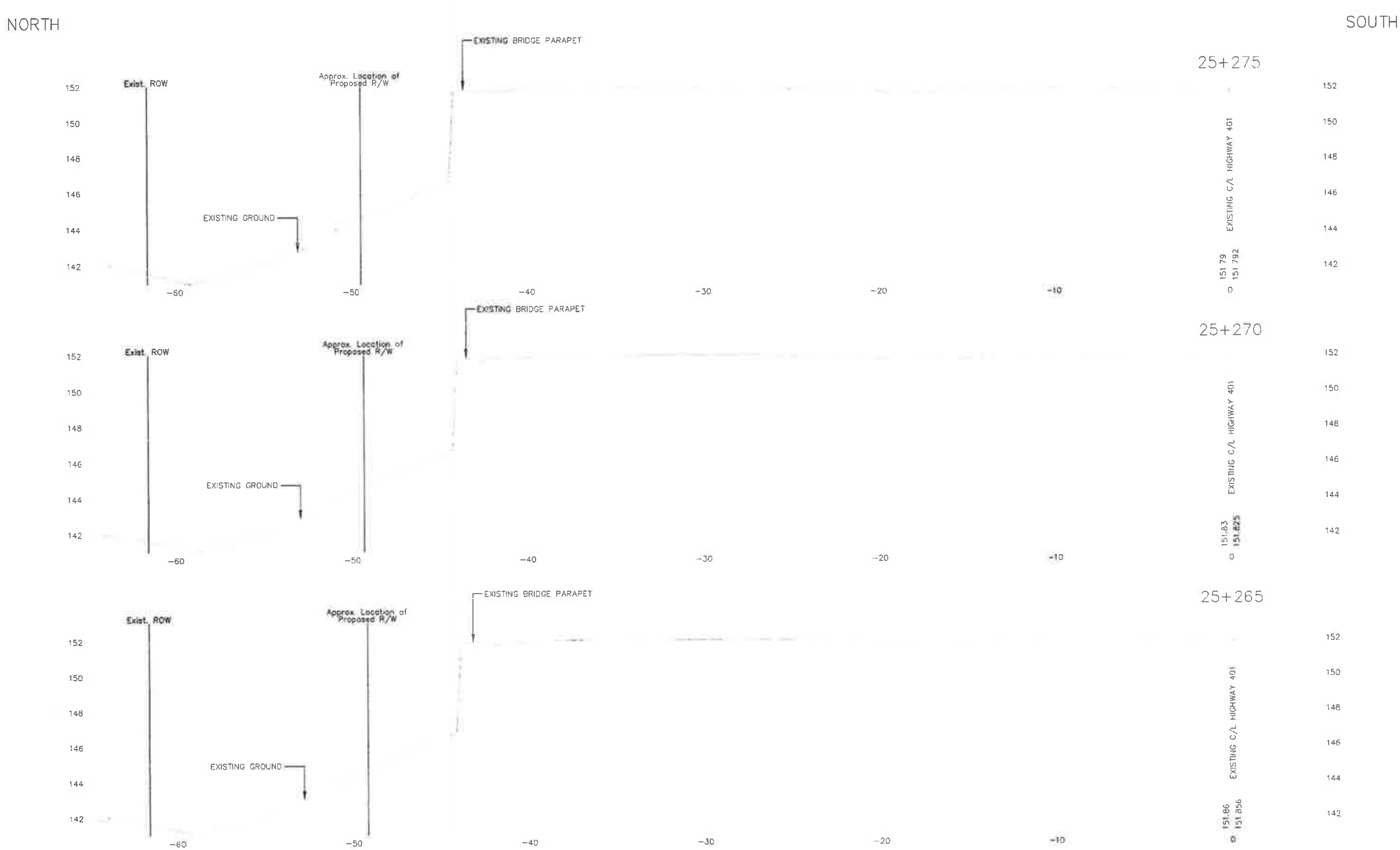


NORTH
SOUTH
25+230
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.08
152.081
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0
25+225
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.12
152.119
25+220
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.15
152.150

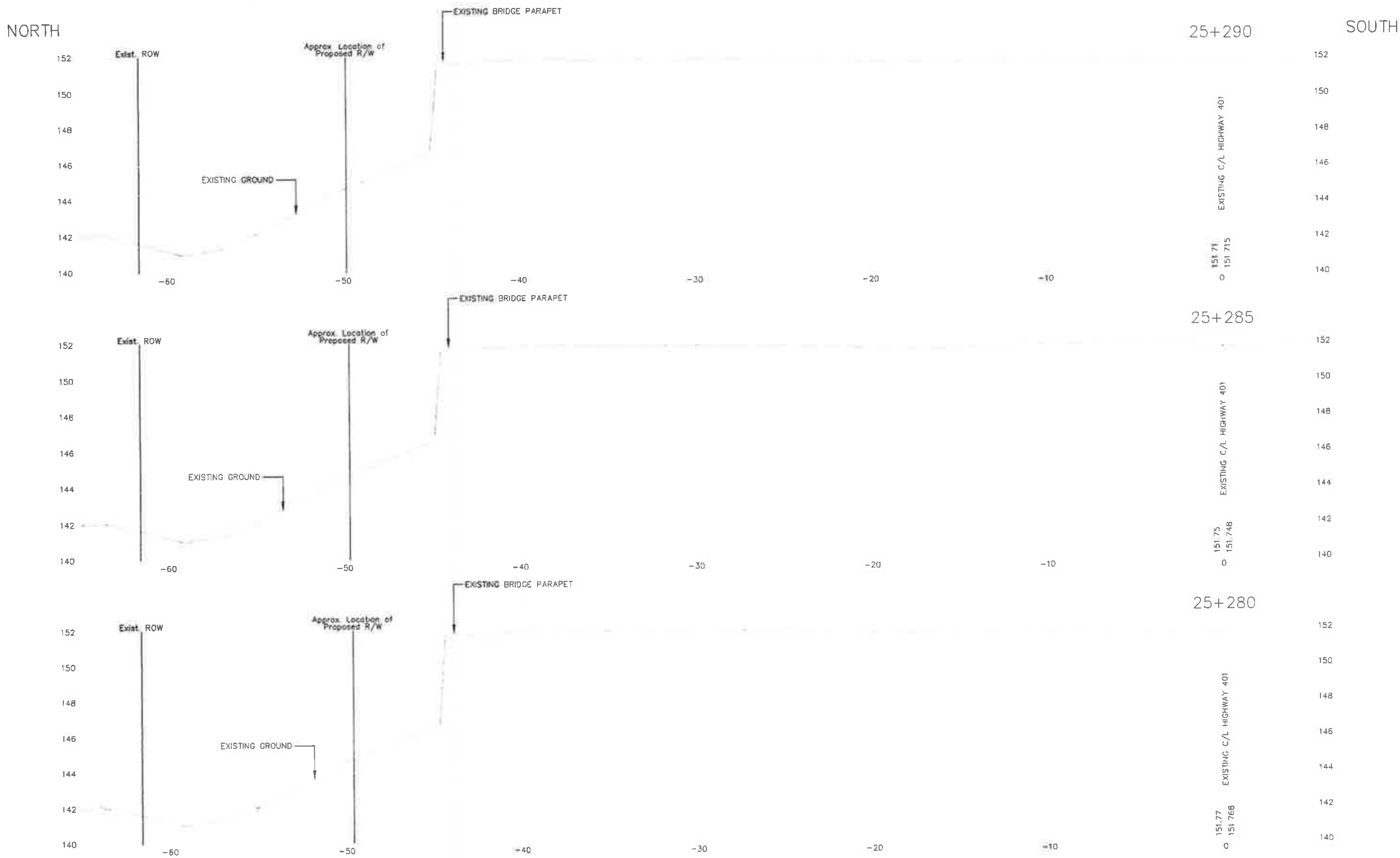

NORTH
SOUTH
25+245
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151 97
151 973
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0
25+240
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152.00
152.000
25+235
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
152 04
152 037

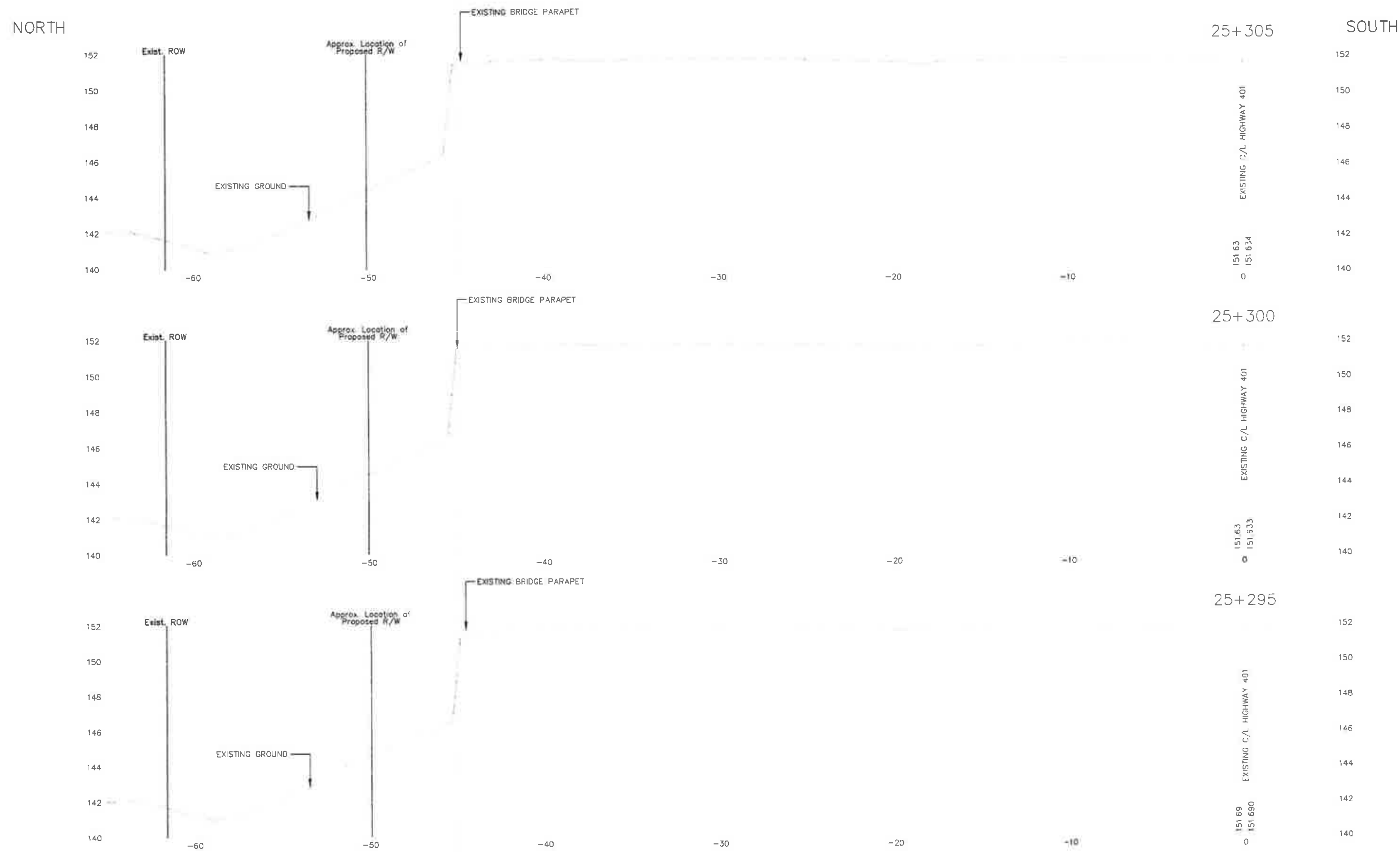


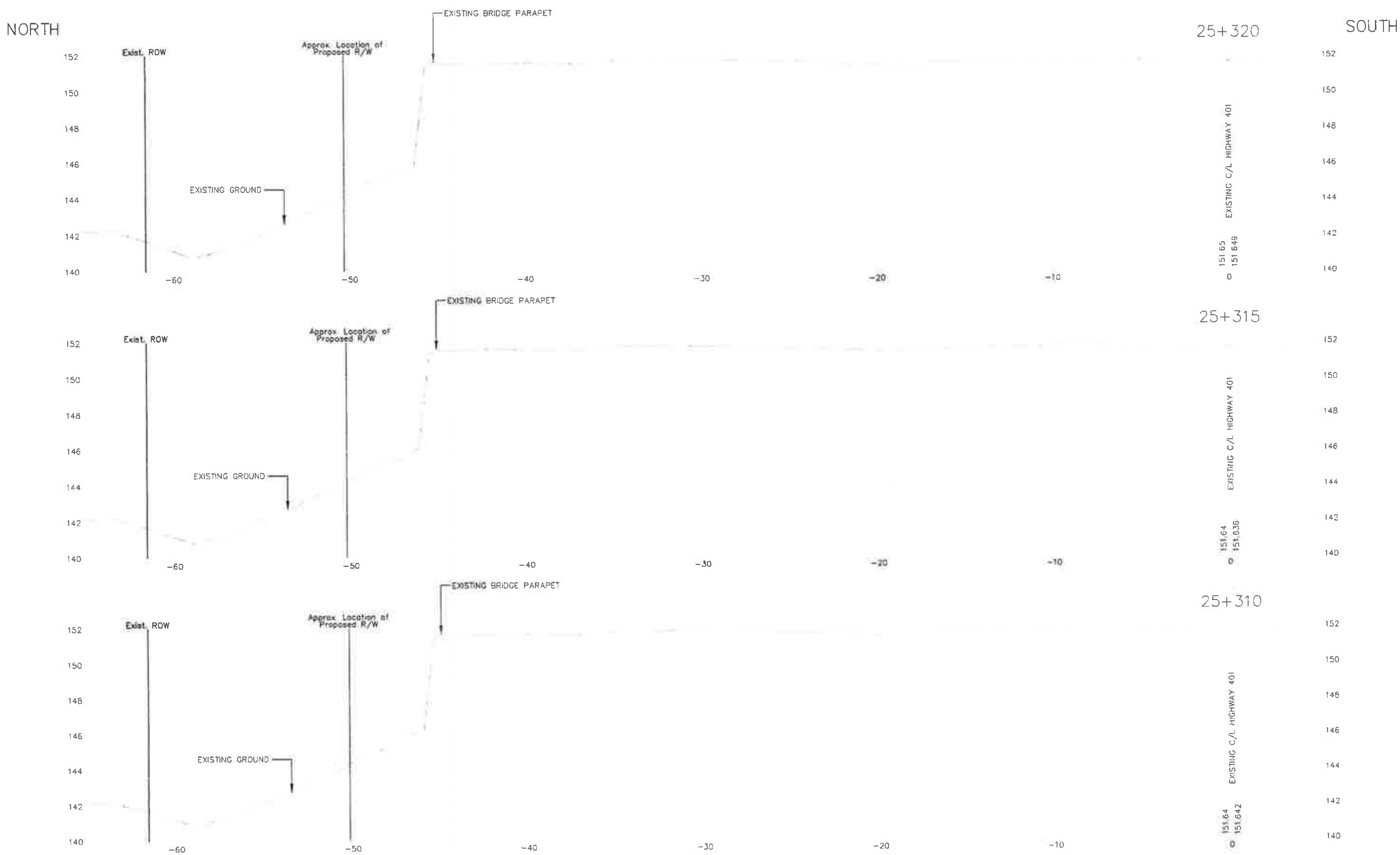
NORTH
SOUTH
25+260
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.89
151 886
25+255
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151 90
151 903
25+250
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151 95
151 953
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0



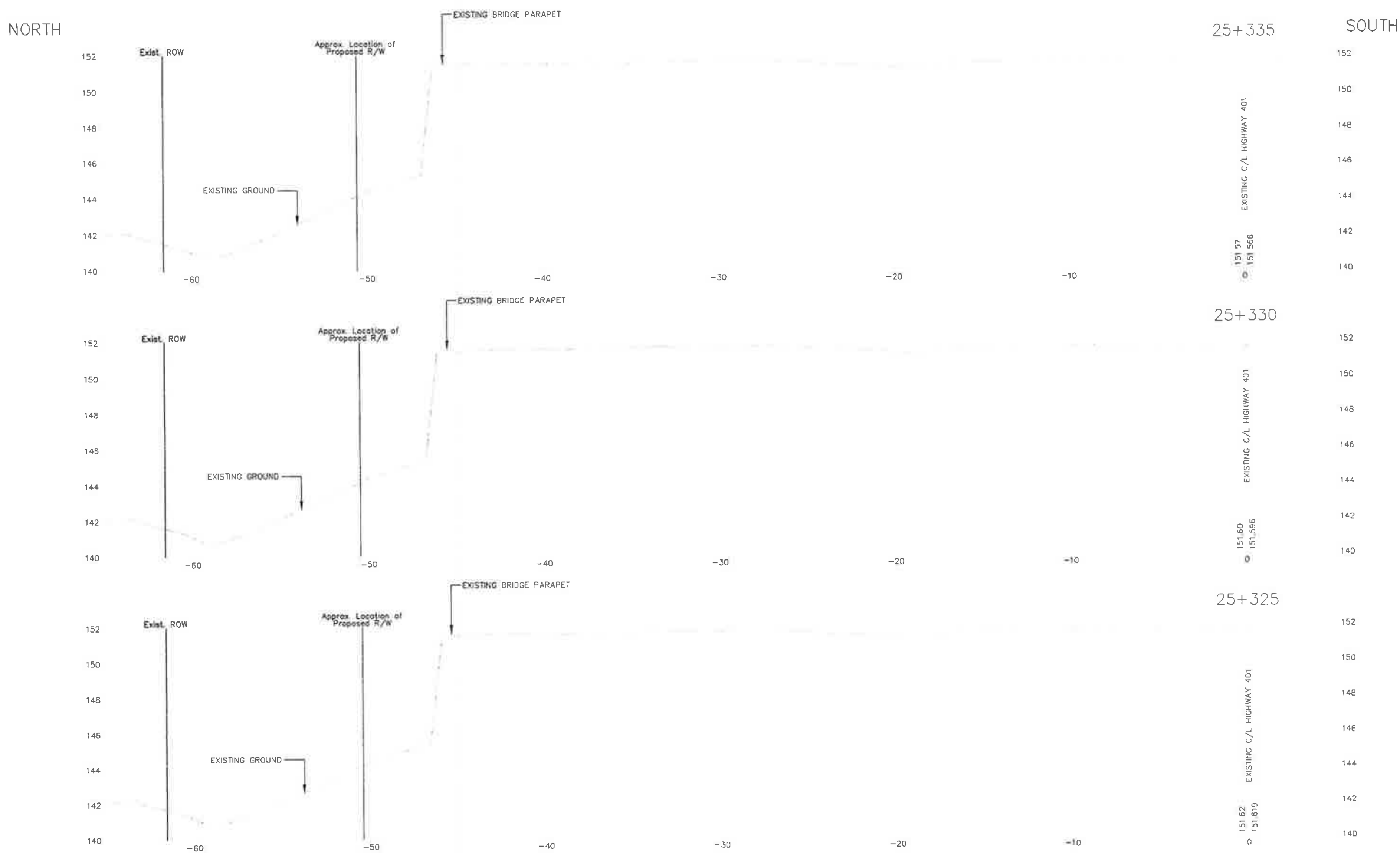
NORTH
SOUTH
25+275
25+270
25+265
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151 79
151 792
151.83
151.825
151.86
151 856
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0

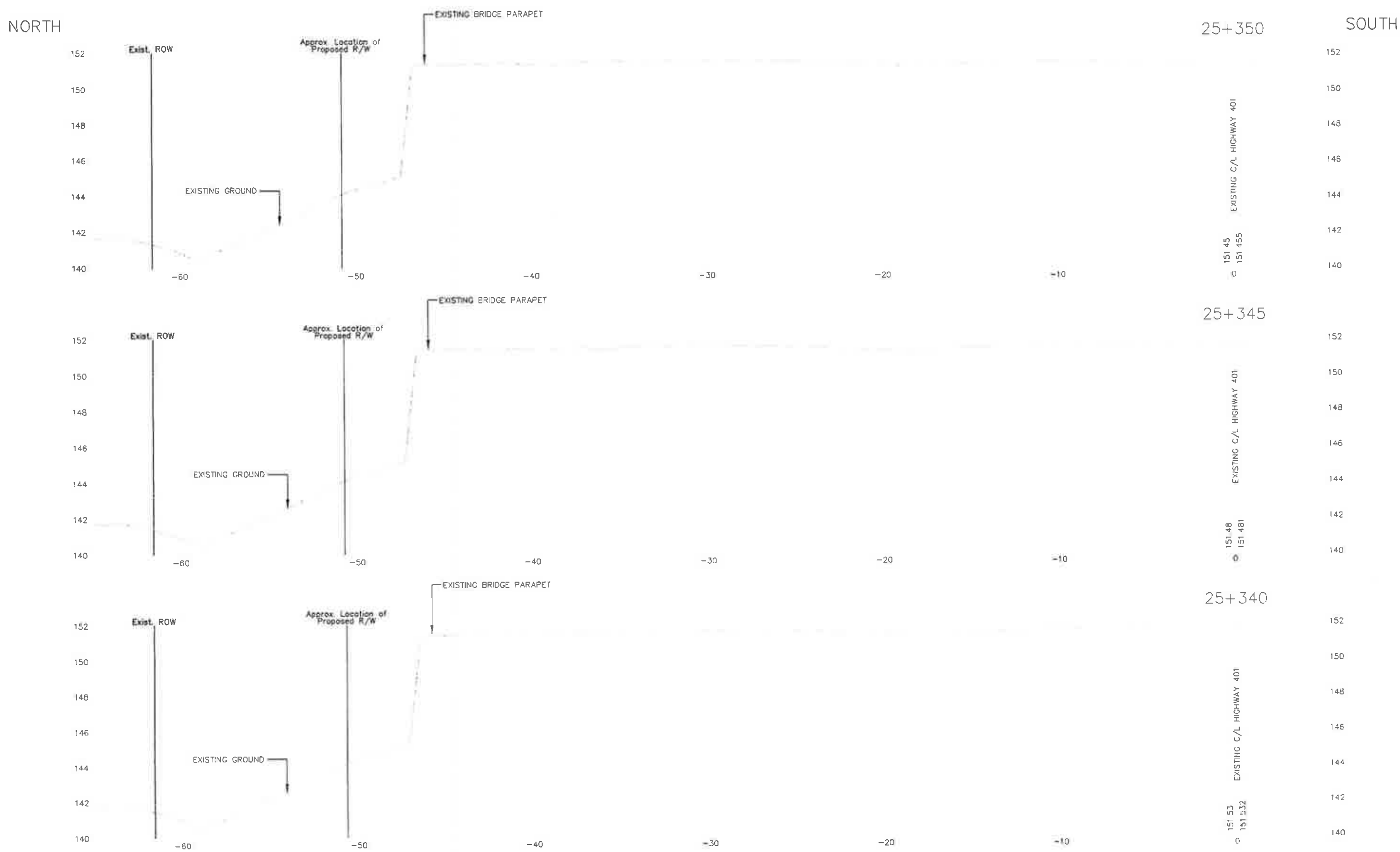

NORTH
SOUTH
25+290
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.71
151.715
25+285
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.75
151.748
25+280
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.77
151.768
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

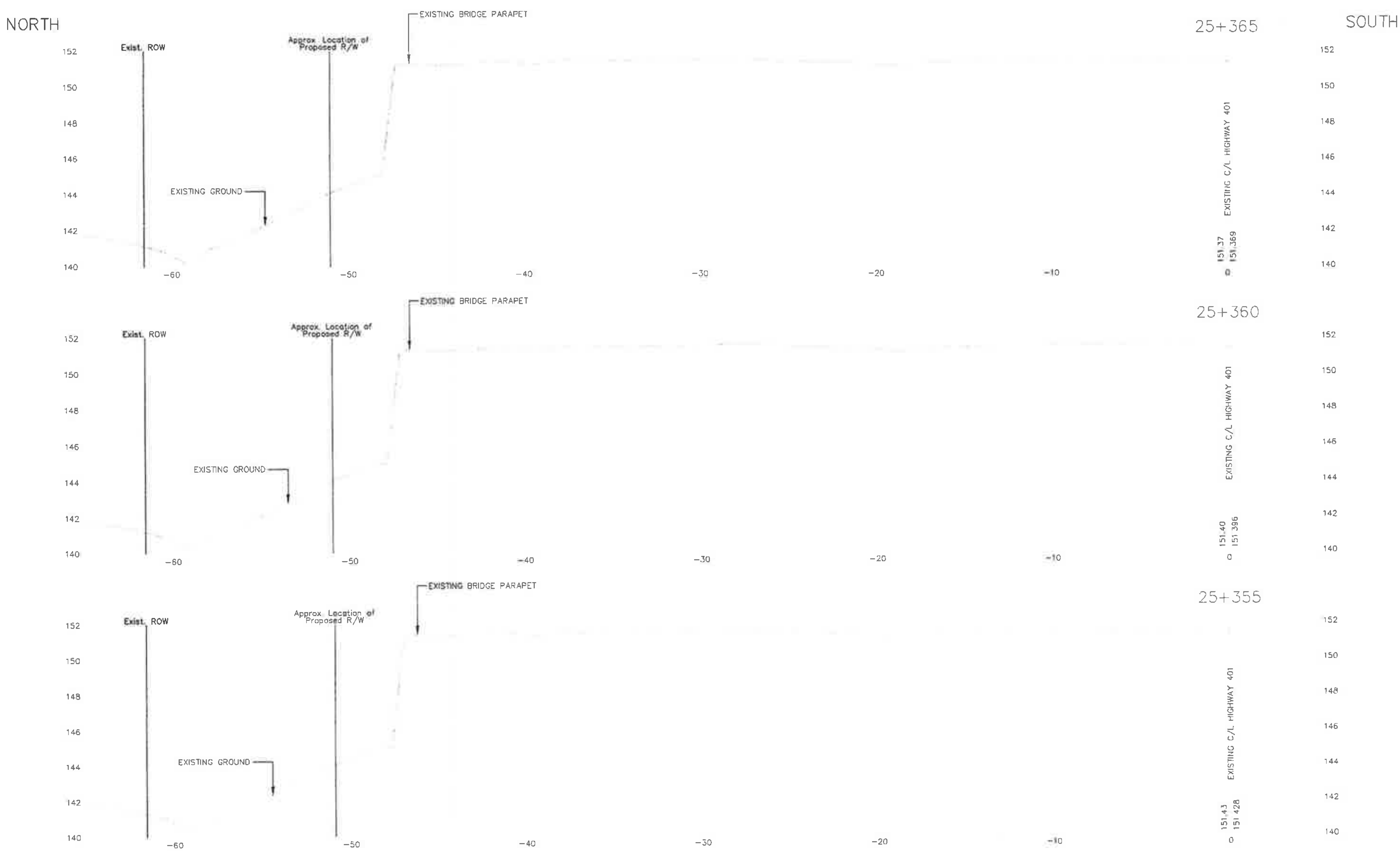

NORTH
SOUTH
25+305
25+300
25+295
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.63
151.634
151.633
151.69
151.690
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

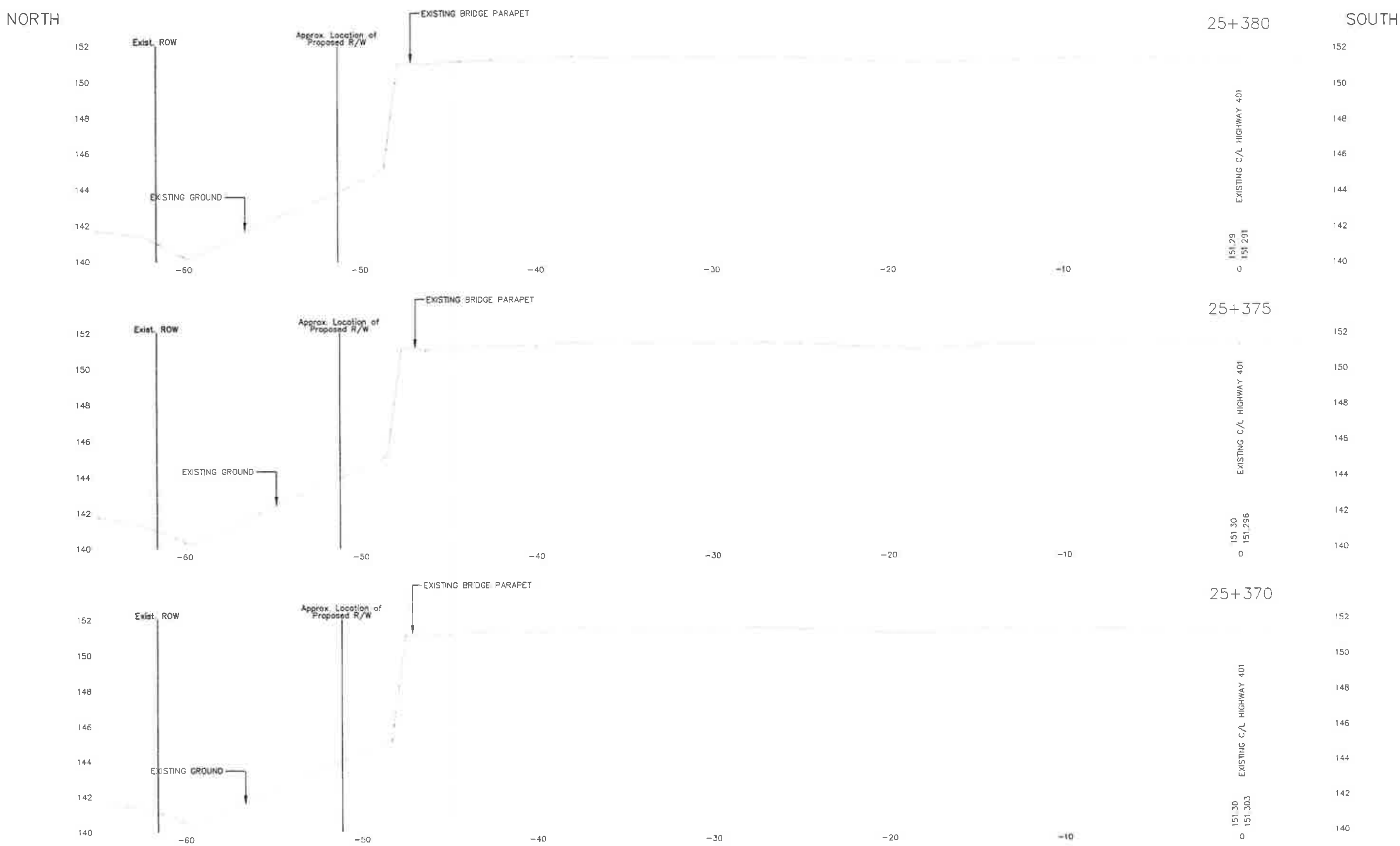


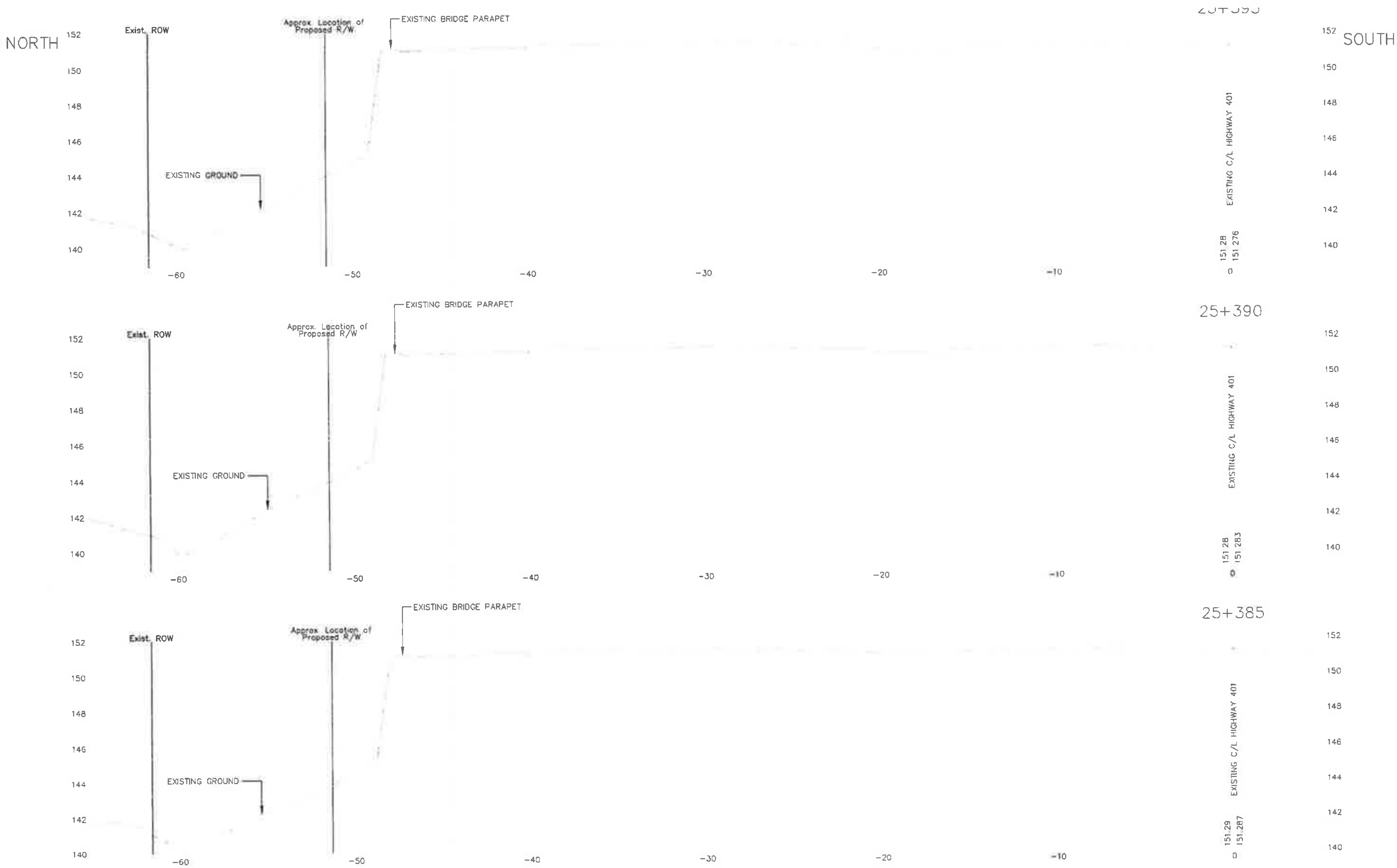
NORTH
SOUTH
25+320
25+315
25+310
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.65
151.649
151.64
151.636
151.64
151.642
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

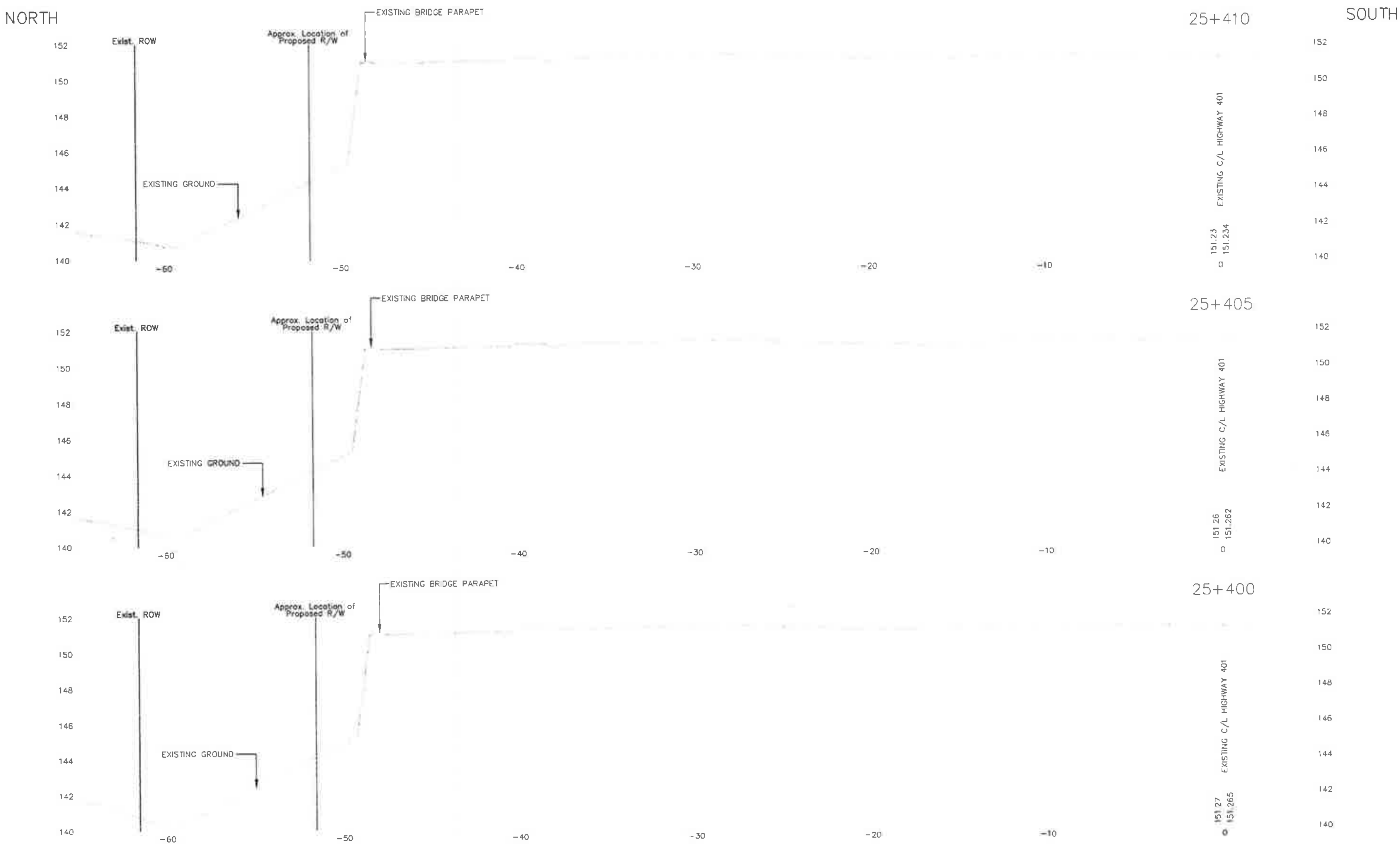

NORTH
SOUTH
25+335
25+330
25+325
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.57
151.566
151.60
151.596
151.62
151.619
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

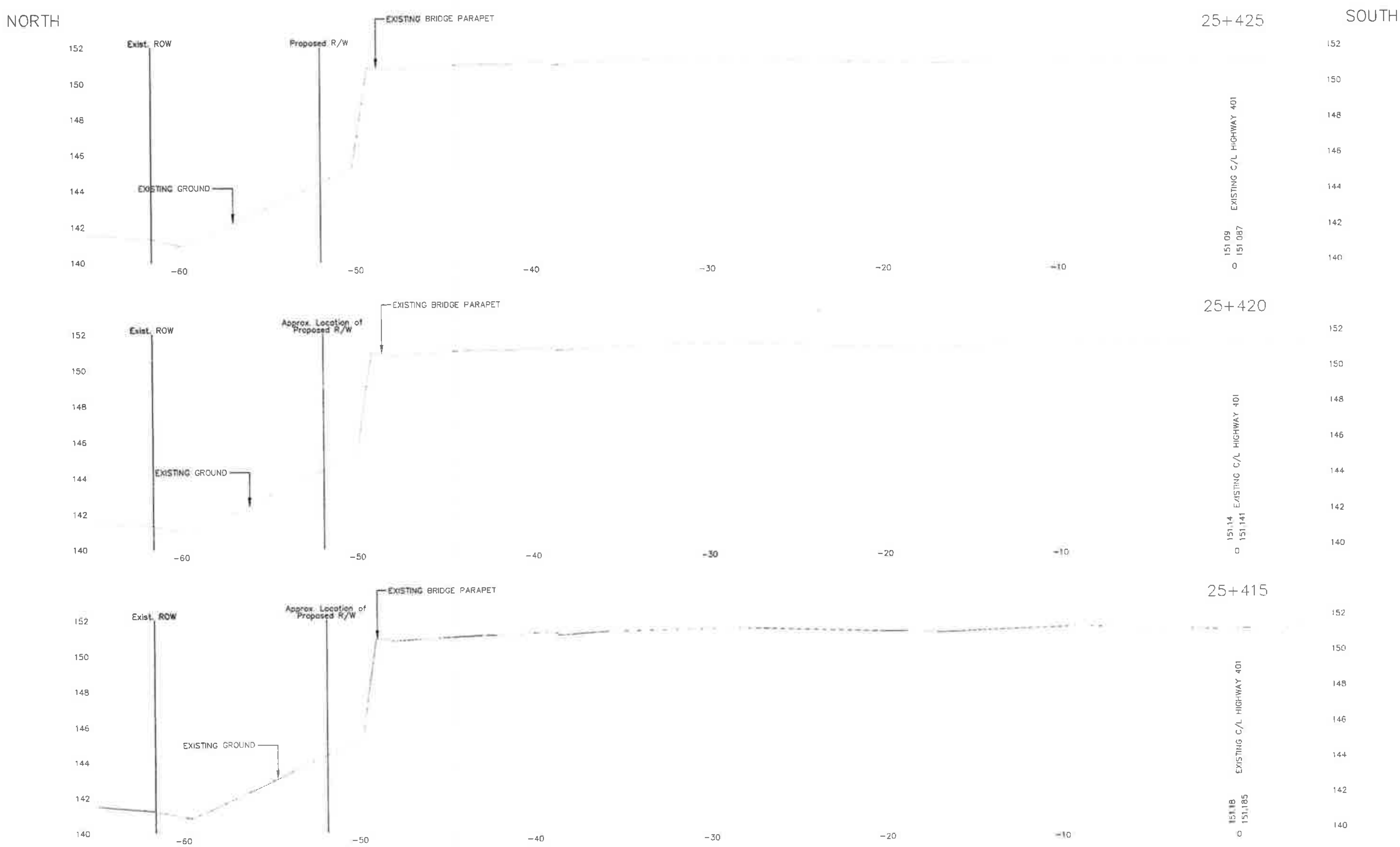

NORTH
SOUTH
25+350
25+345
25+340
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151 45
151 455
151 48
151 481
151 53
151 532
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

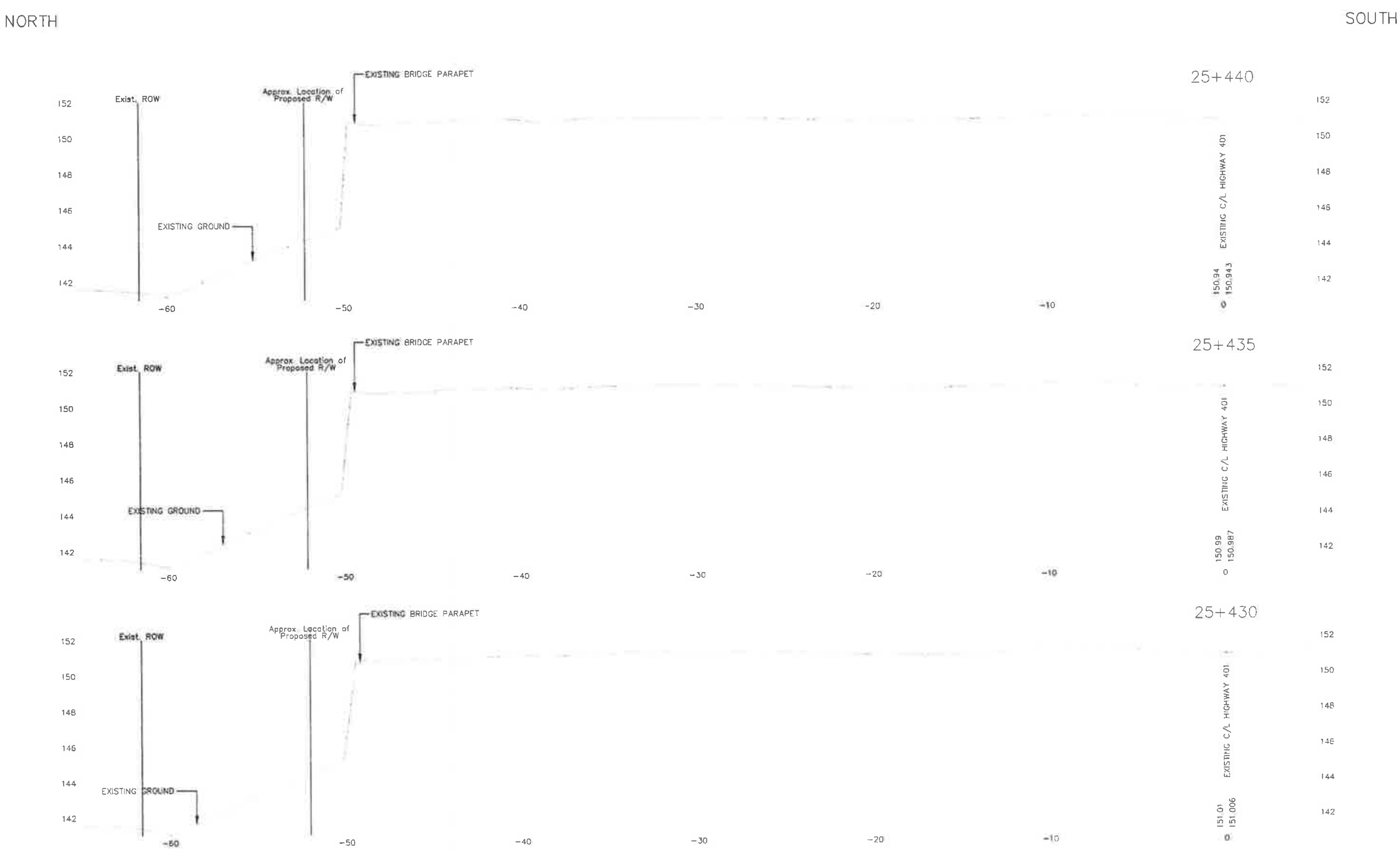

NORTH
SOUTH
25+365
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.37
151.369
25+360
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.40
151.396
25+355
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.43
151.428
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0


NORTH
SOUTH
25+380
25+375
25+370
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.29
151.291
151.30
151.296
151.30
151.303
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

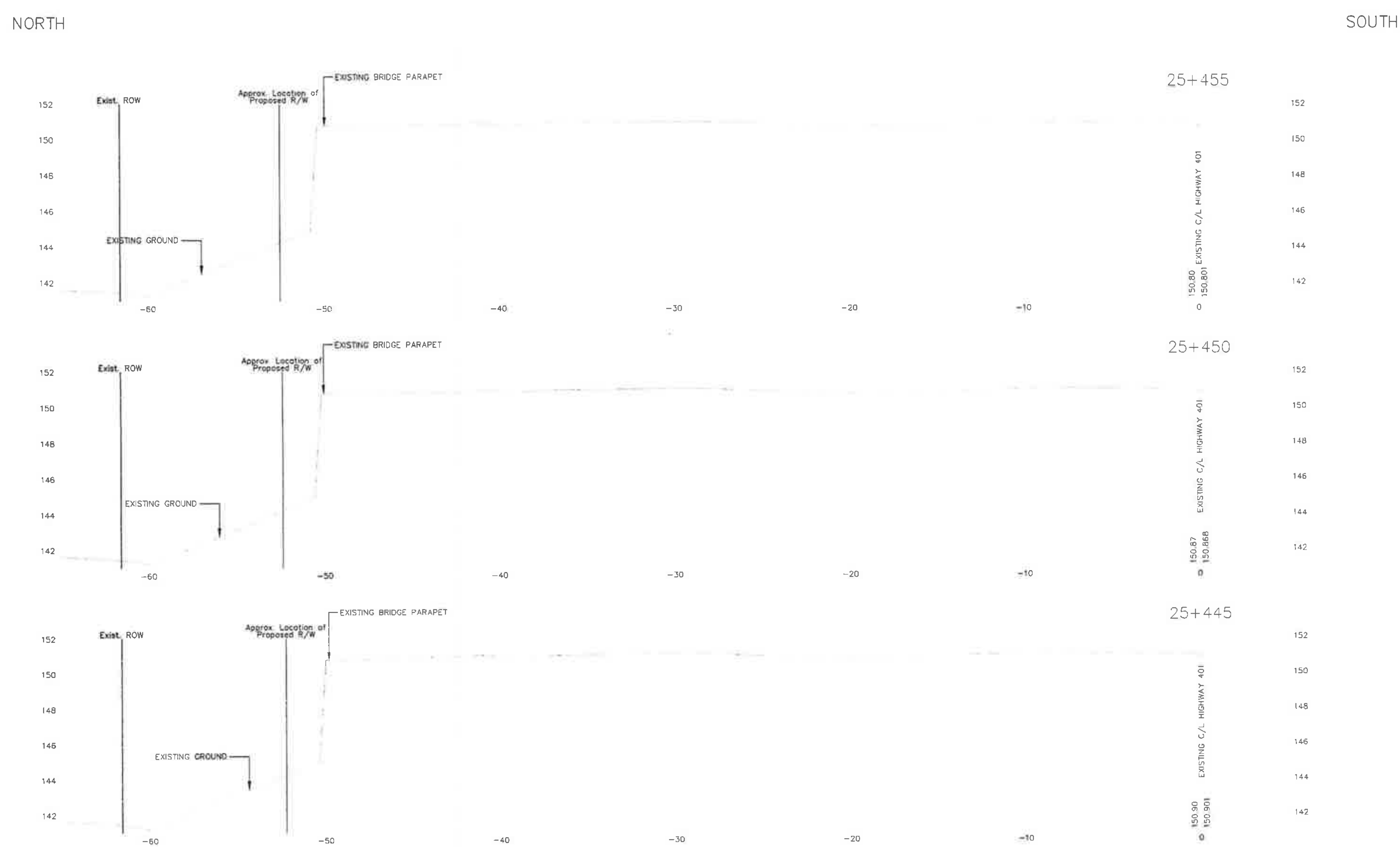

NORTH
SOUTH
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151 28
151 276
25+390
151 28
151 283
25+385
151.29
151.287
152
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0

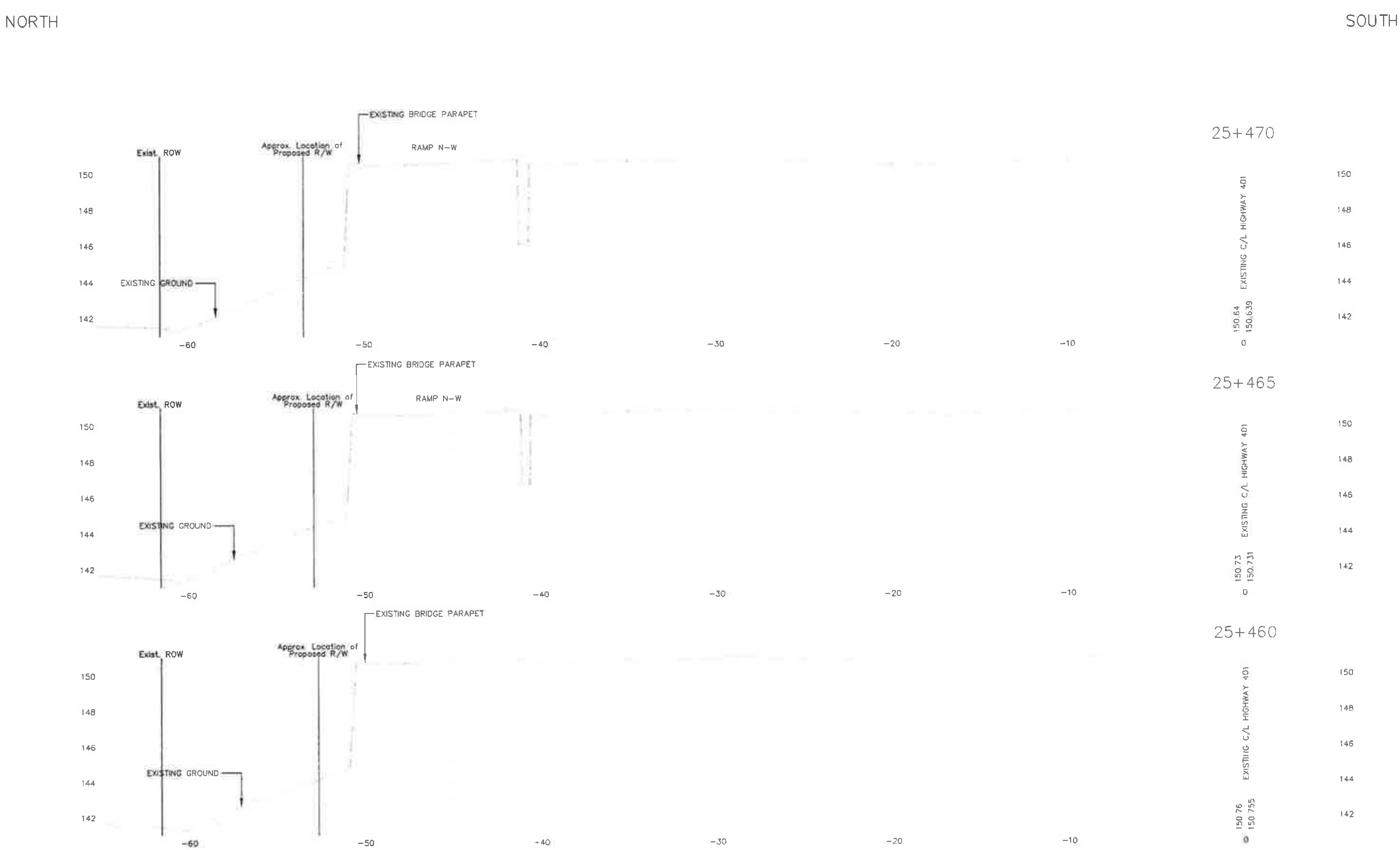

NORTH
SOUTH
25+410
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.23
151.234
25+405
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.26
151.262
25+400
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.27
151.265


NORTH
SOUTH
25+425
Exist. ROW
Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.09
151.087
25+420
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.14
151.141
25+415
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.18
151.185

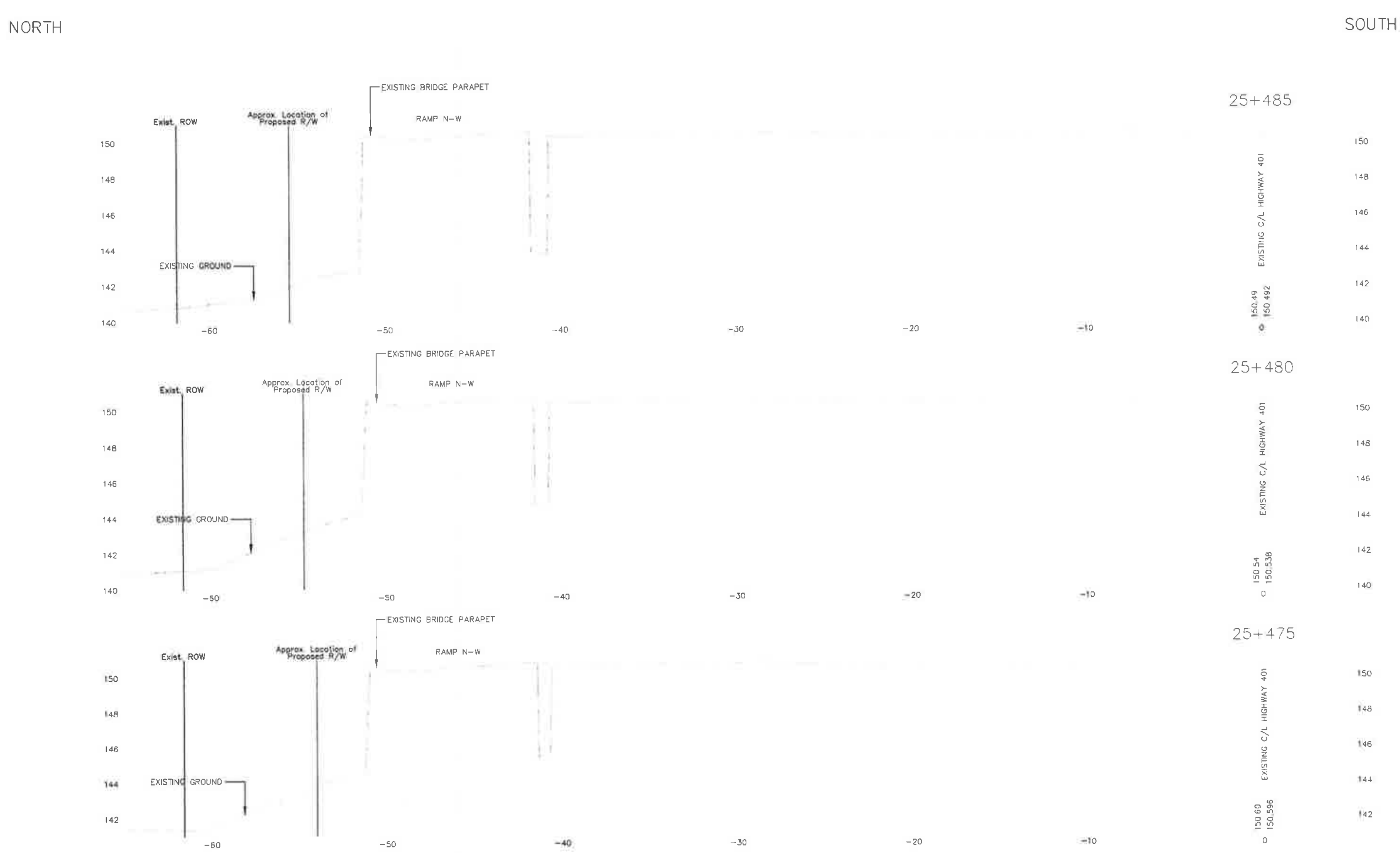


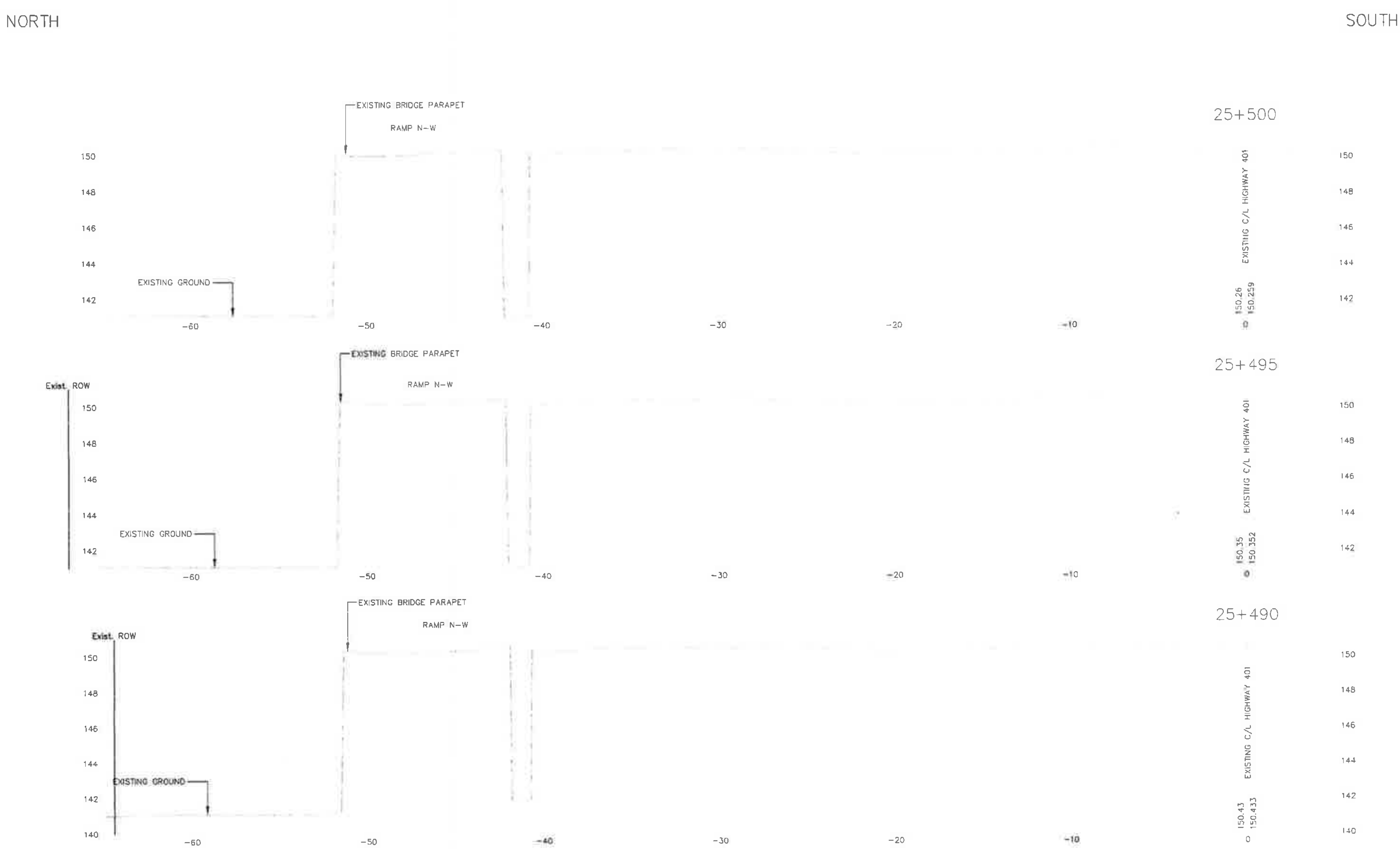
NORTH
SOUTH
25+440
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.94
150.943
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0
25+435
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.99
150.987
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0
25+430
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
151.01
151.006
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0

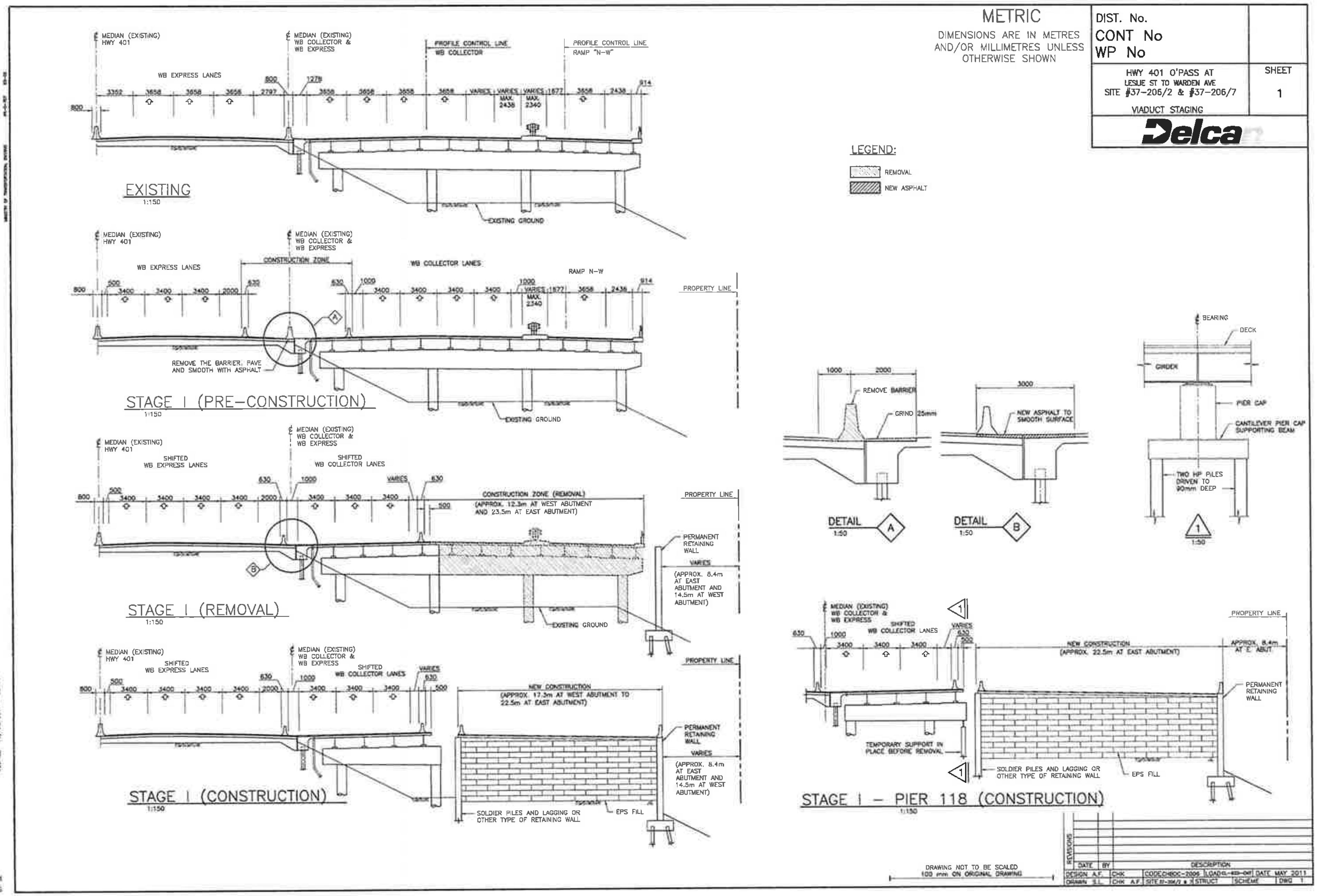

NORTH
SOUTH
25+455
25+450
25+445
Exist. ROW
Approx. Location of Proposed R/W
EXISTING BRIDGE PARAPET
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.80
150.801
150.87
150.868
150.90
150.901
152
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0



NORTH
SOUTH
25+470
EXISTING BRIDGE PARAPET
RAMP N-W
Exist. ROW
Approx. Location of Proposed R/W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.64
150.639
25+465
EXISTING BRIDGE PARAPET
RAMP N-W
Exist. ROW
Approx. Location of Proposed R/W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.73
150.731
25+460
EXISTING BRIDGE PARAPET
Exist. ROW
Approx. Location of Proposed R/W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.76
150.755
150
148
146
144
142
-60
-50
-40
-30
-20
-10
0


NORTH
SOUTH
25+485
EXISTING BRIDGE PARAPET
RAMP N-W
Exist. ROW
Approx. Location of Proposed R/W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.49
150.492
25+480
EXISTING BRIDGE PARAPET
RAMP N-W
Exist. ROW
Approx. Location of Proposed R/W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.54
150.538
25+475
EXISTING BRIDGE PARAPET
RAMP N-W
Exist. ROW
Approx. Location of Proposed R/W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.60
150.596


NORTH
SOUTH
25+500
EXISTING BRIDGE PARAPET
RAMP N-W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.26
150.259
25+495
Exist. ROW
EXISTING BRIDGE PARAPET
RAMP N-W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.35
150.352
25+490
Exist. ROW
EXISTING BRIDGE PARAPET
RAMP N-W
EXISTING GROUND
EXISTING C/L HIGHWAY 401
150.43
150.433
150
148
146
144
142
140
-60
-50
-40
-30
-20
-10
0


METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN
DIST. No.
CONT No
WP No
HWY 401 O'PASS AT LESLIE ST TO WARDEN AVE SITE #37-206/2 & #37-206/7
VIADUCT STAGING
SHEET
1
Delcan
LEGEND:
REMOVAL
NEW ASPHALT
EXISTING
1:150
STAGE I (PRE-CONSTRUCTION)
1:150
STAGE I (REMOVAL)
1:150
STAGE I (CONSTRUCTION)
1:150
STAGE I - PIER 118 (CONSTRUCTION)
1:150
DETAIL A
1:50
DETAIL B
1:50
1
1:50
MEDIAN (EXISTING) HWY 401
MEDIAN (EXISTING) WB COLLECTOR & WB EXPRESS
WB EXPRESS LANES
WB COLLECTOR LANES
SHIFTED WB EXPRESS LANES
SHIFTED WB COLLECTOR LANES
PROFILE CONTROL LINE WB COLLECTOR
PROFILE CONTROL LINE RAMP "N-W"
RAMP N-W
CONSTRUCTION ZONE
PROPERTY LINE
EXISTING GROUND
REMOVE THE BARRIER, PAVE AND SMOOTH WITH ASPHALT
CONSTRUCTION ZONE (REMOVAL) (APPROX. 12.3m AT WEST ABUTMENT AND 23.5m AT EAST ABUTMENT)
PERMANENT RETAINING WALL
VARIES
(APPROX. 8.4m AT EAST ABUTMENT AND 14.5m AT WEST ABUTMENT)
NEW CONSTRUCTION (APPROX. 17.3m AT WEST ABUTMENT TO 22.5m AT EAST ABUTMENT)
SOLDIER PILES AND LAGGING OR OTHER TYPE OF RETAINING WALL
EPS FILL
NEW CONSTRUCTION (APPROX. 22.5m AT EAST ABUTMENT)
APPROX. 8.4m AT E. ABUT
TEMPORARY SUPPORT IN PLACE BEFORE REMOVAL
REMOVE BARRIER
GRIND 25mm
NEW ASPHALT TO SMOOTH SURFACE
BEARING
DECK
GIRDER
PIER CAP
CANTILEVER PIER CAP SUPPORTING BEAM
TWO HP PILES DRIVEN TO 90mm DEEP
DRAWING NOT TO BE SCALED 100 mm ON ORIGINAL DRAWING
REVISIONS
DATE
BY
DESCRIPTION
DESIGN A.F.
CHK
CODE CHBDC-2006
DATE MAY 2011
DRAWN S.L.
CHK A.F.
STRUCT
SCHEME
DWG 1

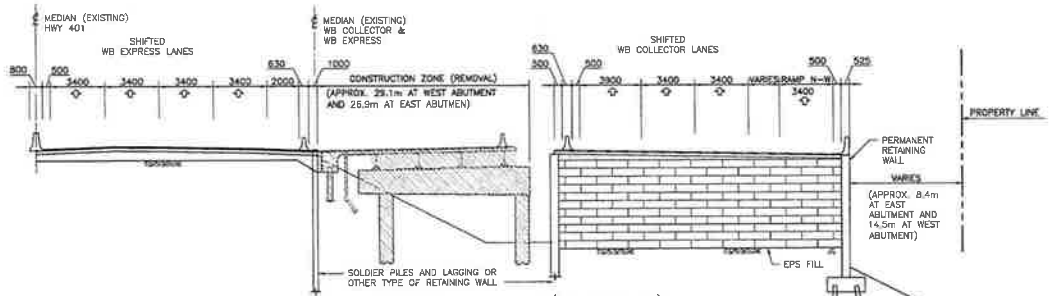
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

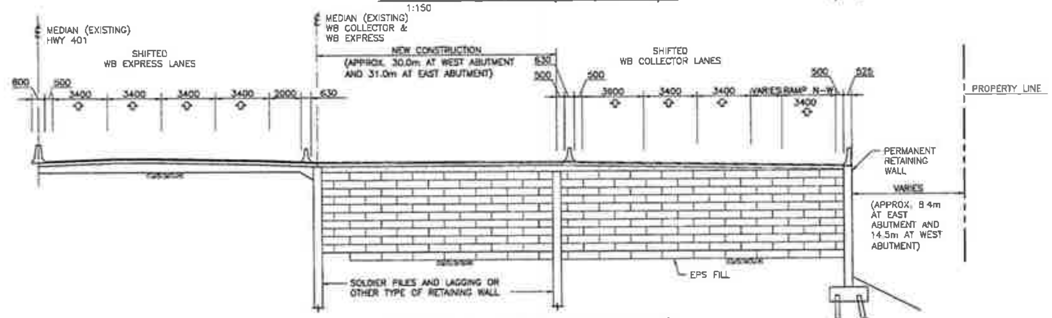
LEGEND:

REMOVAL

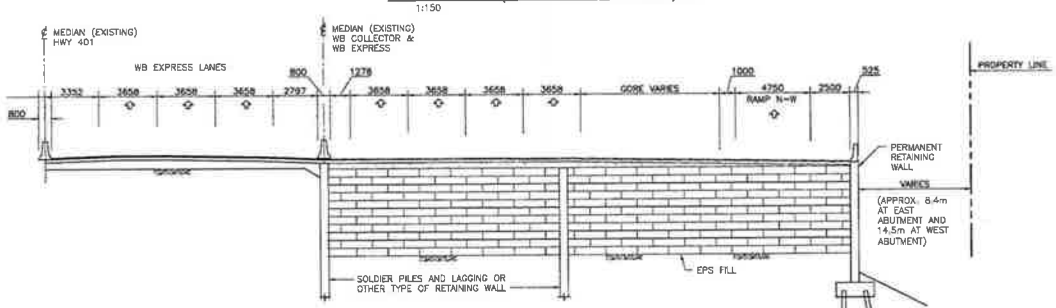
DIST. No. CONT No WP No	
HWY 401 O'PASS AT LESLIE ST TO WARDEN AVE SITE #37-206/2 & #37-206/7 VIADUCT STAGING	SHEET 2
Delcan	



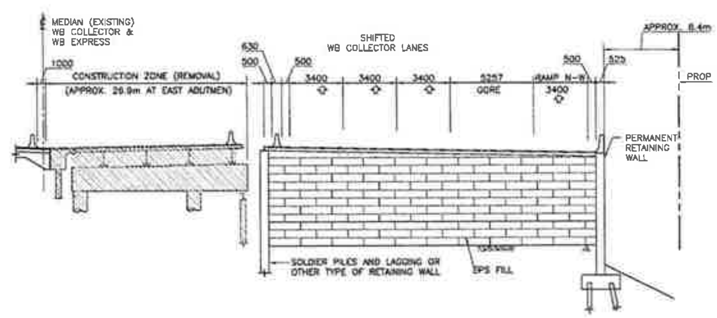
STAGE II TYP. (REMOVAL)
1:150



STAGE II (CONSTRUCTION)
1:150



FINAL STAGE (ULTIMATE)
1:150



STAGE II – PIER 118 ONLY (REMOVAL)
1:150

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		
DATE	BY	DESCRIPTION
DESIGN A.F.	CHK	CODE CHBDC-2006
DRAWN S.L.	CHK A.F.	SITE 37-206/2 & 37-206/7 STRUCT

LOAD CL-625-ONT DATE MAY 2011 SCHEME DWG 2

Appendix H

Foundation Elements-highway 401 and Leslie Street Interchange

Table H-1 Foundation Elements - Highway 401 and Leslie Street Interchange
Project Number: TRANETOB01245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-builts	Depth	Diameter	Base Diameter	Battered	Notes
											Legend and Notes Signifies event took place Not Clear indicates data either not available or could not be read drawing.
			(ft)		(in)	(in)	(m)	(mm)	(mm)		Note data based on interpretation of drawings and/or tables. No as-built construction records were located.
37-206/1	418	12BP53 H-Pile	150	*			45.7			1 to 3	two piles went to 45 m depth during the installation
	418	12BP53 H-Pile	88	*			26.8				
Hwy 401 over pass at Leslie	419	Concrete Caisson			36						working load 250 Ton/Caisson
EBL Collectors	420	12BP53 H-Pile	n/a	n/a			n/a			1 to 6	design load 60 Ton/pile
(16 Spans)	421	12BP53 H-Pile	n/a	n/a			n/a			1 to 6	design load 60Ton/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		30	42	24.7	762	1067		
	424	Concrete Caisson#2	78		30	42	23.8	762	1067		
	424-3	12BP53 H-Pile									caisson blow-out/Replaced by H-Pile, 60 Ton/Pile
	425	Concrete Caisson#1	82		30	42	25.0	762	1067		
	425	Concrete Caisson#2	76		30	42	23.2	762	1067		
	425	Concrete Caisson#3	78		30	42	23.8	762	1067		
	426	Concrete Caisson#1	81		30	42	24.7	762	1067		
	426	Concrete Caisson#2	78		30	42	23.8	762	1067		
	426-3	12BP53 H-Pile									caisson blow-out/Replaced by H-Pile, 60 Ton/Pile
	427	Concrete Caisson#1	77		30	42	23.5	762	1067		
	427	Concrete Caisson#2	77		30	42	23.5	762	1067		
	427	Concrete Caisson#3	76		30	42	23.2	762	1067		
	428	Concrete Caisson#1	79		30	42	24.1	762	1067		
	428	Concrete Caisson#2	78		30	42	23.8	762	1067		
	428	Concrete Caisson#3	76		30	42	23.2	762	1067		
	429	Concrete Caisson#1	76		30	42	23.2	762	1067		
	429	Concrete Caisson#2	77		30	42	23.5	762	1067		
	429	Concrete Caisson#3	70		30	42	21.3	762	1067		
	430	Concrete Caisson#1	76		30	42	23.2	762	1067		
	430	Concrete Caisson#2	76		30	42	23.2	762	1067		
	430	Concrete Caisson#3	76		30	42	23.2	762	1067		
	430	Concrete Caisson#4	76		30	42	23.2	762	1067		
	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 3	

Table H-1 Foundation Elements - Highway 401 and Leslie Street Interchange
Project Number: TRANETOB01245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-builts	Depth	Diameter	Base Diameter	Battered	Notes Legend and Notes Signifies event took place Not Clear indicates data either not available or could not be read drawing.
			(ft)		(in)	(in)	(m)	(mm)	(mm)		Note data based on interpretation of drawings and/or tables. No as-built construction records were located.
37-206/2	101	12BP53 H-Pile	54	*			16.5			1 to 3	
Hwy 401 over pass at Leslie St.	102	Concrete Caisson	50	*	30	36	15.2	762	914		
WBL Collectors	103	Concrete Caisson	50	*	30	36	15.2	762	914		
(33 Spans)	104	Concrete Caisson	51	*	30	36	15.5	762	914		
	105	Concrete Caisson	52.5	*	30	36	16.0	762	914		
	106	Concrete Caisson	63	*	30	36	19.2	762	914		
	107	Concrete Caisson	69	*	30	36	21.0	762	914		
	108	Concrete Caisson	74	*	30	36	22.6	762	914		
	109	Concrete Caisson	72	*	30	36	21.9	762	914		
	110	Concrete Caisson	67	*	30	36	20.4	762	914		
	111	Concrete Caisson	87	*	30	36	26.5	762	914		
	112	Concrete Caisson	80.5	*	30	36	24.5	762	914		
	113	Concrete Caisson	70.5	*	30	36	21.5	762	914		embankment failure north of the bent
	114	Concrete Caisson	70.5	*	30	36	21.5	762	914		embankment failure north of the bent
	115	Concrete Caisson	72	*	30	36	21.9	762	914		embankment failure north of the bent
	116	Concrete Caisson	72	*	30	36	21.9	762	914		embankment failure north of the bent
	117	Concrete Caisson	74	*	30	36	22.6	762	914		embankment failure north of the bent
	118	Concrete Caisson	80	*	30	36	24.4	762	914		embankment failure north of the bent/working load 200 Ton/caisson
	119	Not Clear									embankment failure north of the bent
	120	12BP53 H-Pile	n/a	n/a							design load 60Ton/pile
	121	12BP53 H-Pile	n/a	n/a							design load 60Ton/pile
	123	Concrete Caisson	77	*	30	38	23.5	762	914		settlement problem, caisson replaced with 6 new tube piles of 220 mm diameter, also replaced the Pier
	124	Concrete Caisson	76	*	30	36	23.2	762	914		
	125	Concrete Caisson	77	*	30	36	23.5	762	914		
	126	Concrete Caisson	78	*			23.8				settlement problem, caisson replaced with 6 new tube piles of 220 mm diameter, also replaced the Pier
	127	Concrete Caisson	77	*	30	36	23.5	762	914		
	128	Concrete Caisson	78	*	30	36	23.8	762	914		
	129	Concrete Caisson	78	*	30	36	23.8	762	914		
	130	Concrete Caisson	78	*	30	36	23.8	762	914		
	131	12BP53 H-Pile	64	*			19.5			1 to 6	
	132	12BP53 H-Pile	64	*			19.5			1 to 6	
	133	12BP53 H-Pile	64	*			19.5			1 to 6	
	134	12BP53 H-Pile	78	*			23.8			1 to 3	

Table H-1 Foundation Elements - Highway 401 and Leslie Street Interchange

Project Number: TRANETOB01245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-builts	Depth	Diameter	Base Diameter	Battered	Notes Legend and Notes Signifies event took place Not Clear indicates data either not available or could not be read drawing.
			(ft)		(in)	(in)	(m)	(mm)	(mm)		Note data based on interpretation of drawings and/or tables. No as-built construction records were located.
37-206/3	318	12BP53 H-Pile									
Hwy 401 over pass at Leslie St.	319	Not Clear									
EBL Express	320	Not Clear									
(16 Spans)	321	Not Clear									Replaced the Pier
	322	Not Clear									
	323	Not Clear									
	324	12BP53 H-Pile	78	*	30	42	23.8	762	1067	not clear	Previously recognized as caisson, H pile design load 60 Ton
	325	Concrete Caisson#1	85		30	42	25.9	762	1067		
	325	Concrete Caisson#2	98		30	42	29.9	762	1067		
	325	Concrete Caisson#3	98		30	42	29.9	762	1067		
	326	Concrete Caisson#1	82		30	42	25.0	762	1067		
	326	Concrete Caisson#2	81		30	42	24.7	762	1067		
	326	Concrete Caisson#3	81		30	42	24.7	762	1067		
	327	Concrete Caisson#1	82		30	42	25.0	762	1067		
	327	Concrete Caisson#2	92		30	42	28.0	762	1067		
	327	Concrete Caisson#3	92		30	42	28.0	762	1067		
	328	Concrete Caisson#1	79		30	42	24.1	762	1067		
	328	Concrete Caisson#2	91		30	42	27.7	762	1067		
	328	Concrete Caisson#3	91		30	42	27.7	762	1067		
	329	Concrete Caisson#1	77		30	42	23.5	762	1067		
	329	Concrete Caisson#2	89		30	42	27.1	762	1067		
	329	Concrete Caisson#3	89		30	42	27.1	762	1067		
	330	Concrete Caisson#1	80		30	42	24.4	762	1067		
	330	Concrete Caisson#2	91		30	42	27.7	762	1067		
	330	Concrete Caisson#3	91		30	42	27.7	762	1067		
	331	12BP53 H-Pile	64	*			19.5			1 to 6	
	332	12BP53 H-Pile	64	*			19.5			1 to 6	
	333	12BP53 H-Pile	64	*			19.5			1 to 6	
	334	12BP53 H-Pile	78	*			23.8			1 to 3	

Table H-1 — Foundation Elements - Highway 401 and Leslie Street Interchange

Project Number: TRANETOB01245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-builts	Depth	Diameter	Base Diameter	Battered	Notes 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.
			(ft)		(in)	(in)	(m)	(mm)	(mm)		
37-206/4	218	Not clear	installed to suficial sand								Embankment failure north of the bent, pile tip at surficial sand deposit
Hwy 401 over pass at Leslie St.	219	12BP53 H-Pile	installed to v.dense till								
WBL Express	220	12BP53 H-Pile	installed to v.dense till								
(16 Spans)	221	12BP53 H-Pile	installed to v.dense till								Replaced the Pier
	222	12BP53 H-Pile	installed to v.dense till								
	223	12BP53 H-Pile	installed to v.dense till								
	224	12BP53 H-Pile	installed to suficial sand							not clear	Previously recognized as caisson, H pile design load 60 Ton
	225	Concrete Caisson#1	84		30	42	25.6	762	1067		
	225	Concrete Caisson#2	84		30	42	25.6	762	1067		
	225	Concrete Caisson#3	84		30	42	25.6	762	1067		
	226	Concrete Caisson#1	78		30	42	23.8	762	1067		
	226	Concrete Caisson#2	78		30	42	23.8	762	1067		
	226	Concrete Caisson#3	82		30	42	25.0	762	1067		
	227	Concrete Caisson#1	77		30	42	23.5	762	1067		
	227	Concrete Caisson#2	77		30	42	23.5	762	1067		
	227	Concrete Caisson#3	82		30	42	25.0	762	1067		
	228	Concrete Caisson#1	74		30	42	22.6	762	1067		
	228	Concrete Caisson#2	74		30	42	22.6	762	1067		
	228	Concrete Caisson#3	79		30	42	24.1	762	1067		
	229	Concrete Caisson#1	76		30	42	23.2	762	1067		
	229	Concrete Caisson#2	76		30	42	23.2	762	1067		
	229	Concrete Caisson#3	77		30	42	23.5	762	1067		
	230	Concrete Caisson#1	75		30	42	22.9	762	1067		
	230	Concrete Caisson#2	75		30	42	22.9	762	1067		
	230	Concrete Caisson#3	80		30	42	24.4	762	1067		
	231	12BP53 H-Pile	64	*			19.5			1 to 6	
	232	12BP53 H-Pile	64	*			19.5			1 to 6	
	233	12BP53 H-Pile	64	*			19.5			1 to 6	
	234	12BP53 H-Pile	78	*			23.8			1 to 3	

Table H-1 Foundation Elements - Highway 401 and Leslie Street Interchange

Project Number: TRANETOB01245AA

Structure No.	Bent No.	Foundation Type	Depth	* Estimated	Diameter	Base Diameter ** Estimated based on table no as-builts	Depth	Diameter	Base Diameter	Battered	Notes
											Legend and Notes Signifies event took place Not Clear indicates data either not available or could not be read drawing.
			(ft)		(in)	(in)	(m)	(mm)	(mm)		Note data based on interpretation of drawings and/or tables. No as-built construction records were located.
37-206/5	618	12BP53 H-Pile	90	*			27.4			1 to 3	one pile went to 45 m depth during the installation
Hwy 401 over pass at Leslie St.	619	Concrete Caisson	85	*	30	42	25.9	762	1067		
RAMP W-N/S	620	12BP53 H-Pile	60	*			18.3			1 to 6	
(6 Spans)	621	12BP53 H-Pile	56	*			17.1			1 to 6	
	622	Concrete Caisson	77	*	30	42	23.5	762	1067		
	623	Concrete Caisson	77	*	30	42	23.5	762	1067		
	624	12BP53 H-Pile	88	*			26.8			1 to 3	
37-206/6	728	12BP53 H-Pile	84	*			25.6			1 to 3	
Hwy 401 over pass at Leslie St.	729	Concrete Caisson#1	75		30	42	22.9	762	1067		
RAMP N-E	729	Concrete Caisson#2	75		30	42	22.9	762	1067		
(2 Spans)											
37-206/7	518	Concrete Caisson	74	*	30	36	22.6	762	914		working load 200 Ton/caisson
Hwy 401 over pass at Leslie St.	519	Concrete Caisson	72.5	*	30	36	22.1	762	914		working load 200 Ton/caisson
RAMP N-W	520	12BP53 H-Pile								1 to 6	
(6 Spans)	521	12BP53 H-Pile								1 to 6	
	522	Concrete Caisson									working load 200 Ton/caisson
	523	Concrete Caisson	77	*	30	36	23.5	762	914	1 to 3	working load 200 Ton/caisson
	524	12BP53 H-Pile	88	*			26.8				

Appendix I

Shear Strength Estimation

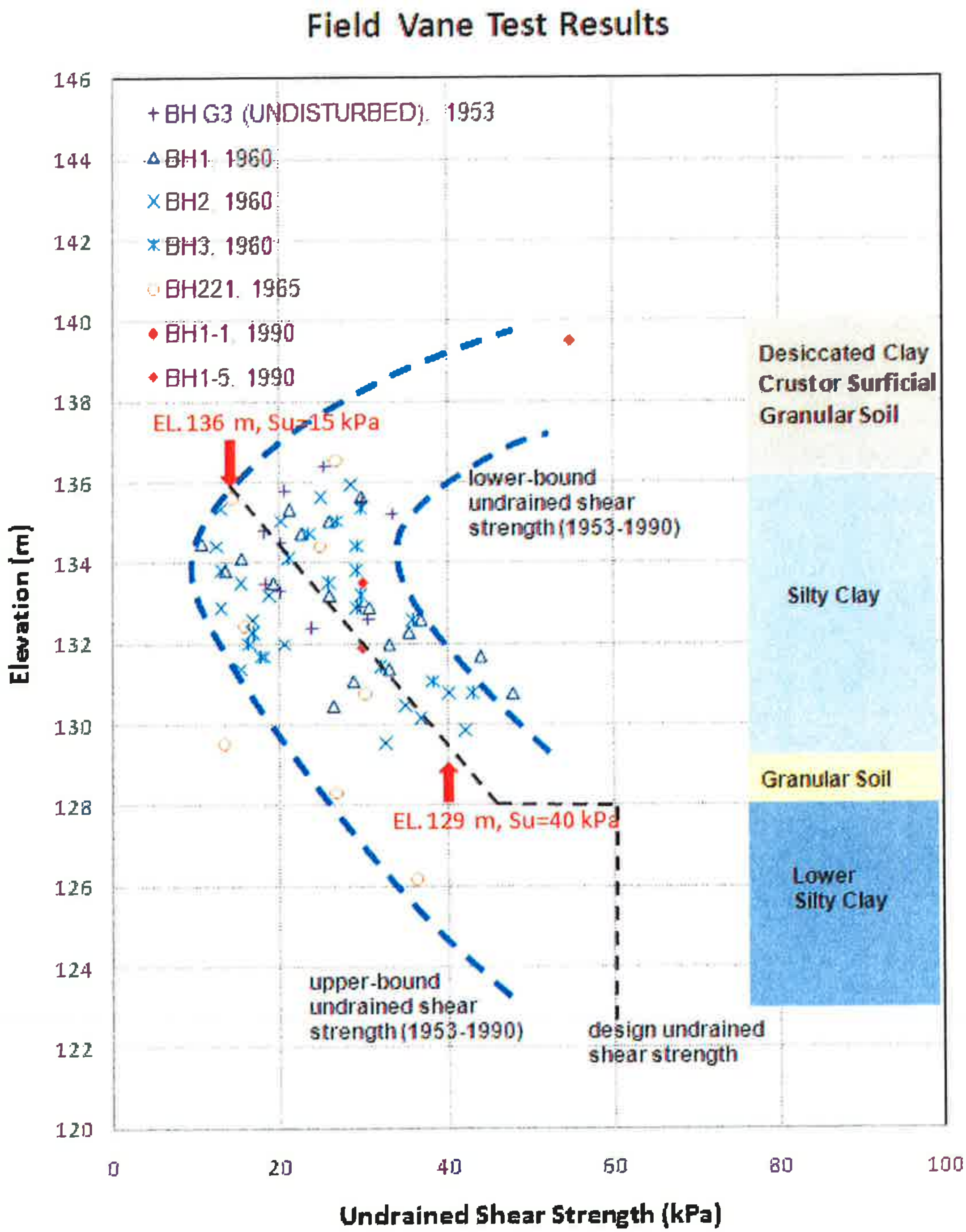


Figure I-1 Upper-bound, lower-bound undrained shear strengths and design undrained shear strength values vs elevation for past construction (1953-1990)

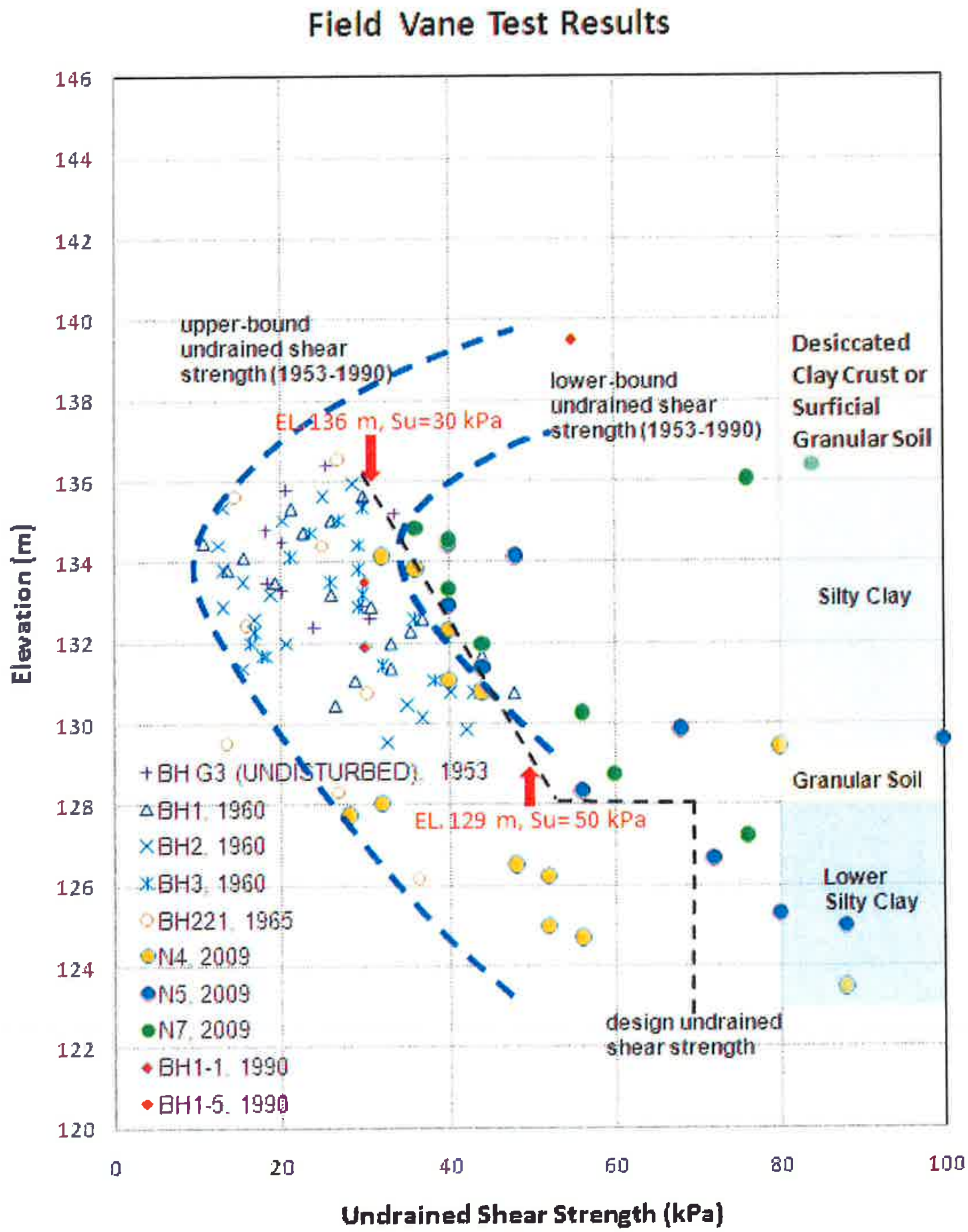


Figure I-2 Design undrained shear strength values vs elevation for a new construction.

Appendix J
Slope Stability Analyses

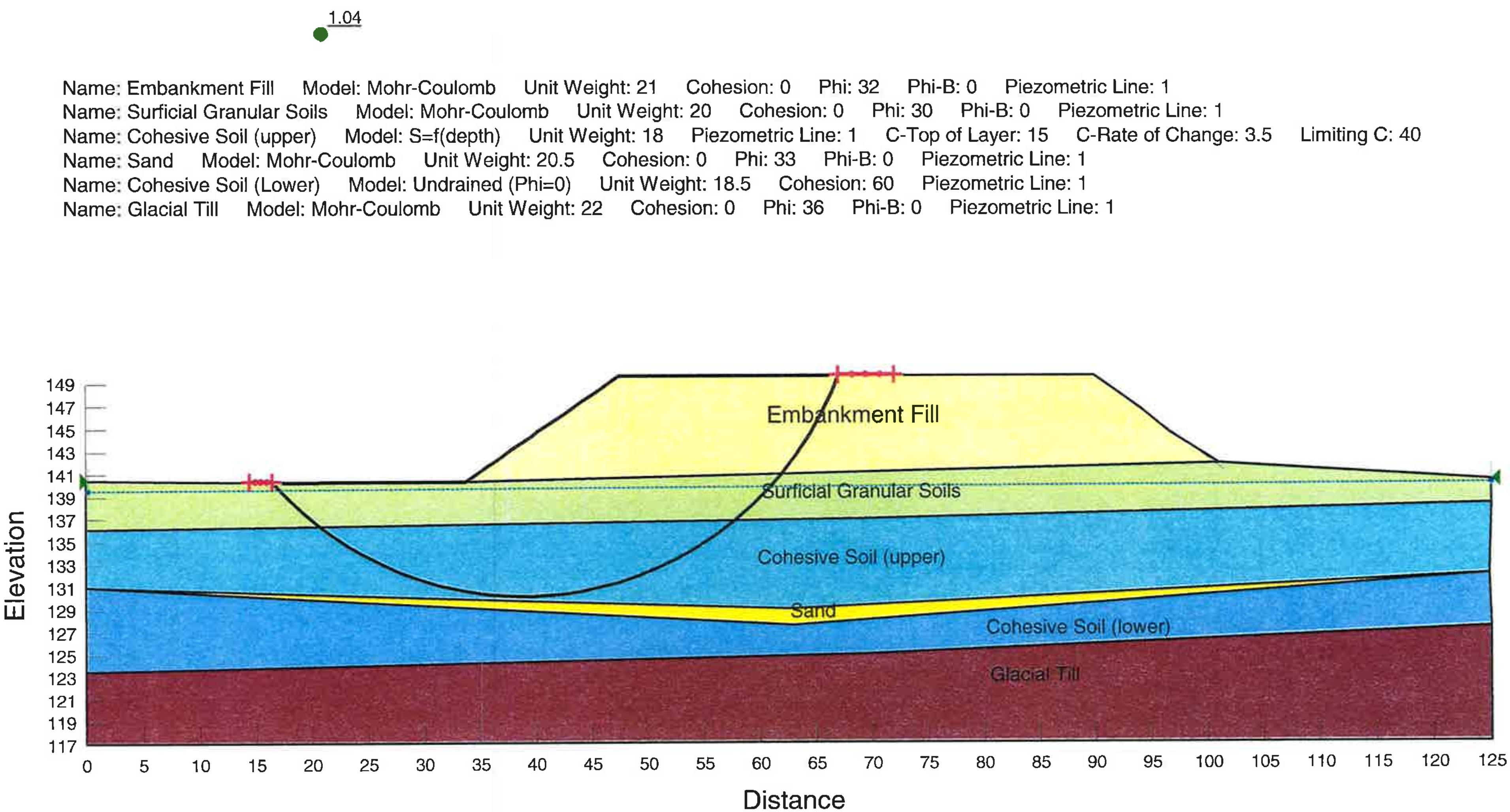


Figure J-1 Previous embankment failure (short term analysis)

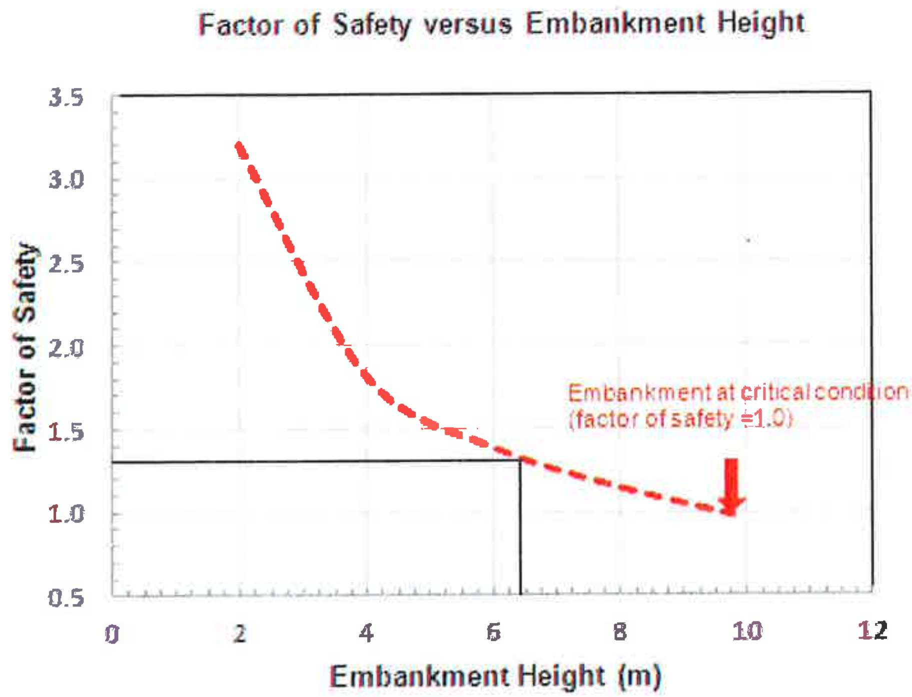


Figure J-2 Factor of safety vs embankment height

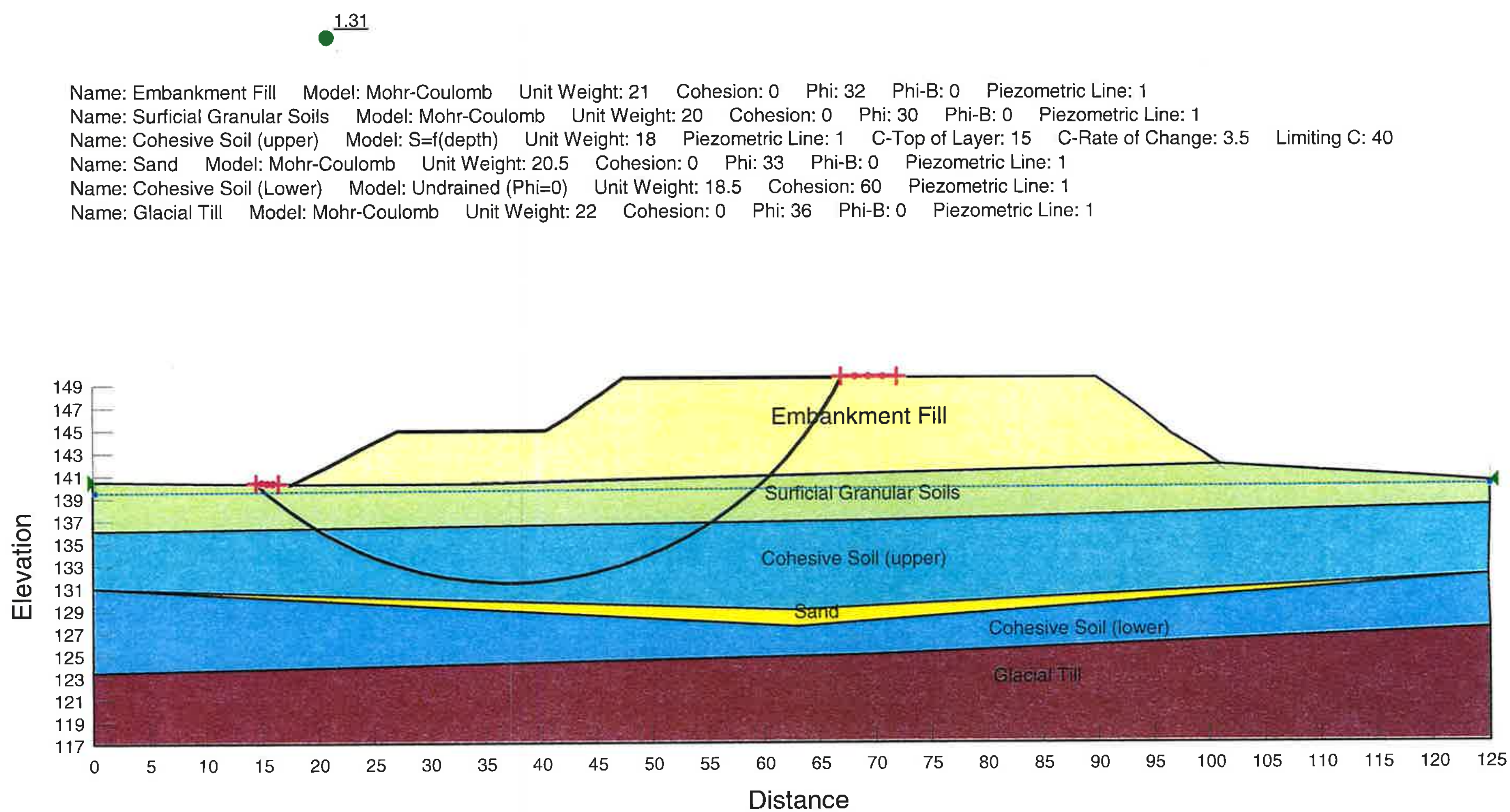


Figure J-3 Slope instability remedial with 4.5 m high berm (short term analysis)

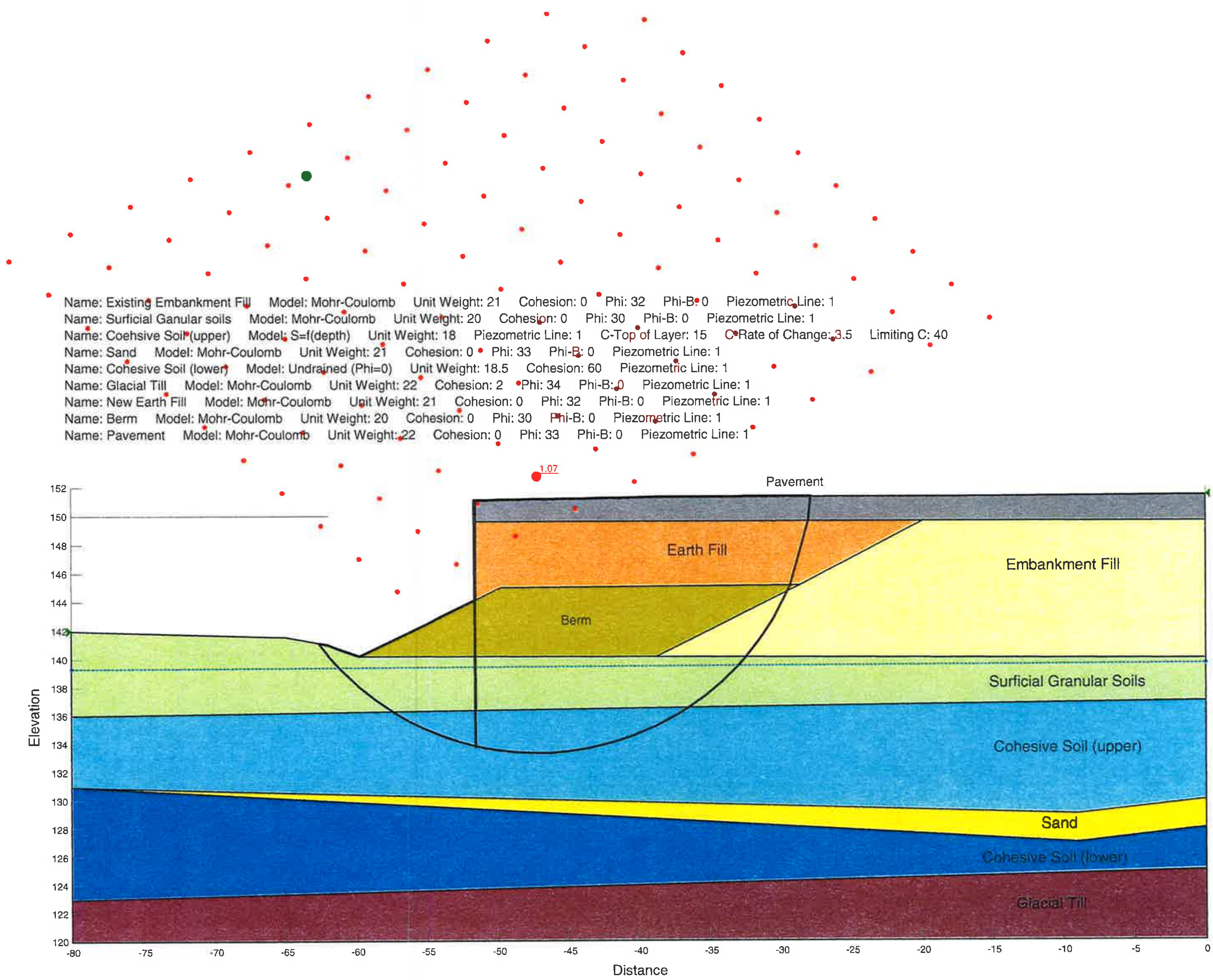


Figure J-4 New embankment with earth fill (short term)

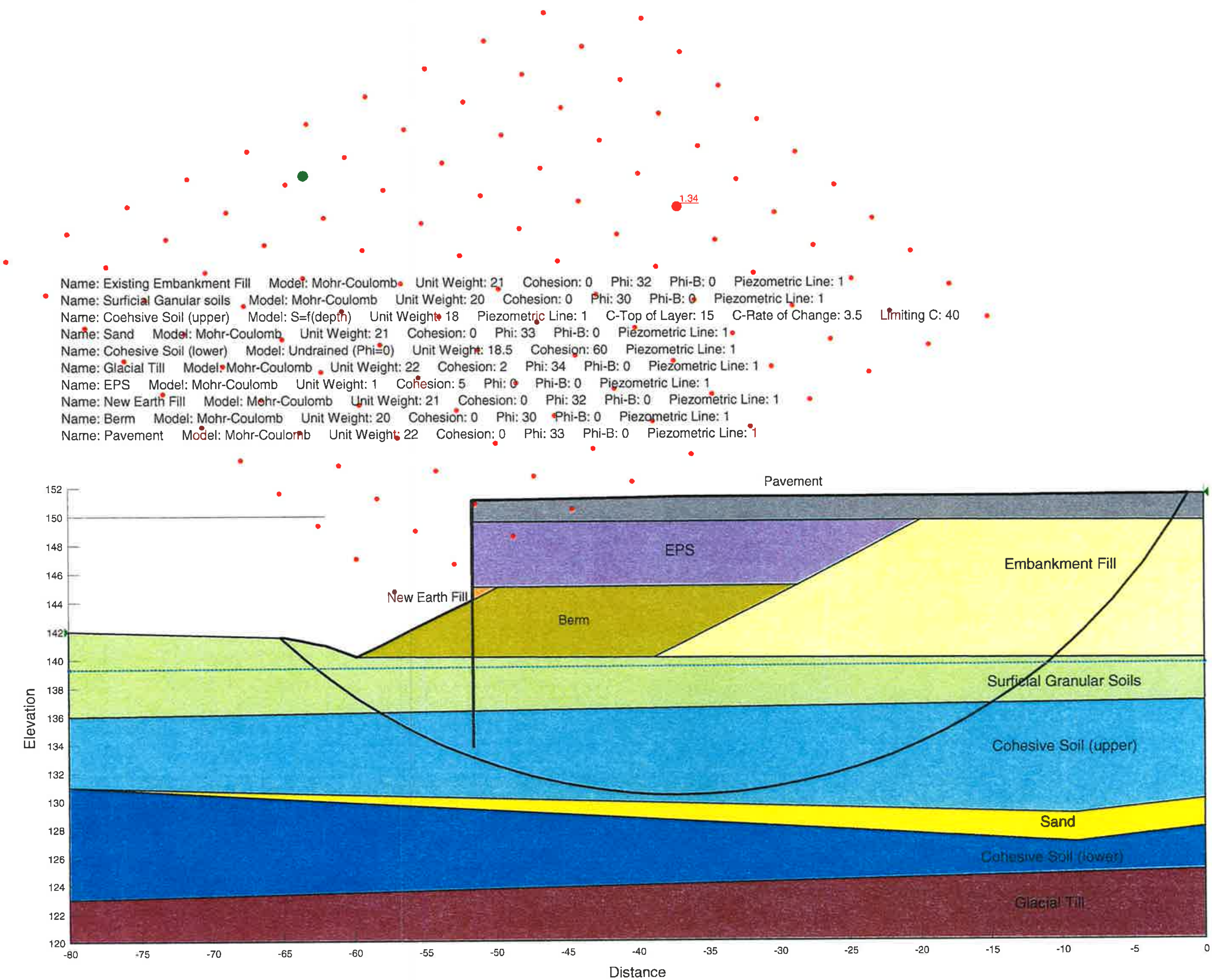


Figure J-5 New embankment with EPS (short term)

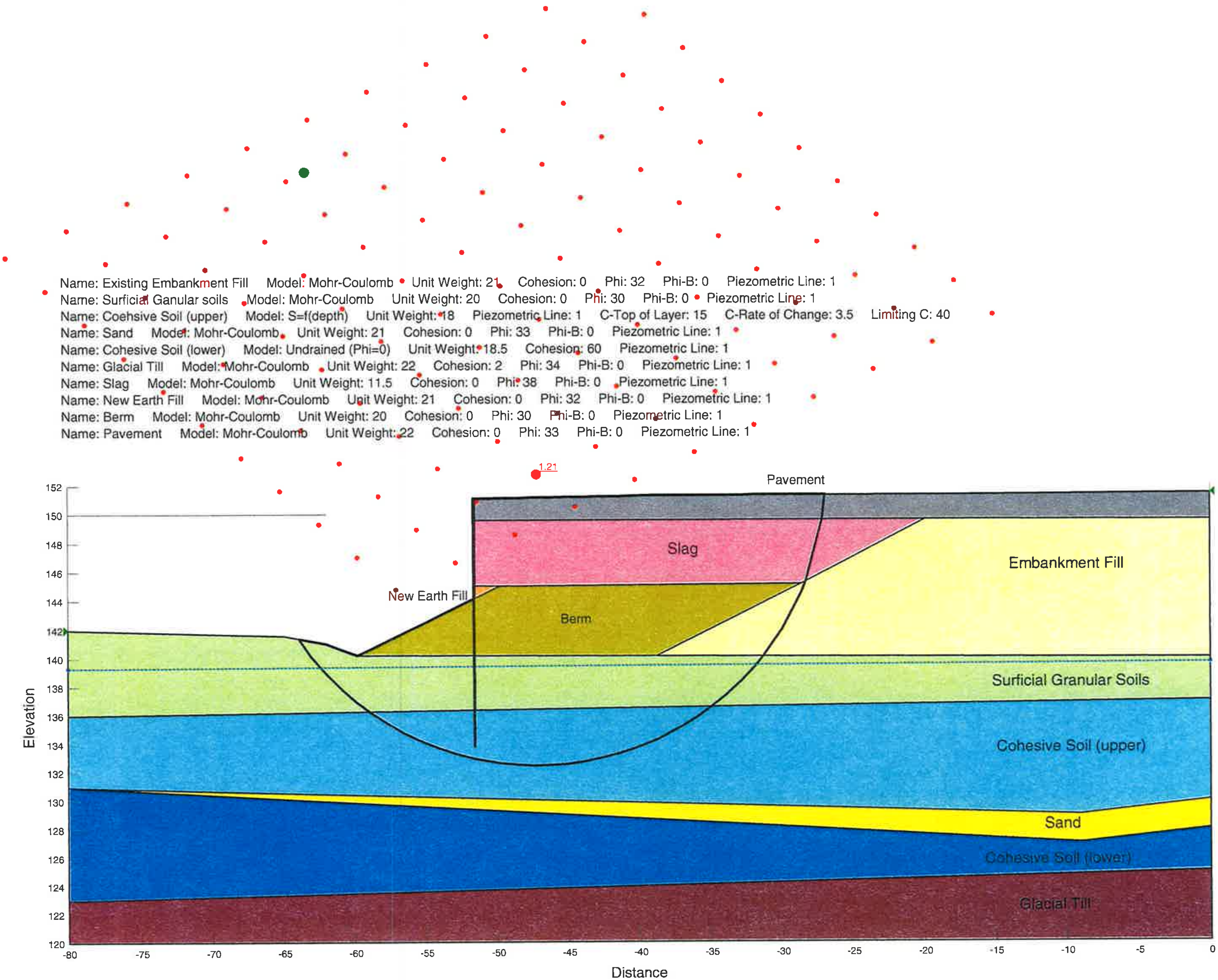


Figure J-6 New embankment with slag (short term)

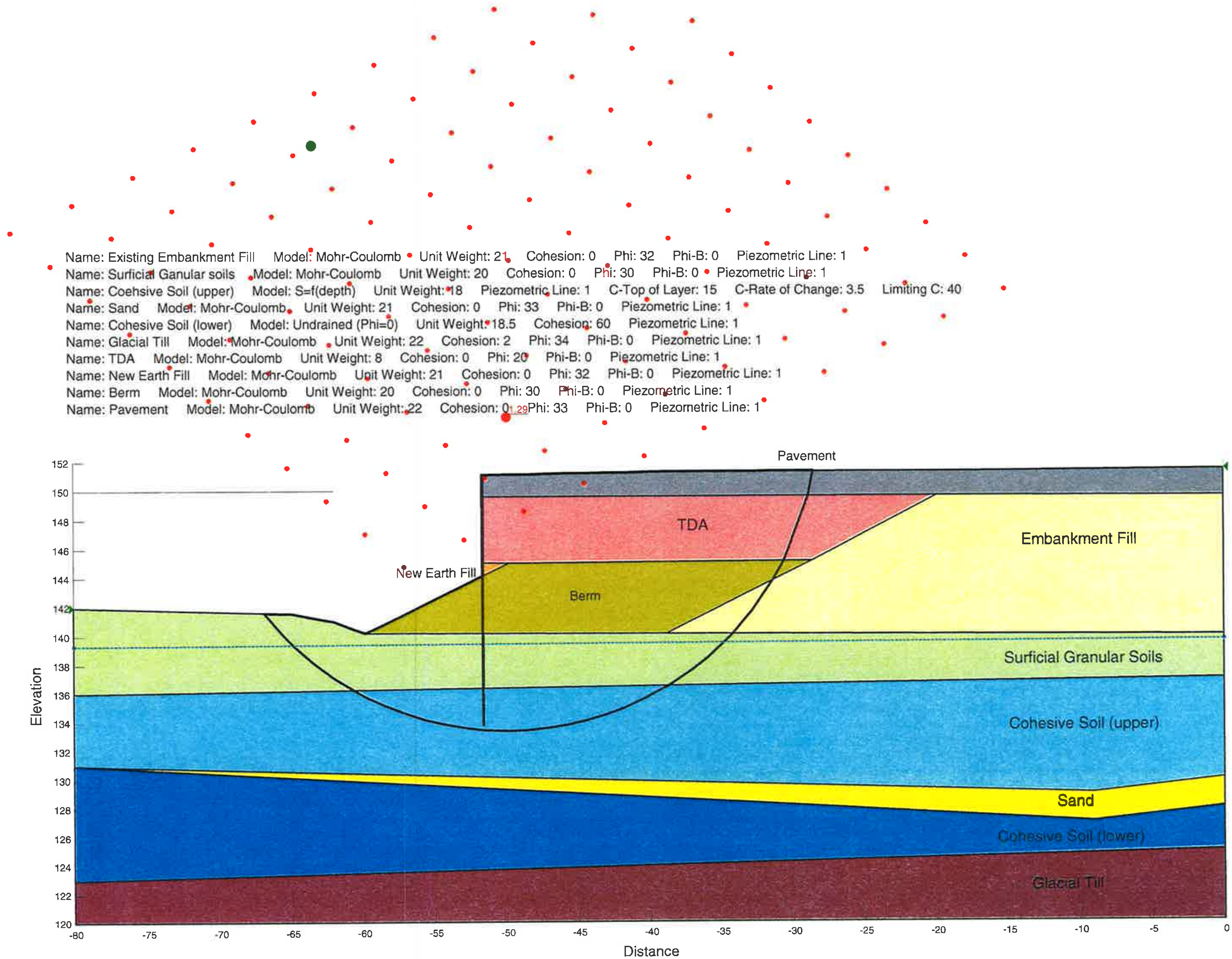


Figure J-7 New embankment with TDA (short term)

Appendix K

Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Table K-1. Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Embankment foundation/construction method	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Structural support for earth fill	-Normal earth fill can be used -Existing foundations can be utilized -New foundation installation may be difficult -High cost	-Residual capacity and integrity of the existing foundations should be verified -additional foundations are required and differential settlements between the existing and new foundations are anticipated - Installation of new foundations within the congested existing foundation setting may be difficult, especially if the existing bridge can not be demolished to install the new foundations	High	Not recommended due to cost effectiveness
Surcharge and preloading with/without wick drain	-Reduce the compressibility of soil -Strengthen the soil at the site -Time restriction	-Differential settlements between existing highway and new section -possible drag down of the existing highway embankment and the new retaining wall	Moderate	Not recommended due to time restraints
Ground improvement techniques (deep soil mixing, stone column and electro-osmotic treatment)	-Reduce the compressibility of soil -Strengthen the soil at the site -Expensive due to special equipment / material and specialist contractor	-Quality control is difficult to achieve -Problems may arise during the installation due to cobbles, boulders and congested existing foundations -not an MTO approved approach -not feasible without demolishing existing bridge	Moderate to high	Not recommended due to combined effects of cost effectiveness, quality control and time restraints
Light weight fill (EPS,TDA Blast Furnace Slag, Expanded Clay)	-Minimize stability and settlement problems -Relatively shorter construction period	-Except for EPS option, settlement and stability are not satisfactory	High	The most favourable option considering time saving that would be realized during construction on this important highway. As well, this option makes design and construction of high retaining wall on the north side feasible.

Appendix L

OPSDs, OPSSs and NSSPs

OPSDs

OPSD 3000.100 Foundation Piles Steel H-pile Driving

OPSD3101.150 Walls, Abutment, Backfill Minimum Granular Requirement

OPSD3101.200 Walls, Abutment, Backfill Rock

OPSSs

OPSS 539 - Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures

NSSPs

NSSP – Vibration Monitoring

NSSP-Caisson Piles

NSSP- Expended Polystyrene Embankment

<u>VIBRATION MONITORING</u> - 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.

<u>CAISSON PILES</u> - 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.

<u>EXPANDED POLYSTYRENE EMBANKMENT</u> – Item No.

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

2.1 National Standards of Canada

CAN/CGSB - 51.20 M87

2.2 ASTM

ASTM D1621	Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM C203	Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Test Method for Measuring the Minimum Oxygen Content
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS - Ontario Provincial Standard Specification

OPSS 212	Borrow
OPSS 501	Compaction
OPSS 517	Dewatering
OPSS 1010	Aggregates – Granular A, B, M, and Selected Subgrade Material
OPSS 1605	Expanded Extruded Polystyrene Pavement Insulation
OPSS 1860	Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

g) The method of placement of side slope cover.

6.4 Quality Verification Engineer

(1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

(2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular "A" material with gradation and physical requirements as specified in OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
    1. Geometry
    2. Nominal Density
    3. Compressive Strength
    4. Flexural Strength
    5. Thermal Resistance
    6. Dimensional Stability
    7. Flammability
    8. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	 1200 x 600 x 300 with tolerances ± 1% 10 mm in 3 m ± 0.5% -3, +5	
Compressive Strength	kPa (min)	110	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	$m^2.°C/W$ (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be ± 1%. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%. The thickness shall be within –3 to +5 mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be 0.7 $m^2.^{\circ}C/W$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25mm} = \frac{R_{measured}}{\text{Thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less then 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

(1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.

(2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.

(3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.

(4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.

(5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.

(6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.

(7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.

(8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

(9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

(10) The top surface and side surfaces of the expanded polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

(11) The contractor shall install the concrete base pad as detailed elsewhere in the contract.

(12) The side slope of the rigid expanded polystyrene embankment shall be covered with Lightweight fill and waste material as detailed elsewhere in this contract.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 QUALITY ASSURANCE

General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 PAYMENT

Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

Appendix M

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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