



July 2015

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CNR Overhead Replacement

Site Number 19-371

**Highway 401 Interchange Improvements/
Structural Replacements**

GWP 3054-11-00, Assignment No. 1 (3011-E-0046)

Ministry of Transportation, Ontario - West Region

Submitted to:

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REPORT



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TABLE I – COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

LIST OF ABBREVIATIONS

LIST OF SYMBOLS

RECORD OF BOREHOLE SHEETS

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DRAWING 1 - Borehole Locations and Soil Strata

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APPENDIX A

Laboratory Test Data

APPENDIX B

Site Photograph

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Record of Boreholes - Geocres No. 40114-110



PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

**CNR OVERHEAD REPLACEMENT
SITE NUMBER 19-371
HIGHWAY 401 INTERCHANGE IMPROVEMENTS/STRUCTURAL IMPROVEMENTS
GWP 3054-11-00, ASSIGNMENT No. 1 (3011-E-0046)
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design and 30% detailed design work for GWP 3030-11-00, 3054-11-00, 3053-11-00, 3070-11-00, 3059-11-00, and 3055-11-00. The project involves the preliminary design and 30% detailed design for ten (10) bridges, including improvements at five (5) interchanges and two culverts, on Highway 401.

This report addresses the replacement of the CNR Overhead structure between Wellington Road and Highbury Avenue in London, Ontario (Site 19-371).

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P2-1132-0076-P01 dated September 10, 2012. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated November 2012.



2.0 SITE DESCRIPTION

2.1 General

The CNR Overhead structure is located in the southern portion of the City of London, Ontario as shown on the Key Plan, Figure 1.

For the purposes of this report, Highway 401 and the CNR lines are assumed to be oriented east-west and north-south, respectively. This section of Highway 401 is currently a six lane divided freeway oriented generally northeast-southwest. The existing structure was constructed in 1954 and consists of a single span, pre-cast concrete beam (U-shape) structure. The area adjacent to the site consists of relatively flat-lying industrial and commercial lands. It is understood that the existing structure will be demolished and replaced with a wider structure along the same alignment.

A site photograph is provided in Appendix B.

2.2 Site Geology

This project lies within the physiographic region known as the Westminster Moraine. The physiographic mapping indicates that the CNR Overhead site is situated on an undrumlinized till plane¹. Geologic mapping indicates that the surficial material consists of Port Stanley silty clay till and clayey silt till which is, in places, covered by thin layers of lacustrine silt.²

The bedrock in the area of the site is reported to consist of medium brown, microcrystalline limestone of the Dundee Formation of the Hamilton Group of Middle Devonian Age.³ The bedrock surface is at about elevation 210 metres, some 60 metres below ground surface.

¹ Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2.

² Dreimanis, A., 1963: Pleistocene Geology of the St. Thomas Area (East Half), Southern Ontario. Ontario Department of Mines, Preliminary Geological Map 238, scale 1:50,000.

³ Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between February 11 and 22, 2013, during which time two boreholes were drilled at the approximate locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations and borehole depths:

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
201	4 755 661	411 389	269.5	30.9
202	4 755 566	411 356	267.4	21.5

The investigation was carried out using track mounted drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.75 metre intervals of depth in the critical foundation zones and 1.5 metre intervals beyond this depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures (ASTM D1586). The split spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 38 millimetres; therefore, particles or objects that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. The results of the SPT testing as presented on the Record of Borehole sheets and in Section 4.0 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.) and therefore represent field values using an automatic hammer.

The boreholes were terminated 30.9 and 21.5 metres below the ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and piezometers installed in borehole 201. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder staff members who also located the boreholes in the field, monitored the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labelled containers and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations, were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

Borehole 201 was advanced on the north side of Highway 401 east of the CNR tracks and borehole 202 was advanced on the south side of Highway 401, west of the tracks. The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.

In addition, information from the original subsurface investigation for the existing structure was incorporated into this report. Records of Boreholes 6 through 10, 103, 110, 111 and 112 from Geocres No. 40114-110 titled "C.N.R. Overhead Widening, STR Site 19-94-371" was used to supplement the current data.



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CNR OVERHEAD, SITE NUMBER 19-371**

The Record of Borehole sheets for the previous boreholes are presented in Appendix C in their original format. The table below summarizes the locations, ground surface elevations and depths of the previous boreholes.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
6	4 755 597	411 380	268.3	12.7
7	4 755 635	411 379	268.0	18.7
8	4 755 583	411 383	267.8	37.2
9	4 755 577	411 368	268.3	18.6
10	4 755 649	411 367	269.0	21.8
103	4 755 385	411 387	276.2	9.6
110	4 755 638	411 389	276.4	10.2
111	4 755 625	411 359	276.3	11.1
112	4 755 595	411 362	276.0	8.8

The previous borehole locations were originally referenced to the North American Datum (NAD) 27 coordinate system and have been adjusted to reflect the Modified Transverse Mercator (MTM) coordinate system currently used by MTO.

The approximate locations of the previous boreholes are shown in plan on Drawing 1.



4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are provided on the attached Record of Borehole sheets following the text of this report and in Appendices A and C. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered layers of topsoil and/or fill overlying layers of clayey silt, silt, sandy silt, silty sand, silty clay and clayey silt till.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawing 1. Detailed descriptions of the subsurface conditions encountered in the current boreholes are provided on the Record of Borehole sheets and are summarized in the following sections.

Materials described as silty clay and gravelly sand in the boreholes advanced for Geocres Report No. 40I14-110 have been classified as clayey silt and sand and gravel, respectively, based on the results of laboratory testing and comparison of the stratigraphy in adjacent boreholes. These classifications have been reflected in the following report sections and the inferred profile, Drawing 1.

4.1.1 Topsoil and Fill

Topsoil was encountered at the ground surface in borehole 201 and was about 0.2 metres thick.

Boreholes 103 (40I14-110), 110 (40I14-110), 111 (40I14-110) and 112 (40I14-110) were drilled through the existing highway embankment and encountered between 7.5 and 11.1 metres of embankment fill materials.

Layers of fill materials were encountered beneath the surficial topsoil in borehole 201 and at the ground surface in boreholes 202, 6 (40I14-110), 7 (40I14-110), 103, 110, 111 and 112 (40I14-110). The fill materials consisted of silty sand, sand and gravel, silty clay and clayey silt and were about 1.0 to 7.5 metres thick, where fully penetrated. Boreholes 110, 111 and 112 were terminated in the fill after exploring it for about 8.8 to 11.1 metres. The granular fills had N values, as determined in the standard penetration testing, ranging from 3 to 68 blows per 0.3 metres with water contents of 8 to 15 per cent. The cohesive fills had N values ranging from 7 to 35 blows per 0.3 metres with water contents of 7 to 29 per cent. Atterberg limits testing carried out for Geocres Report No. 40I14-110 yielded liquid and plastic limits ranging from 14 to 34 per cent and 12 to 17 per cent, respectively, and plasticity indices of 8 to 19 per cent, indicating low plasticity. The results of the Atterberg limits testing carried out on samples of clayey silt from Geocres Report No. 40I14-110 are provided in Appendix C.

A layer of buried topsoil fill about 1.3 metres thick was encountered within the fill in borehole 201 at elevation 268.3 metres. The buried topsoil had an N value of 9 blows per 0.3 metres with water contents of about 28 and 31 per cent.



“Organic clay” layers, 0.5 to 2.1 metres thick, were encountered from the ground surface in boreholes 8 (40I14-110) and 9 (40I14-110) and beneath the fill materials in boreholes 7 (40I14-110) and 112 (40I14-110). Borehole 112 (40I14-110) was terminated after penetrating about 0.2 metres into the organic clay. Measured N values in the organic clay ranged from 6 to 17 blows per 0.3 metres. A sample of the organic clay had a water content of 31 per cent, liquid and plastic limits of 58 and 30 per cent, respectively, and a plasticity index of 28 per cent, indicating high plasticity.

Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

4.1.2 Clayey Silt

Layers of firm to hard clayey silt were encountered at the ground surface in borehole 10 (40I14-110), beneath the fill in boreholes 201, 202, 6 (40I14-110) and 103 (40I14-110), beneath the organic clay in boreholes 7 to 9 (40I14-110) and beneath layers of silt in boreholes 7 (40I14-110) and 8 (40I14-110) between elevations 244.8 and 269.0 metres. The clayey silt layers were between 1.5 and 6.7 metres thick, where fully penetrated. Borehole 8 (40I14-110) was terminated in clayey silt after penetrating the layer for 14.2 metres.

The N values in the clayey silt ranged from 7 to greater than 100 blows per 0.3 metres. Samples of the clayey silt had water contents ranging from 12 to 22 per cent. The low plasticity clayey silt from the current investigation had corresponding average plastic and liquid limits of 16 and 28 per cent, respectively, based on four Atterberg limits determinations, the results of which are shown on Figure A-1. Atterberg limits testing carried out for Geocres Report No. 40I14-110, as shown in Appendix C, yielded liquid and plastic limits ranging from 23 to 34 per cent and 11 to 18 per cent, respectively, and plasticity indices of 6 to 15 per cent, indicating low plasticity.

Grain size distribution curves for samples of the clayey silt from the current investigation are shown on Figure A-2.

4.1.3 Silt

Layers of compact to very dense silt were encountered beneath the clayey silt in boreholes 201, 202 and 6 to 10 (40I14-110) as well as beneath a layer of silty sand in borehole 201 between about elevations 250.3 and 263.7 metres and were 3.0 to 9.1 metres thick where fully penetrated. Boreholes 6 (40I14-110), 7 (40I14-110), 9 (40I14-110) and 10 (40I14-110) were terminated in the silt after penetrating the layers for 1.6 to 16.2 metres.

The silts had N values of 20 to greater than 100 blows per 0.3 metres with water contents of about 14 to 24 per cent. The silt in boreholes 201 and 202, below about elevations 263.0 and 256.1 metres, respectively, was noted to contain clayey silt layers. An Atterberg limits determination carried out on the silt from this level in borehole 202 indicated plastic and liquid limits of 14 and 22 per cent, respectively. These data are included on Figure A-1. Atterberg limits testing carried out for Geocres Report No. 40I14-110, as shown in Appendix C, yielded liquid and plastic limits ranging from 20 to 30 per cent and 14 to 19 per cent, respectively, and plasticity indices of 2 to 15 per cent, with the higher values indicative of clayey silt layers noted in the silt.



Grain size distribution curves for samples of the silt from the current investigation are provided on Figure A-3.

4.1.4 Silty Clay

A layer of very stiff silty clay about 0.7 metres thick was encountered at about elevation 251.7 metres in borehole 201. The silty clay had an N value of 27 blows per 0.3 metres for a test partially completed in the layer with a water content of about 18 per cent.

4.1.5 Silty Sand

Beneath the silty clay in borehole 201, a layer of compact silty sand about 0.7 metres thick was encountered at about elevation 251.0 metres. The silty sand had an N value of 27 blows per 0.3 metres for a test partially completed in the layer with a water content of about 16 per cent.

4.1.6 Clayey Silt Till

Layers of very stiff to hard clayey silt till were encountered beneath the silts in boreholes 201 and 202 at elevations 245.7 and 254.6 metres, respectively. Borehole 201 was terminated in the clayey silt till after exploring it for about 7.2 metres. In borehole 202, the clayey silt till was about 4.1 metres thick.

The clayey silt till had N values of 17 to 47 blows per 0.3 metres with water contents of 12 to 19 per cent. The clayey silt till had corresponding average plastic and liquid limits of 15 and 25 per cent, respectively, based on two Atterberg limits determinations, indicating low plasticity. These data are provided on Figure A-1.

Grain size distribution curves for samples of the clayey silt till are provided on Figure A-4.

Although not explicitly encountered during drilling, cobbles and boulders should be expected in the clayey silt till.

4.1.7 Sandy Silt

Very dense sandy silt was encountered beneath the clayey silt till in borehole 202 at about elevation 250.5 metres. Borehole 202 was terminated in the sandy silt after exploring it for about 4.6 metres. The sandy silt had N values from 57 to greater than 100 blows per 0.3 metres with water contents of 12 to 14 per cent.

Grain size distribution curves for samples of the sandy silt are shown on Figure A-5.

4.1.8 Sand and Gravel

Very dense sand and gravel was encountered beneath the clayey silt in borehole 103 (40114-110) at elevation 267.2 metres. Borehole 103 (40114-110) was terminated in the sand and gravel after penetrating the layer for about 0.6 metres. The sand and gravel had an N value of 56 blows per 0.3 metres. Cobbles and boulders should be expected in the sand and gravel.



4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling and piezometers were installed in borehole 201 as shown on the Record of Borehole sheets.

The encountered and subsequently measured groundwater levels in the boreholes are summarized below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Artesian Conditions
201	269.5	267.1	-
202	267.4	263.1	Encountered at elev. 250.6 m
6 (40I14-110)	268.3	267.9	
7 (40I14-110)	268.0	267.7	
8 (40I14-110)	267.8	273.0	Encountered at elev. 230.6 m
9 (40I14-110)	268.3	267.6	
10 (40I14-110)	269.0	267.9	
103 (40I14-110)	276.2	268.1	
110 (40I14-110)	276.4	269.0	
111 (40I14-110)	276.3	267.9	
112 (40I14-110)	276.0	*	

* Groundwater level not established.

Borehole	Ground Surface Elevation (m)	Installation	Measured Groundwater Elevation (m)	
			March 8, 2013	April 3, 2013
201	269.5	Upper Piezometer	Piezometer damaged	
		Lower Piezometer	266.3	266.2

Artesian conditions were encountered in borehole 202 and borehole 8 (40I14-110) at about elevations 250.6 and 230.6 metres, respectively. Borehole 8 (40I14-110) indicated a head of 5.2 metres above ground surface.

The above-noted water levels are not considered to be representative of the long-term, stabilized groundwater conditions. Based on the encountered and subsequently measured groundwater levels and the soil colour change from brown to grey, the inferred groundwater level within the clayey silt is at about elevation 266 metres. Groundwater levels should be expected to fluctuate seasonally and in response to periods of sustained precipitation or spring melt conditions.



5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Lantech Drilling Services Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Brett Thorner under the direction of Mr. David J. Mitchell.

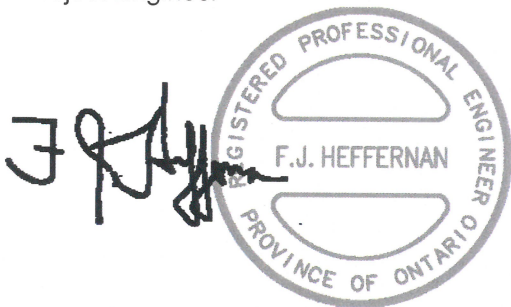
The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. and the Team Leader, Dr. Storer J. Boone, P. Eng. The report was reviewed by Mr. Michael E. Beadle, P.Eng., a senior geotechnical engineer and Associate. Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment conducted an independent quality review of the report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT

**CNR OVERHEAD REPLACEMENT
SITE NUMBER 19-371**

**HIGHWAY 401 INTERCHANGE IMPROVEMENTS/STRUCTURAL IMPROVEMENTS
GWP 3054-11-00, ASSIGNMENT No. 1 (3011-E-0046)
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the replacement of CNR Overhead structure (Site 19-371). The recommendations are based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

Based on the preliminary design information provided by Dillon, the replacement structure will have integral abutments supported on driven steel H-piles and three spans of 13.5, 21.0 and 13.5 metres with piers supported on drilled shafts (caissons). The replacement structure is to be constructed along the alignment of the existing structure. It is proposed to construct the pier caissons using a top-down approach where the caissons will be drilled from approximately the proposed underside of pier cap elevation and the pier caps constructed prior to excavation to the approximate rail elevation. The soils between the existing and new abutments will be excavated and the existing structure demolished after completion of the new structure. The concrete piles associated with the existing structure are to remain in place.

6.2 Existing Structure

For the purposes of this report, Highway 401 and the CNR tracks are assumed to be oriented east-west and north-south directions, respectively. This section of Highway 401 is currently a six lane divided freeway oriented generally northeast-southwest. The existing structure was constructed in 1954 and consists of a single span, pre-cast concrete beam (U-shape) structure. The span is approximately 10.5 metres. It is understood that the foundations for the original structure are approximately 0.41 and 0.48 metre diameter expanded base Franki piles founded at about elevation 260.0 metres with pile caps at elevation 265.9 metres. In 1989, the structure was widened about 4.3 metres to the north and about 3.7 metres to the south. The existing abutments were modified to accommodate the deck widening. New retaining walls were constructed on shallow foundations at all four corners of the abutments.

6.3 Bridge Foundations

The draft general arrangement for the CNR Overhead shows a 3-span slab-on-girder bridge. We understand that the outer portions of the westbound and eastbound bridge will be constructed first giving overhead clearance for caisson and pile driving rigs. The inner portions of the bridge will be subsequently constructed. A crash wall will be attached to the caissons.

The subsurface conditions at the site typically consist of layers of topsoil and fill materials overlying layers of silty clay, silt, clayey silt, silt, sand, sandy silt and clayey silt till.



The presence of the existing embankment fill precludes the use of shallow foundations for the abutments. Consideration is being given to designing integral abutments, and steel H-piles are the preferred technical alternative for support of the abutments. If conventional abutments are to be designed, the abutments can be supported on closed-end concrete filled steel tube piles or steel H-piles as previously planned. The piers are to be founded on concrete filled steel tube piles or drilled shafts. Recommendations for each of these foundation systems are provided below. A comparison of various foundation alternatives is provided in Table I following the text of this report.

6.3.1 Geotechnical Axial Resistance

Driven Piles

Steel HP 310 x 110 piles or 324 millimetre outside diameter (OD) concrete filled steel tube piles with 9.5 millimetre wall thicknesses driven to the elevations indicated in the table below may be designed using factored geotechnical resistances at Ultimate Limits States (ULS) of 900 and 1,000 kilonewtons (kN), respectively, and geotechnical resistances at Serviceability Limits States (SLS) of 700 and 800 kN, respectively. The SLS values correspond to a maximum of 25 millimetres of total settlement for new abutment construction.

Pile Type	Anticipated Tip Elevation (m)
324 mm dia. Steel Tube Piles	251
HP 310 x 110 H-Piles	248

Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose, uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement. A Non-Standard Special Provision (NSSP) for CSP Integral Abutments detailing the sand gradation should be included in the Contract Documents.

Due to the presence of artesian conditions at the site, the possibility of groundwater migration around the pile annulus exists. To minimize the creation of a potential void around the pile as it is driven, driving shoes, reinforcement to flanges, splice plates and the like should add as little as possible to the pile cross-sectional dimensions. Further, it is recommended that the piles be driven from a 0.5 metre thick pad of Granular A which would act like a filter should sustained groundwater flow occur. A NSSP for a granular pad should be included in the Contract Documents.

Piles should be installed and monitored in accordance with Ontario Provincial Standard Specification (OPSS) 903. The maximum ultimate resistance of two times the factored ULS value indicated above should also be noted on the foundation drawing.

The actual pile penetration and pile set characteristics will be dependent, to some extent, upon the driving equipment selected by the contractor. It is recommended that, following the selection of the driving equipment,



the piling contractor submit for review to the geotechnical engineer his proposed pile driving criteria based on the characteristics of the hammer and equipment intended for use. The pile driving operation should be carefully monitored by this office to confirm that the design pile capacities are being achieved.

Drilled Shafts

Due to the presence of artesian groundwater conditions at the site, caissons should not be advanced deeper than elevation 255 metres in order to maintain the stability of the founding surface.

For preliminary design, the vertical load carrying capacity of the caissons derived from skin friction may be calculated using the following equation.

$$Q_s = \pi B \Delta z f_{SN}$$

Where Q_s is the nominal skin friction in kN, B is the shaft diameter in metres, Δz is the thickness of the soil layer over which resistance is calculated in metres and f_{SN} is the nominal unit skin friction in kilopascals (kPa). The upper 1.2 metres below the ground surface should be neglected to account for frost action. Any portion of the caisson within fill materials should also be neglected.

Assuming that caissons greater than 1 metre in diameter will be used, the component of the vertical load carrying capacity that may be derived from end bearing in the native soils may be calculated using the following equation:

$$Q_b = q_{BN} A_t$$

Where Q_b is the toe resistance in kN, q_{BN} is the nominal unit base resistance in kPa and A_t is the cross-sectional area of the caisson in square metres. Caissons founded in the native soils may be designed using the nominal unit side and base resistances provided in the following table. The stratigraphy presented in the table below has been simplified for the purposes of this report.

Soil Type	Elevation (m)	f_{SN} (kPa)	q_{BN} (kPa)	Unit Weight (kN/m ³)
Fill	Where applicable	-	-	19.0
West Pier				
- Very Stiff Clayey Silt	263 to 266	5	1,500	20.0
- Compact to Very Dense Silt	255 to 263	85	5,000	20.0
- Very Stiff to Hard Clayey Silt Till	at 255	-	3,000	21.0
East Pier				
- Very Stiff to Hard Clayey Silt	260 to 263	5	1,500	20.0
- Compact to Very Dense Silt	255 to 260	85	5,000	20.0



The ultimate resistance Q_u is the sum of Q_b and Q_s . A resistance factor of 0.5 should be applied to Q_u to obtain the factored axial resistance at ULS.

As indicated above, caissons should not be advanced lower than elevation 255 metres. The ground surface elevation at the pier locations is understood to be about 269 metres. For the purposes of preliminary design, the following factored geotechnical resistances at ULS may be used for preliminary design of caissons with a nominal diameter of 1.2 metres founded at the elevations recommended in the table below.

Location and Founding Strata	Design Tip Elevation (m)	Embedded Caisson Length (m)	Factored Geotechnical Resistance at ULS (kN)
West Pier			
- Compact to Very Dense Silt	260	9	3,300
- Very Stiff to Hard Clayey Silt Till	255	14	3,000
East Pier			
- Compact to Very Dense Silt	260	9	2,800
- Compact to Very Dense Silt	255	14	3,600

6.3.2 Downdrag Loads

Since no grade changes are anticipated, downdrag loads are expected to be small and can be neglected in the foundation design.

6.3.3 Frost Protection

Pile caps and the like should be provided with at least 1.2 metres of soil cover or thermal equivalent for frost protection.

6.3.4 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading.

The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

$$\begin{aligned} k_h &= \text{coefficient of horizontal subgrade reaction (MPa/m)} \\ &= n_h (z/d) \quad \text{for cohesionless soils} \\ &= \frac{67 S_u}{d} \quad \text{for cohesive soils} \end{aligned}$$



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
CNR OVERHEAD, SITE NUMBER 19-371

- d = pile width or diameter (m)
 n_h = constant of horizontal subgrade reaction (MPa/m)
 S_u = undrained shear strength of the soil (MPa)
z = depth below ground surface (m)

The range in values reflects the variability in subsurface conditions as well as the two extremes of design: the requirement for flexibility if integral abutments are selected, and the requirement for lateral support in the cases of conventional pile supported abutments.

Location	Soil Type	Elevation (m)	n_h (MPa/m)	S_u (MPa)
CSPs for integral abutments	Granular backfill	Where applicable	1 - 3	-
West Abutment	Existing fill - Dense to very dense sand and gravel to silty sand	Above 266	7 - 14	-
	Very stiff clayey silt	263 to 266	-	175
	Compact to very dense silt	255 to 263	6 - 10	-
	Very stiff clayey silt till	250 to 255	-	200
	Compact to very dense sandy silt to silt	246 to 250	9 - 12	-
East Abutment	Existing fill - compact silty sand	Above 263	7 - 14	-
	Very stiff clayey silt	260 to 263	-	175
	Compact to very dense silt	245 to 260	11 - 20	-
	Very stiff to hard clayey silt to clayey silt till	238 to 245	-	200

The lateral resistances for single piles along both the strong and weak axes, where applicable, were assessed and are summarized in the following table.

Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
Integral Abutments		
- 305 mm OD x 9.5 mm tube	40	10
- HP 310 x 110 weak axis bending	30	10
Semi-integral or conventional abutments		
- 305 mm OD x 9.5 mm tube	225	50
- HP 310 x 110 strong axis bending	250	60



Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
Piers - 1.2 metre diameter caisson	315	130

The lateral resistances were calculated using Brom's hand calculation method as described in Federal Highways Administration (FHWA) Publication No. FHWA HI 97-013.⁴ For integral abutments, a free-head pile was assumed with the horizontal load applied at the ground surface (i.e. underside of abutment). For semi-integral and conventional abutments, a fixed-head pile was assumed with the horizontal load applied at the ground surface (underside of abutment). For pier caissons, free-head piles were assumed with the horizontal load applied at the proposed pile cap elevation. A 28-day compressive strength of 30 megapascals was assumed for the concrete filled steel tube piles and caissons. The SLS values are based on 10 millimetres of deflection at the ground surface.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R , as follows:

<i>Pile Spacing in Direction of Loading, d = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located in London, Ontario. According to Table A.3.1.1 of the CHBDC, the zonal acceleration ratio, A , applicable to this site is 0.00. The corresponding acceleration-related seismic zone, Z_a , is 0.

The replacement bridge is in Seismic Performance Zone (SPZ) 1, based on a CHBDC classification as "Emergency Route Bridge". Based on the site stratigraphy, the soil profile type is categorized as Type II with a seismic site response coefficient, S , of 1.2 based on the CHBDC criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

⁴ Federal Highway Administration, 1998: *Design and Construction of Driven Pile Foundations, Workshop Manual – Volume 1*. Publication No. FHWA HI 97-013.



6.4.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.⁵ Although saturated granular materials are present, they have a normalized N value of greater than 22 blows per 0.3 metres. The liquefaction potential is considered to be low based on the soil profile type, age of the deposits, relative density and the historically low seismicity in this area. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments in accordance with the CHBDC:

- Select, free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type II but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and walls. The fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment wall in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with SP 105S10.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).
- For Case a, the restrained case, the pressures are based on the proposed approach fill materials and the following parameters (unfactored) may be used.

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.36
At rest, K_o	0.55
Passive, K_p	2.8

⁵ FHWA, 1997: "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



- For Case b, the unrestrained case, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>Type II</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.31	0.31
At rest, K_o	0.47	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design.
- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHBDC.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground behind the wall/abutment.

6.6 Embankments

It is anticipated that the approach embankments will be widened approximately 4 metres on each side. All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be removed from areas of proposed embankment widenings. Prior to placement of embankment fill, the exposed subgrade should be proofrolled under the direction of a geotechnical Quality Verification Engineer (QVE). The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 metres where pavement subbase and base materials will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts, properly benched into the existing embankments in accordance with OPSD 208.010 and adequately compacted.

Embankments no steeper than 2 horizontal to 1 vertical constructed on the near surface very stiff clayey silt are considered to be stable and are expected to achieve a Factor of Safety against a deep seated, rotational slope failure of 1.3 for embankments. Embankments greater than 8 metres in height should be provided with a 2 metre wide bench at mid-height.



6.7 Construction Considerations

The existing Franki piles at the site may be left in place provided the new structure is widened sufficiently on both the east and west sides to ensure the new piles are not obstructed by the existing piles.

Also, while not encountered during the current investigation, the native soils in the area are known to contain cobbles and boulders which may interfere with advancement of the piles. A NSSP should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with cobbles and boulders during pile installation.

6.7.1 Driven Piles

As previously indicated, due to the presence of artesian conditions at the site, driving shoes, reinforcement to flanges, splice plates and the like should add as little as possible to the pile cross-sectional dimensions. Further, it is recommended that the piles be driven from a 0.5 metre thick pad of Granular A which would act like a filter should sustained groundwater flow occur. A NSSP for a granular pad should be included in the Contract Documents.

6.7.2 Drilled Shafts

Slurry based or casing based construction methods will be necessary to advance drilled shafts into the cohesionless deposits in order to maintain caisson stability below the groundwater level.

As previously indicated, due to the presence of artesian groundwater conditions at the site, caissons should not be advanced deeper than elevation 255 metres in order to maintain the stability of the founding surface. Special care will be required during construction to ensure that drilled shafts do not extend below this elevation.

6.8 Excavations and Temporary Protection Systems

6.8.1 General

Excavations for the pile caps and/or abutments will penetrate the existing fill and organic materials and extend into the clayey silt. The groundwater level is expected to be at about elevation 265 metres and will fluctuate with seasonal and climatic variations. The excavations will likely extend below the groundwater level; however, seepage volumes are expected to be low and groundwater control may be achieved by using properly constructed and filtered sumps. Sumps should be maintained outside of the actual pile cap and/or abutment limits. Surface water runoff should be directed away from the excavations at all times.

It is expected that excavation of the soil between the existing abutments and new abutments will be completed after the bridge is constructed.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey materials and properly dewatered cohesionless materials would be classified as Type 2 soils.



6.8.2 Temporary Protection Systems

Temporary road and railway protection systems will be required where space is restricted and will not permit open cuts to support the sides of the excavation and permit the use of vertical cuts. The design and limits of the systems are to be determined by the contractor.

Temporary support systems could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds or driven steel sheet piling. Support to the systems could be in the form of struts and walers in the case of pile cap/abutment excavations or rakers and anchors in the case of railway protection. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.



7.0 FOUNDATION ENGINEERING FOR DETAILED DESIGN

Whether integral, semi-integral or conventional abutments are selected, deep foundations are considered the preferred foundation alternative for the bridge replacement. A Foundation Investigation and Design Report will need to be prepared during a future assignment to provide appropriate information for Detail Design. A standard MTO foundation investigation for a bridge structure is considered appropriate for the site. Specifically, a minimum of two boreholes should be advanced, one at each abutment opposite to existing boreholes 201 and 202, advanced to at least elevation 245 metres. Special attention should be paid to documenting the groundwater conditions encountered in the boreholes and provisions should be made to measure the head and flow if artesian conditions are encountered.

Sampling and SPT testing should be carried out at intervals of 0.75 metres within critical foundation zones. Routine soil testing, consisting of grain size analyses, water content determinations and Atterberg limits determinations, where applicable, is considered appropriate.

The preliminary recommendations given in this Preliminary Foundation Design Report should be expanded upon and updated in the Foundation Design Report for detail design in accordance with MTO's standard requirements for foundation engineering assignments. Detailed recommendations should be provided for foundations for the abutments and any wingwalls. If the vertical alignment of the highway or the approach embankments is to be significantly altered, embankment stability and settlement should be re-evaluated. Further, if staged construction is to be used for the construction of the replacement bridge, the discussion on temporary roadway protection should include lateral earth pressures and effect of ground conditions on shoring design.

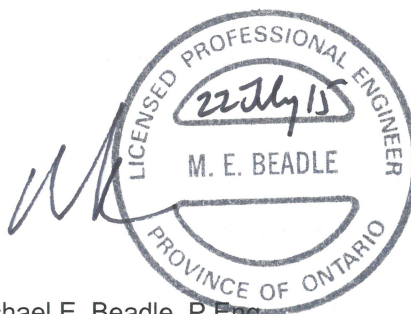


8.0 MISCELLANEOUS

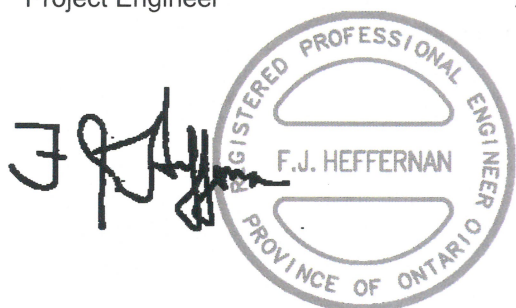
This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. and the Team Leader, Dr. Storer J. Boone, P. Eng. The report was reviewed by Mr. Michael E. Beadle, P.Eng., a senior geotechnical engineer and Associate. Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment conducted an independent quality review of the report.

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

CNR Overhead Replacement, Site 19-371
 Highway 401 Interchange Improvements
GWP 3054-11-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Spread footings supported on very stiff to hard clayey silt till or engineered fill.	<ul style="list-style-type: none"> • Not suitable for integral abutments. 	<ul style="list-style-type: none"> • Least expensive option. • Ease of construction. 	<ul style="list-style-type: none"> • Not compatible with integral abutments. • More settlement expected than with deep foundations. • Larger work area required compared to driven piles. 	<ul style="list-style-type: none"> • Low 	<ul style="list-style-type: none"> • Relatively low risk. • Deeper excavations required if soil at founding elevation is unsuitable.
End bearing steel H-piles or steel tube piles driven to refusal into very dense sandy silt to silt.	<ul style="list-style-type: none"> • Preferred technical alternative for abutments. 	<ul style="list-style-type: none"> • High bearing resistance. • Negligible settlement. • Compatible with all abutments types; however, steel tube piles may have insufficient flexibility for some integral abutment designs. • Depending on abutment design, may require less extensive excavations compared to shallow foundations. 	<ul style="list-style-type: none"> • More expensive than shallow foundations. • Piles can be damaged and deflected by cobbles and boulders within fill and glacial till deposits. • More construction noise and vibration compared to shallow foundations. • Cannot be visually inspected at depth. 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Possible pile tip damage if piles are not adequately protected while driving through fill materials and/or till deposits. • Potential risk for development of a void around the pile due to artesian conditions. • Variation in pile tip elevations.

COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Concrete caissons based in very stiff clayey soils.	<ul style="list-style-type: none"> • Preferred for piers. 	<ul style="list-style-type: none"> • Less construction noise and vibration compared to driven piles. • Faster construction and less work space required compared to shallow foundations. • Less potential for caissons to be impeded by cobbles in native till deposits compared to driven piles. 	<ul style="list-style-type: none"> • Potential for greater settlement compared to driven piles. • Not compatible with integral abutments. • Cannot be visually inspected at depth due to health and safety regulations. • Restriction on depth due to presence of artesian conditions (above elevation 255). 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Cleaning of base could be problematic or overlooked during construction. • Potential risk of base instability due to presence of artesian conditions.

- NOTES:
1. The estimated relative costs are intended to provide a comparison between alternatives rather than actual construction costs, and do not take into consideration additional costs such as traffic control, staging and shoring that may influence the total cost.
 2. Table to be read in conjunction with accompanying report.

Prepared By: NG
 Checked By: MEB

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.)

Consistency

	<u>kPa</u>	<u>psf</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

(b) Cohesive Soils

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
 2 shear strength = (compressive strength)/2
 * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

PROJECT 12-1132-0076		RECORD OF BOREHOLE No 201		1 OF 3	METRIC
W.P. 3054-11-00	LOCATION N 4755661.4 , E 411388.9	ORIGINATED BY BT			
DIST HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY WDF/LMK			
DATUM GEODETIC	DATE February 11, 2013 - February 12, 2013	CHECKED BY			

[illegible]

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

LDN_MTO_06 12-1132-0076-1001.GPJ LDN_MTO.GDT 11/11/13

PROJECT <u>12-1132-0076</u>		RECORD OF BOREHOLE No 201		2 OF 3	METRIC
W.P. <u>3054-11-00</u>	LOCATION <u>N 4755661.4 , E 411388.9</u>	ORIGINATED BY <u>BT</u>			
DIST <u> </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>	COMPILED BY <u>WDF/LMK</u>			
DATUM <u>GEODETIC</u>	DATE <u>February 11, 2013 - February 12, 2013</u>	CHECKED BY <u> </u>			



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		w _p	w	w _L		GR SA SI CL			
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						WATER CONTENT (%)		
							20 40 60 80 100										
	SILT, trace to some clay, with clayey silt layers below about elev. 256.1m Compact to very dense Grey		15	SS	52		254							0 1 84 15			
							253										
			16	SS	34		252										
251.68							251										
17.83	SILTY CLAY, trace sand Very stiff Grey		17	SS	27		250										
251.01							249										
18.50	SILTY SAND, Compact Grey						248										
250.31							247										
19.20	SILT, some sand, some clay Compact to very dense Grey		18	SS	64		246										
							245										
							244										
							243										
							242										
							241										
245.74							240										
23.77	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey		21	SS	17		239										
							238										
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							82										

LDN_MTO_06 12-1132-0076-1001.GPJ LDN_MTO.GDT 11/11/13

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1132-0076</u>		RECORD OF BOREHOLE No 201		3 OF 3		METRIC	
W.P. <u>3054-11-00</u>		LOCATION <u>N 4755661.4 , E 411388.9</u>		ORIGINATED BY <u>BT</u>			
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>WDF/LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>February 11, 2013 - February 12, 2013</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)							
							20	40	60	80	100									
238.57	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff to hard Grey		25	SS	37		239													
30.94	END OF BOREHOLE Groundwater encountered at about elev. 267.1m during drilling on February 11, 2013. Standpipe damaged March 8, 2013 Water level measured in piezometer at elev. 266.3m on March 8, 2013. Water level measured in piezometer at elev. 266.2m on April 3, 2013.													o						

RECORD OF BOREHOLE No 202

1 OF 2

METRIC

PROJECT 12-1132-0076
W.P. 3054-11-00 LOCATION N 4755565.5 , E 411356.4 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / WASH BORING, CASED COMPILED BY WDF/LMK
DATUM GEODETIC DATE February 21, 2013 - February 22, 2013 CHECKED BY

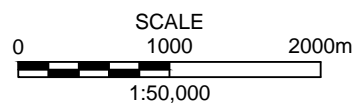
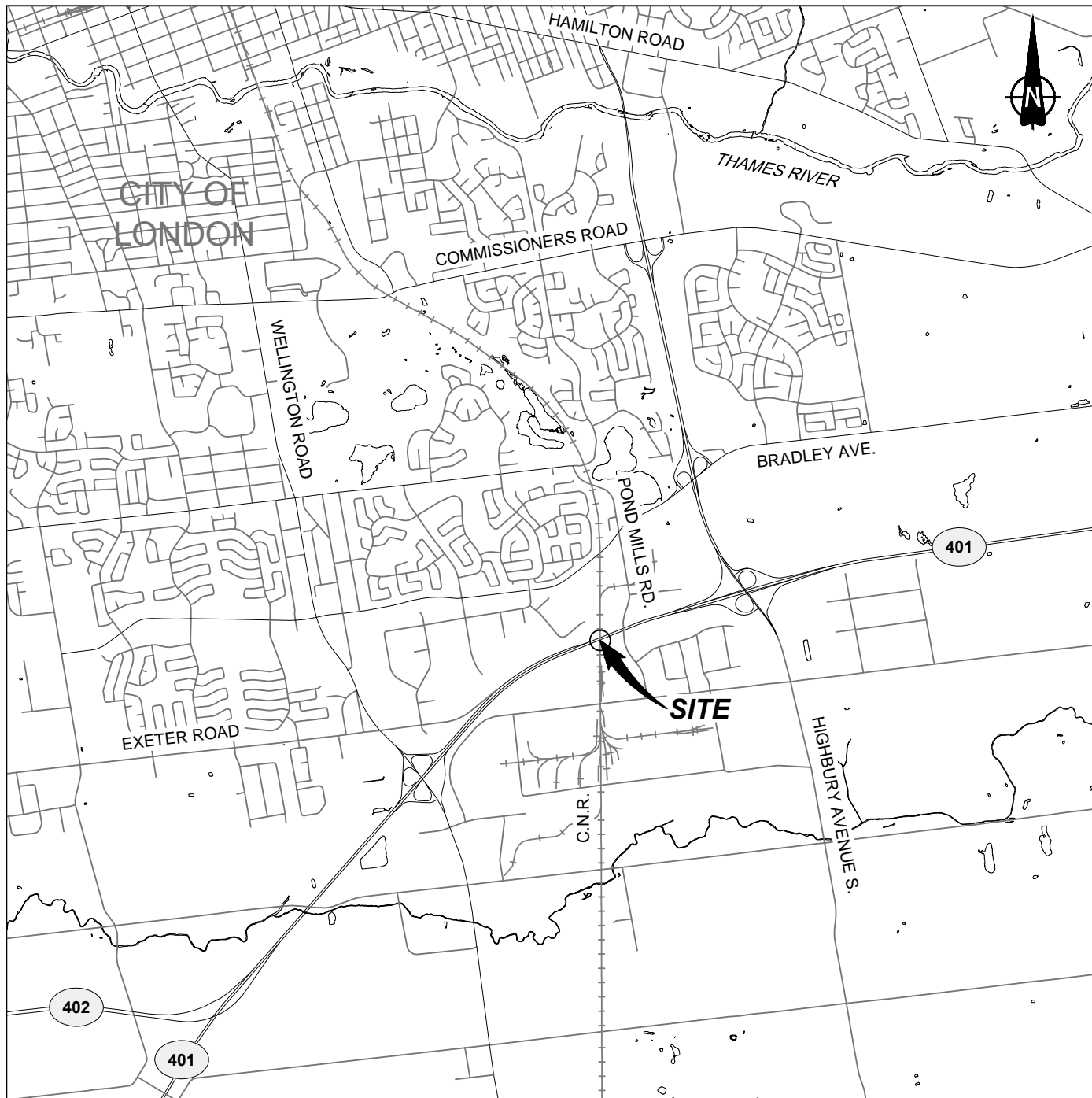
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)					
								20	40	60						80	100				
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						× LAB VANE					
267.40	GROUND SURFACE																				
0.00	FILL, clayey silt, some sand, trace gravel, trace topsoil Stiff Brown		1	SS	9																
266.03	CLAYEY SILT, trace sand, trace gravel Very stiff Mottled brown		2	SS	25																
1.37			3	SS	24																
			4	SS	21																
263.74	SILT, some clay, trace sand, trace gravel, with clayey silt layers Dense Grey		5	SS	37																
3.66			6	SS	28																
262.98	SILT, trace to some clay, trace sand, with clayey silt layers Compact to very dense Grey		7	SS	28																
4.42			8	SS	33																
			9	SS	31																
			10	SS	25																
			11	SS	33																
			12	SS	54																
			13	SS	76																
254.60	CLAYEY SILT, trace sand, (TILL) Very stiff Grey		14	SS	28																
12.80																					

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>12-1132-0076</u>		RECORD OF BOREHOLE No 202		2 OF 2	METRIC
W.P. <u>3054-11-00</u>		LOCATION <u>N 4755565.5 , E 411356.4</u>		ORIGINATED BY <u>BT</u>	
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM / WASH BORING, CASED</u>		COMPILED BY <u>WDF/LMK</u>	
DATUM <u>GEODETIC</u>		DATE <u>February 21, 2013 - February 22, 2013</u>		CHECKED BY <u> </u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	w _p	w	w _L					



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT

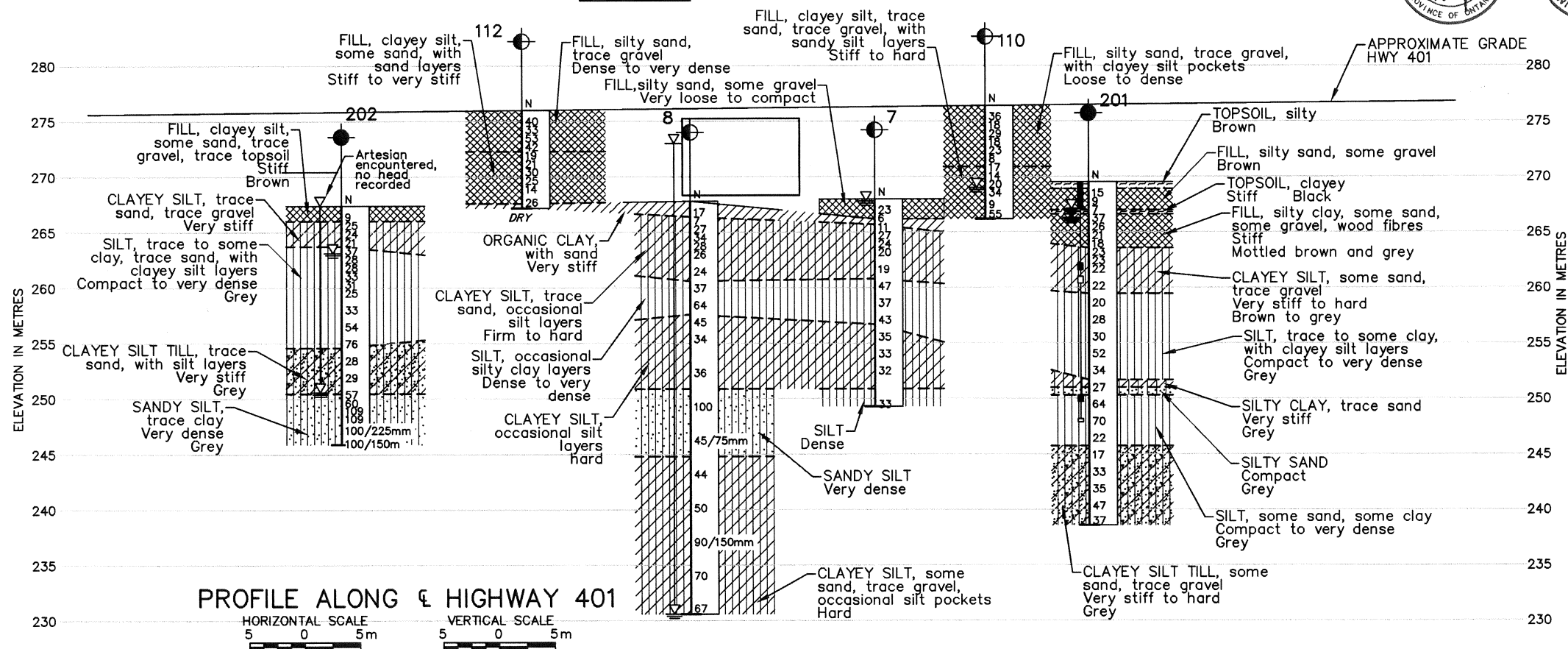
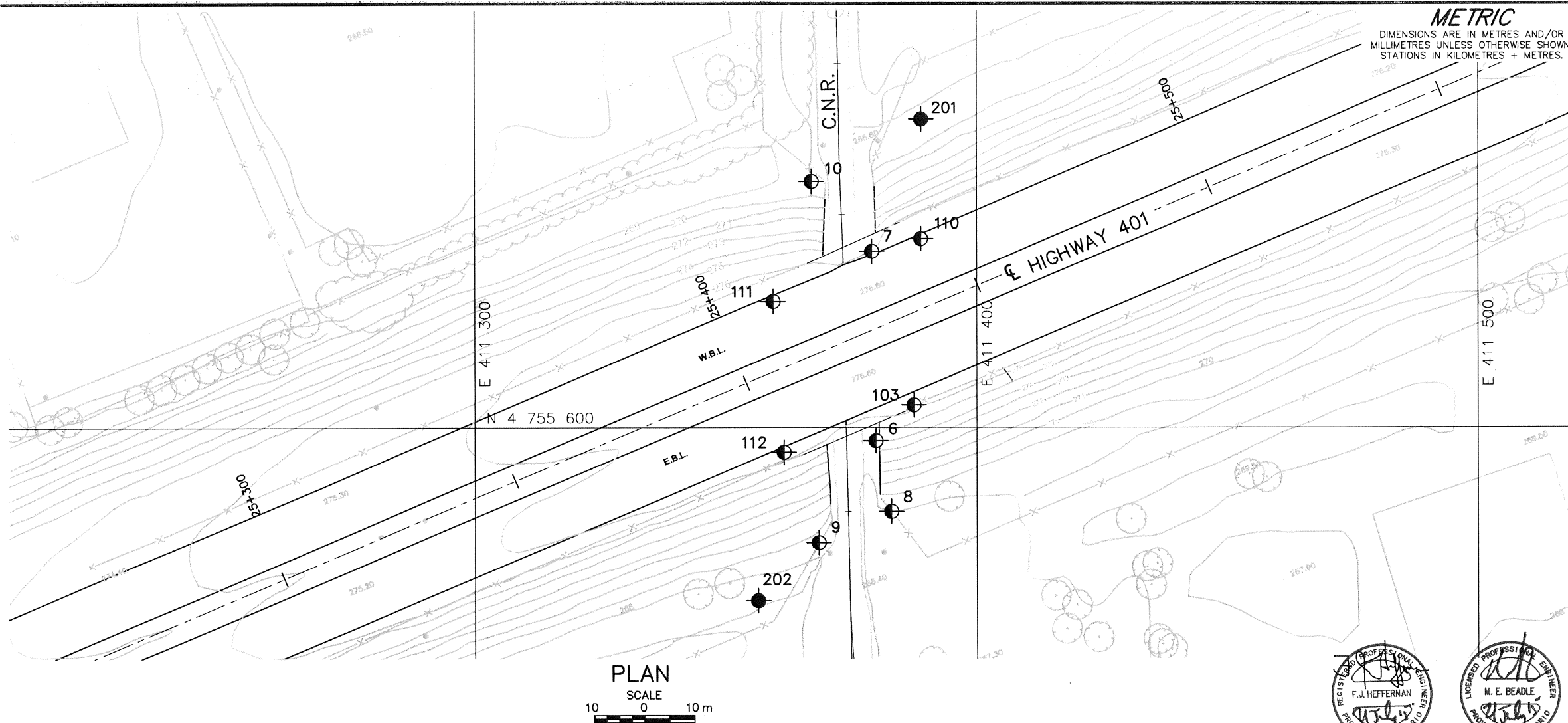
CNR OVERHEAD REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE

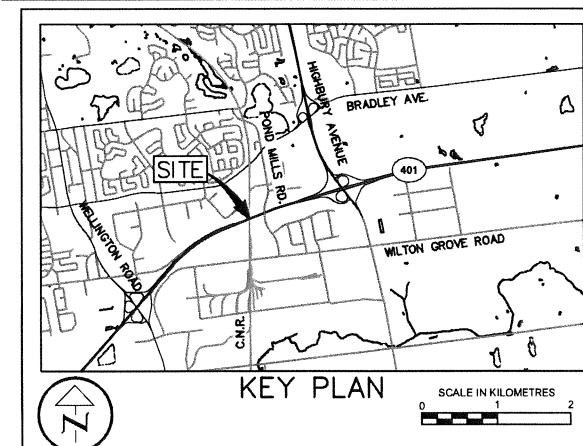
KEY PLAN



PROJECT No. 12-1132-0076		FILE No. 1211320076-1001-F03001	
CADD	LMKWDF	July 31/13	SCALE AS SHOWN REV. 0
CHECK			
			FIGURE 1

CONT No.
WP No. 3054-11-00CN RAIL OVERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA

LEGEND

- Borehole - Current Investigation
- Borehole (Geocres 40114-110)
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on March 8 & April 3, 2013
- Head
- ARTESIAN WATER
- WL encountered during drilling
- DRY Water level not established

No.	ELEVATION	CO-ORDINATES (MTM ZONE 11)	
		NORTHING	EASTING
201	269.51	4 755 661.4	411 388.9
202	267.40	4 755 565.5	411 356.4
Geocres 40114-110			
6	268.3	4 755 597.4	411 379.8
7	268.0	4 755 635.1	411 379.0
8	267.8	4 755 583.3	411 382.9
9	268.3	4 755 577.1	411 368.4
10	269.0	4 755 649.0	411 367.0
103	276.2	4 755 384.9	411 387.2
110	276.4	4 755 637.6	411 388.8
111	276.3	4 755 625.1	411 359.4
112	276.0	4 755 595.1	411 361.5

NOTES

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

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

REFERENCE

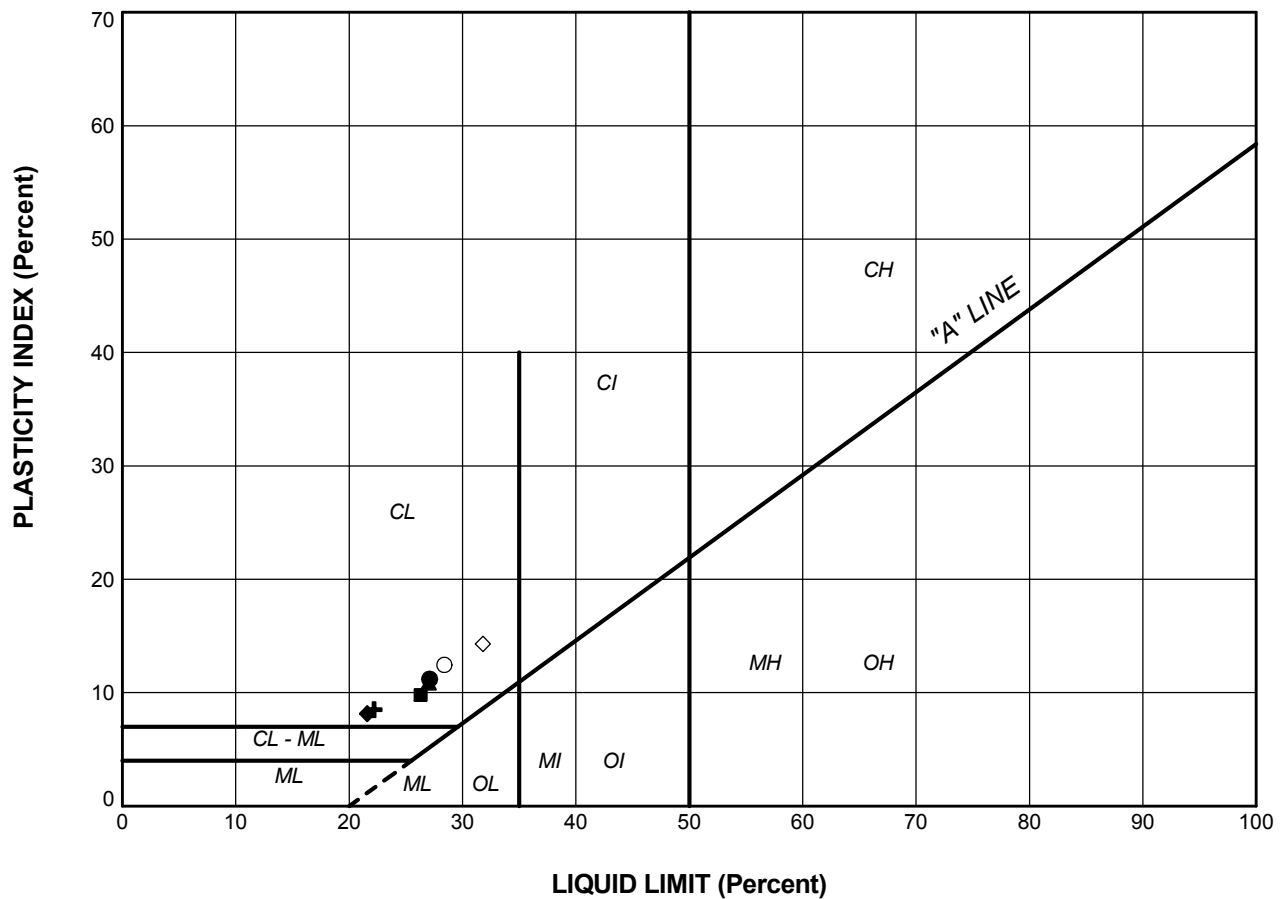
Base plans based on City of London Digital Mapping Disc 2011 (converted to MTM ZONE 11)

NO.	DATE	BY	REVISION
Geocres No. 40114-156			
HWY.	401	PROJECT NO. 12-1132-0076	DIST.
SUBM'D.	NG	CHKD. NAG	DATE: Mar. 25/15
DRAWN:	WDF/LMK	CHKD. MEB	APPD. FJH
			DWG. 1



APPENDIX A

Laboratory Test Data




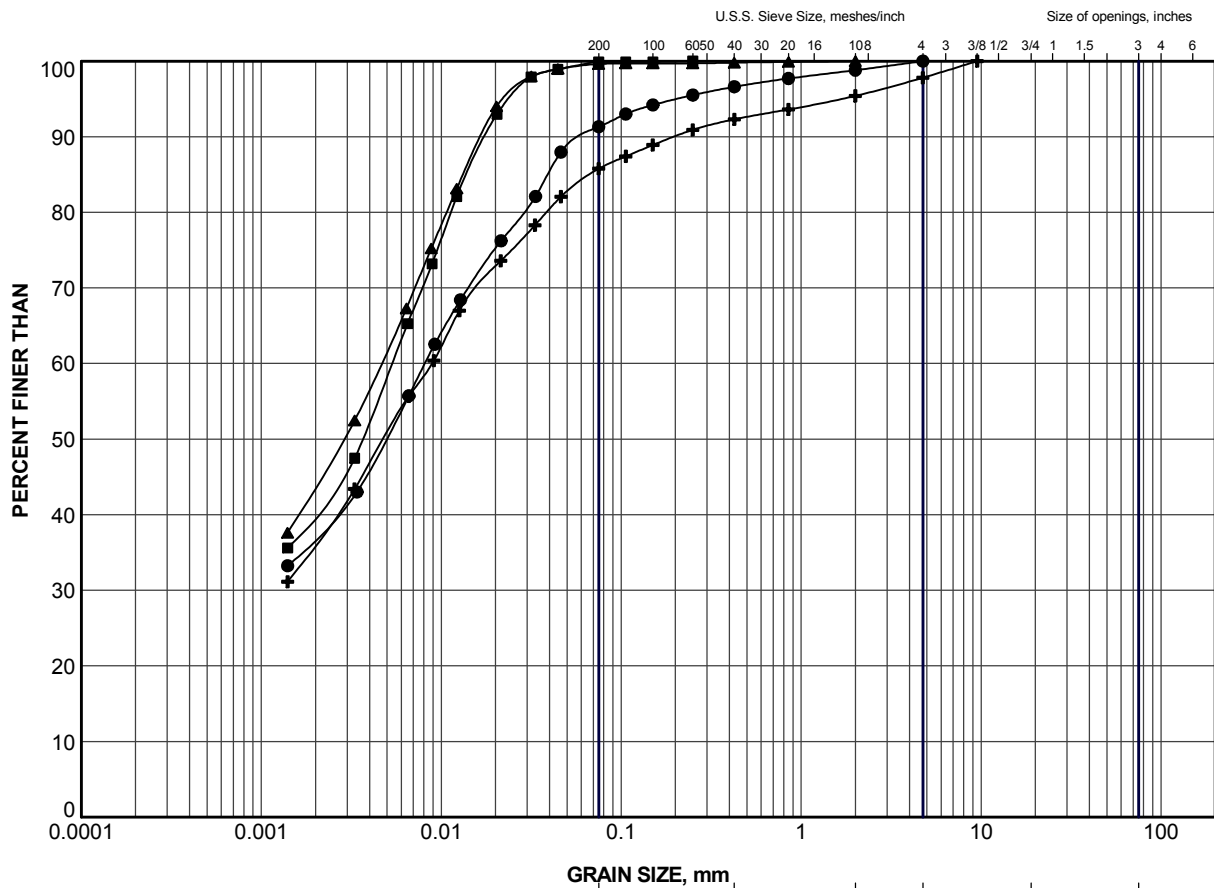
SOIL TYPE
 C = Clay
 M = Silt
 O = Organic

PLASTICITY
 L = Low
 I = Intermediate
 H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
CLAYEY SILT					
●	201	6	27.1	15.9	11.2
■	201	9	26.3	16.5	9.8
▲	201	10	27.0	16.2	10.9
◇	202	3	31.8	17.5	14.3
CLAYEY SILT TILL					
◆	201	22	21.6	13.5	8.2
○	202	15	28.4	16.0	12.5
SILT					
+	201	14	22.2	13.7	8.5


PROJECT			
CNR OVERHEAD REPLACEMENT HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3054-11-00			
TITLE			
PLASTICITY CHART			
PROJECT No. 12-1132-0076		FILE No.1211320076-1001-R030A1	
DRAWN	LMK/WDF	July 31/13	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-1

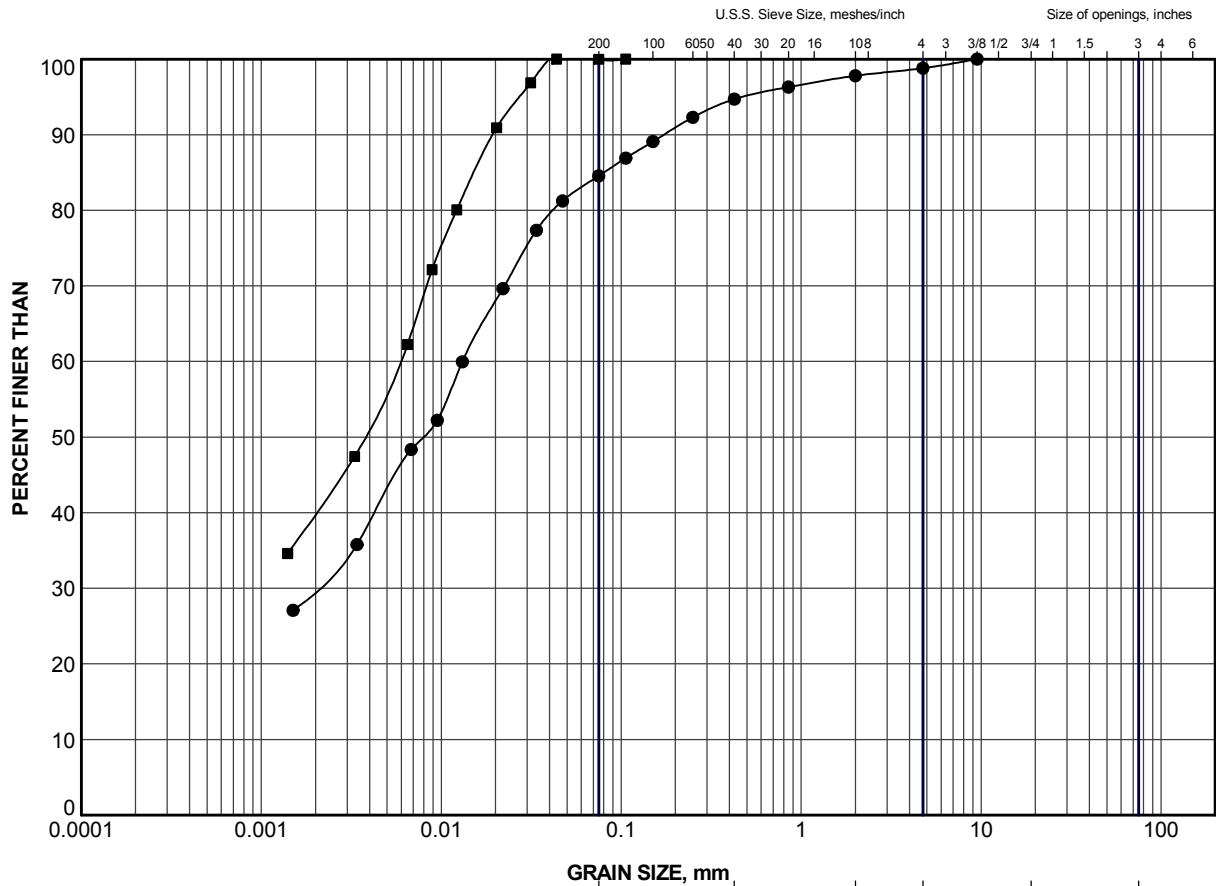


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	6	264.7
■	201	9	262.4
▲	201	10	261.7
+	202	3	264.9


PROJECT				CNR OVERHEAD REPLACEMENT HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3054-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		12-1132-0076		FILE No.		1211320076-1001-R030A2	
DRAWN		LMK/WDF		SCALE		N/A	
CHECK				REV.			
 Golder Associates LONDON, ONTARIO				FIGURE A-2			

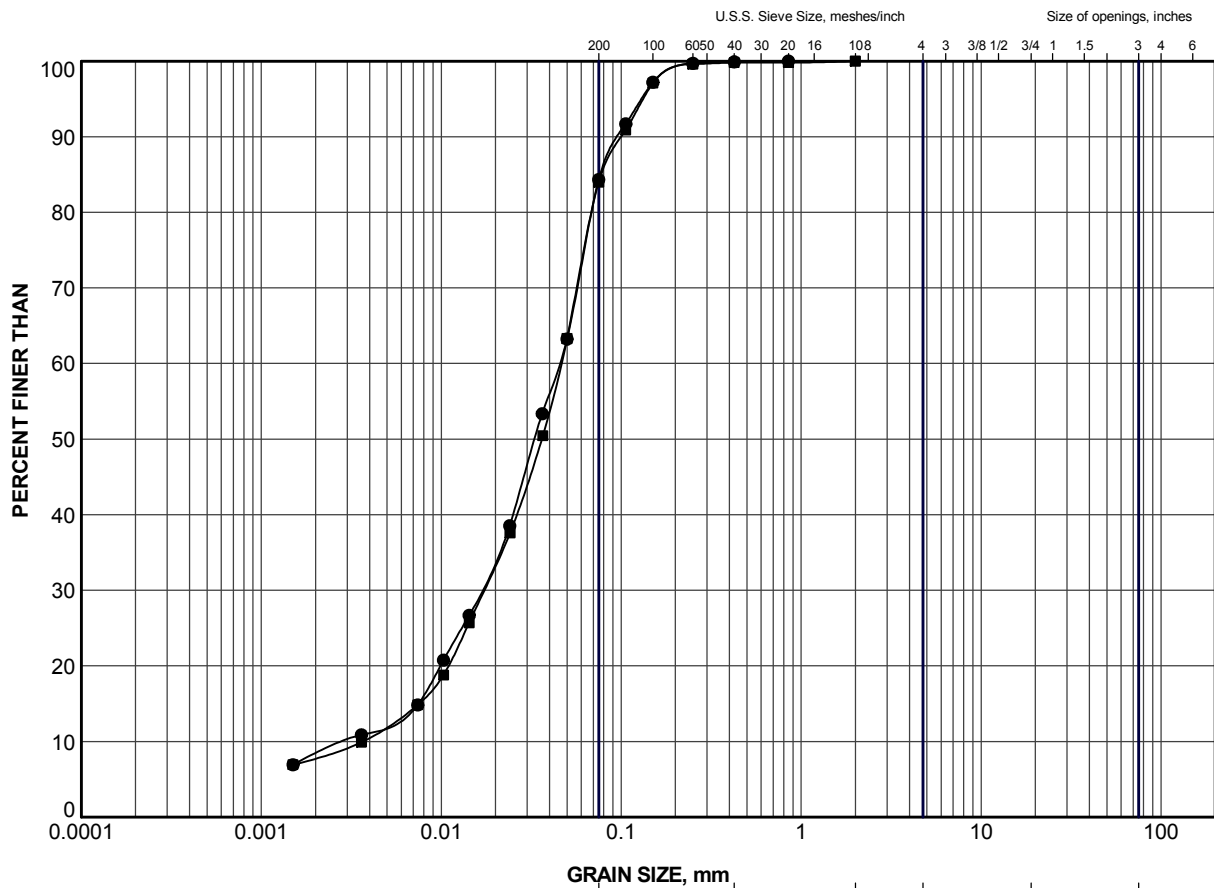


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	22	243.4
■	202	15	251.9


PROJECT				CNR OVERHEAD REPLACEMENT HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3054-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		12-1132-0076		FILE No.		1211320076-1001-R030A4	
DRAWN		WDF		SCALE		N/A	
CHECK				REV.			
		July 31/13					
 Golder Associates LONDON, ONTARIO				FIGURE A-4			



GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	17	249.5
■	202	19	248.1

PROJECT				CNR OVERHEAD REPLACEMENT HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3054-11-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT			
PROJECT No.		12-1132-0076		FILE No.		1211320076-1001-R030A5	
DRAWN		LMK/WDF		SCALE		N/A	
CHECK				REV.			
 Golder Associates LONDON, ONTARIO				FIGURE A-5			



APPENDIX B

Site Photograph



APPENDIX B SITE PHOTOGRAPH



Photograph 1: CNR Overpass, looking north.

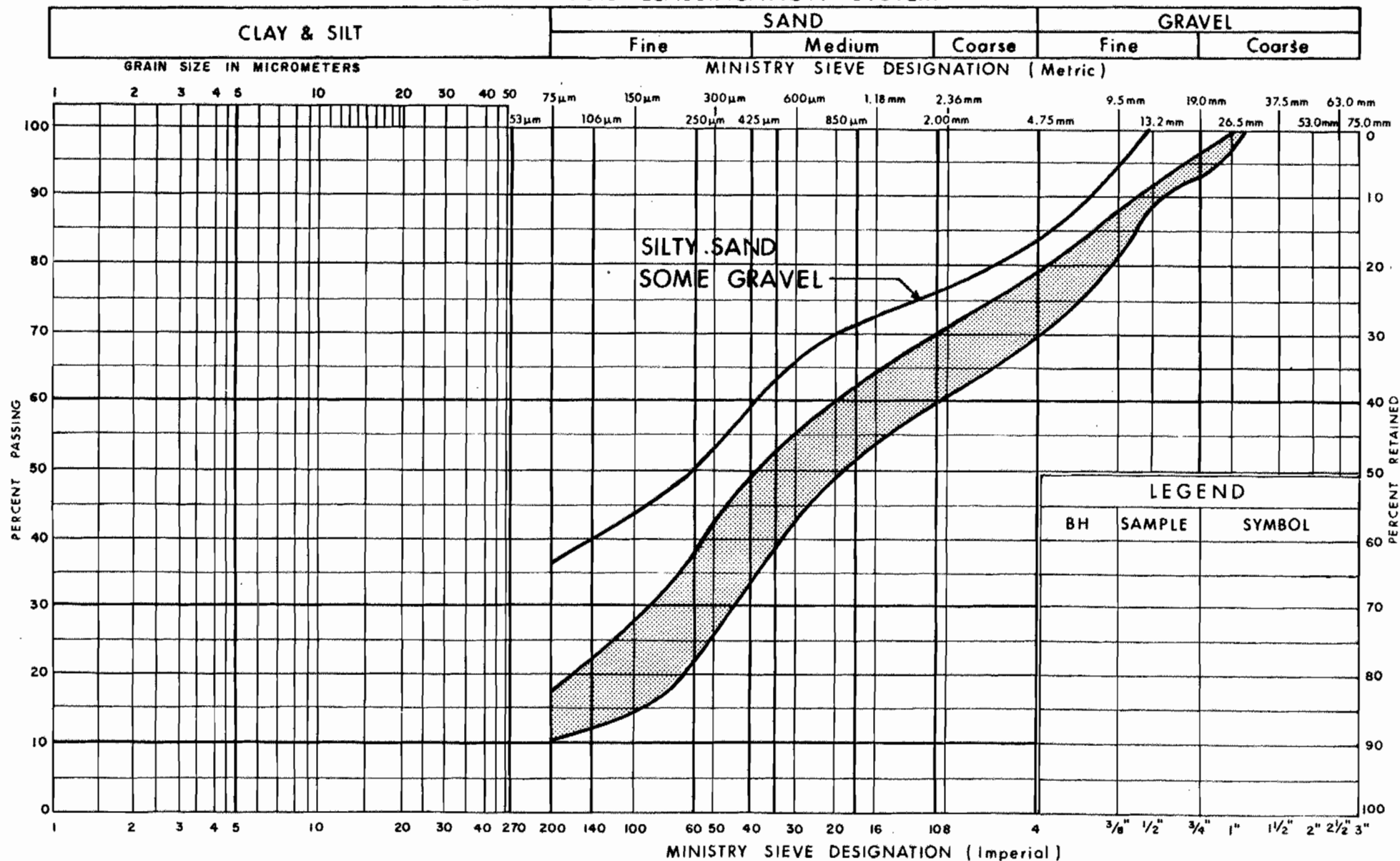
n:\active\2012\1132 - geo\1132-0000\12-1132-0076 dillon-11 structures-3011-e-0046\ph 1000 gwp 3011-e-0046\ph 1001 fdns\rpts\r03 cnr overhead site 19-371\1211320076-1001-r03 jul 22 15 (final) app b-site photos.docx



APPENDIX C

Record of Boreholes - Geocres No. 40I14-110

UNIFIED SOIL CLASSIFICATION SYSTEM



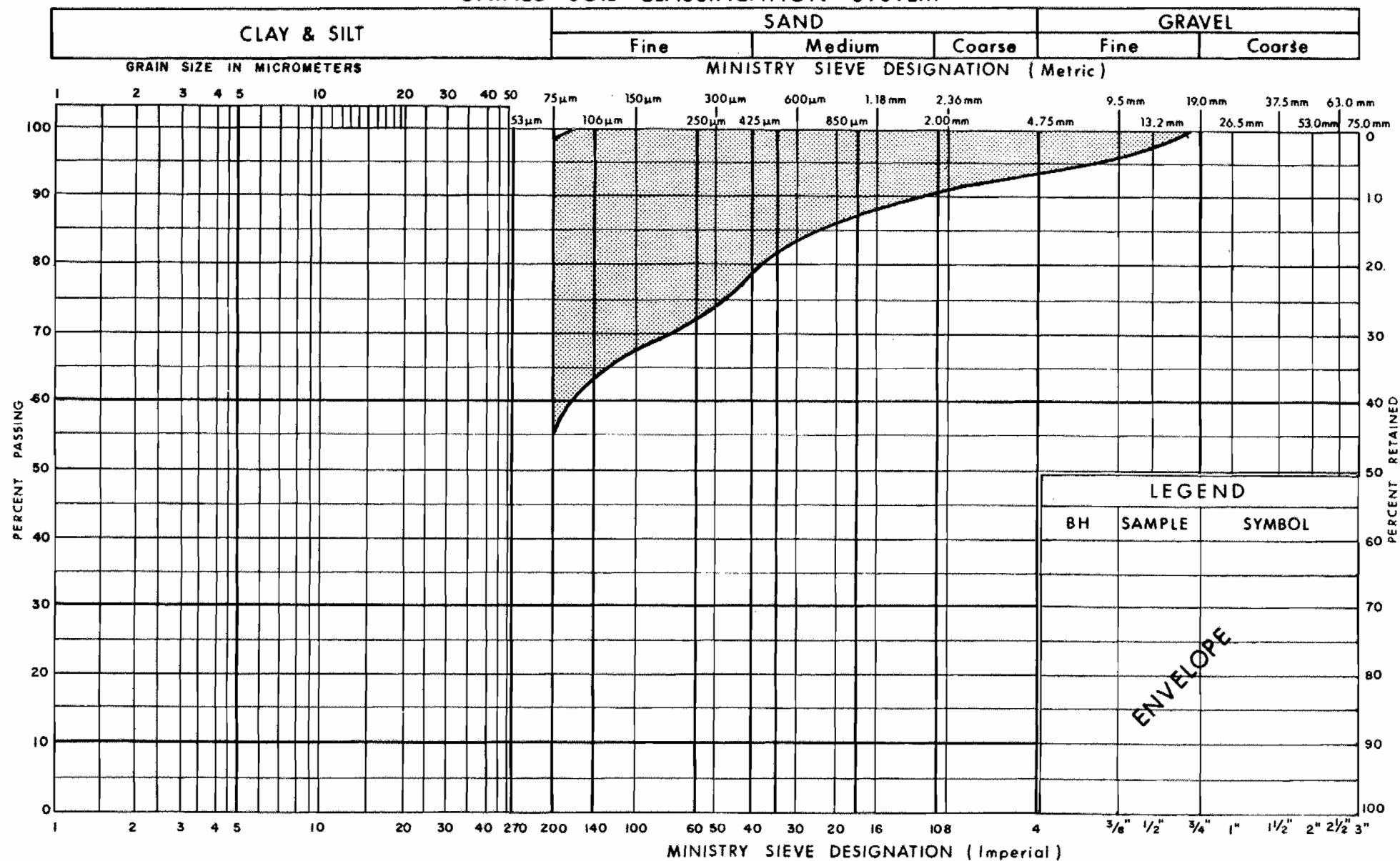
**Ministry of
Transportation and
Communications**

GRAIN SIZE DISTRIBUTION
GRAVELLY SAND SOME SILT (FILL MATERIAL)

FIG No 1

W P 139 - 86 - 03

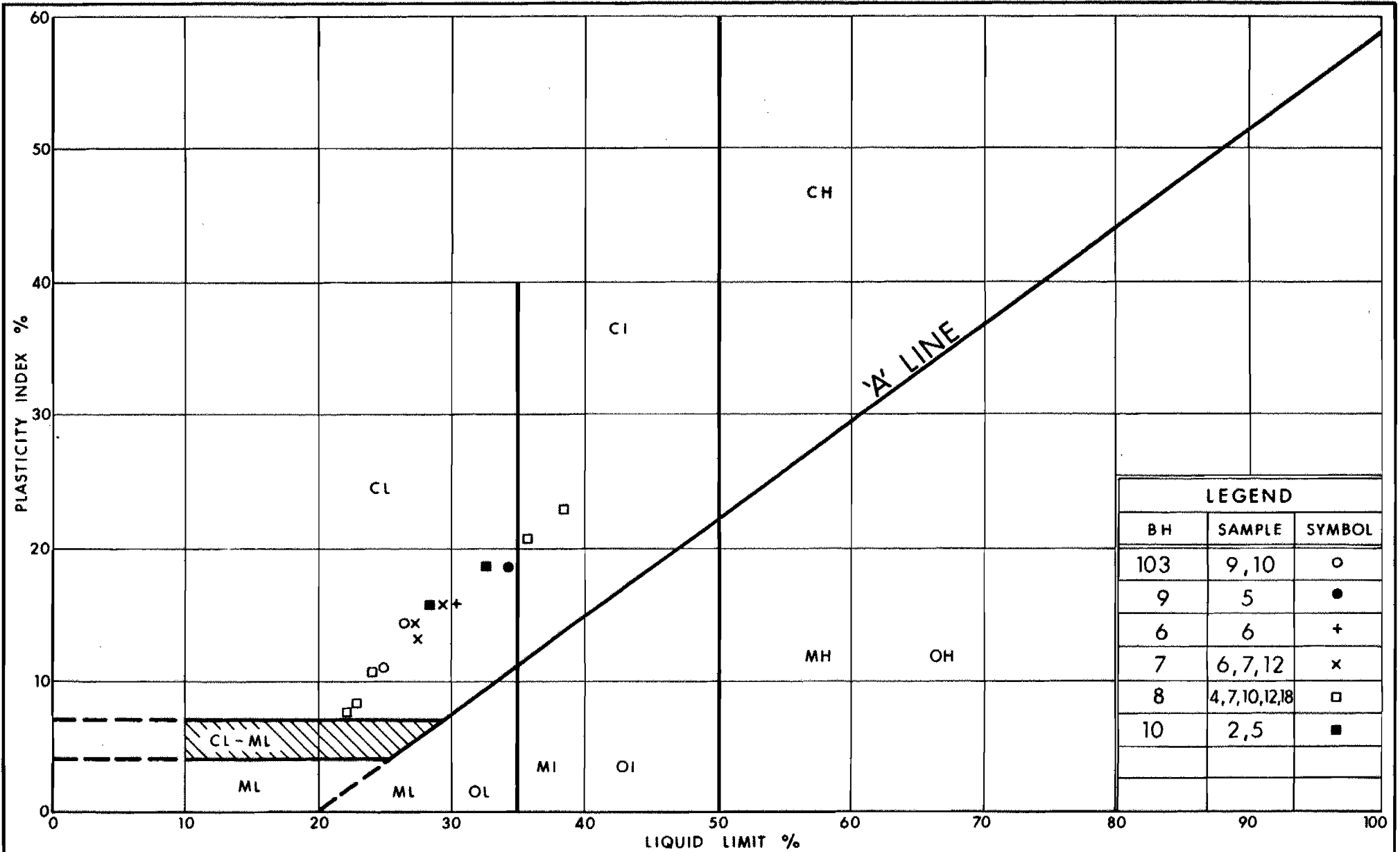
UNIFIED SOIL CLASSIFICATION SYSTEM

Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILTY CLAY, WITH SAND

FIG No 2

W P 139-86-03



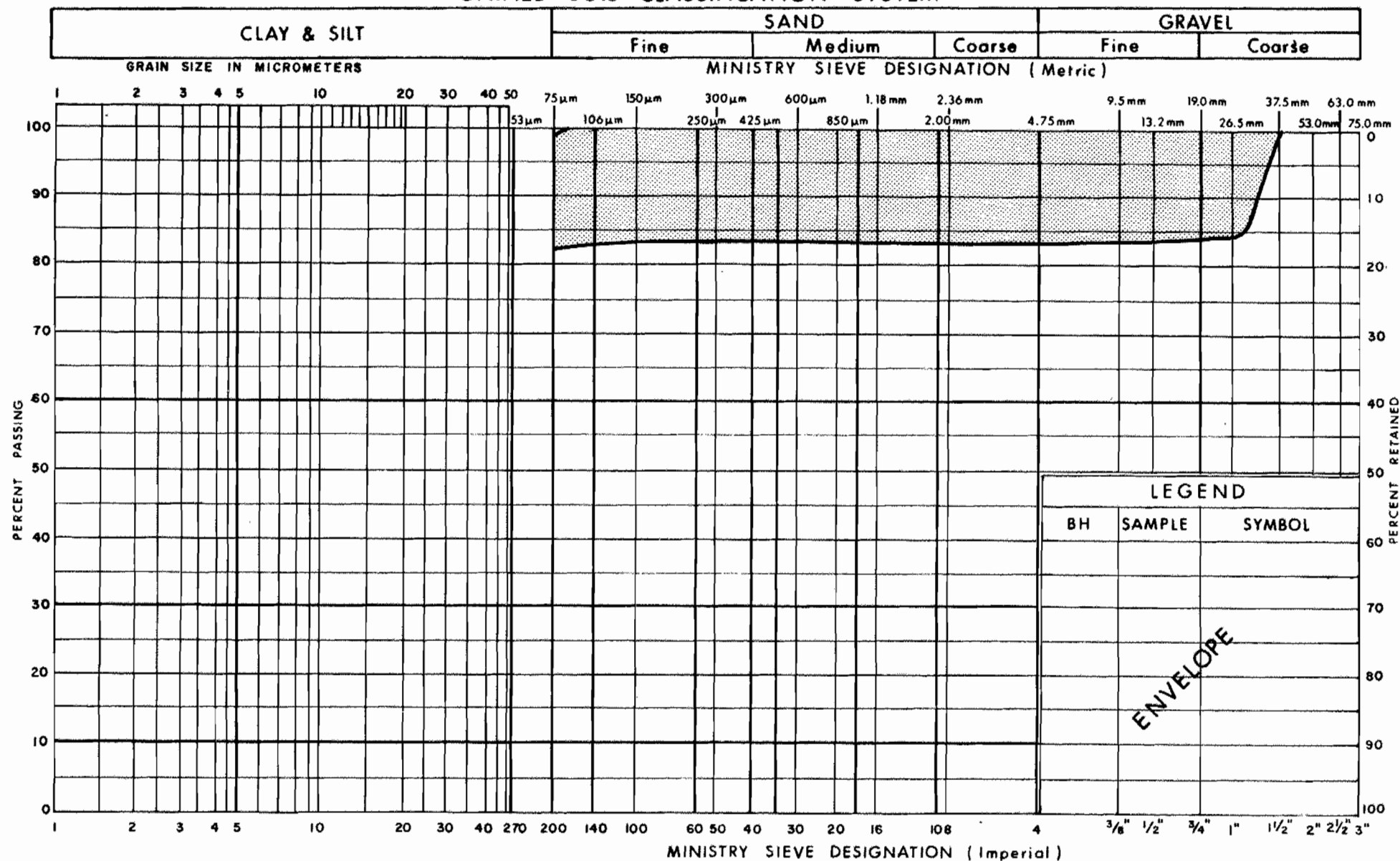
Ministry of
Transportation and
Communications

PLASTICITY CHART SILTY CLAY, WITH SAND

FIG No 3

W P 139 - 86 - 03

UNIFIED SOIL CLASSIFICATION SYSTEM



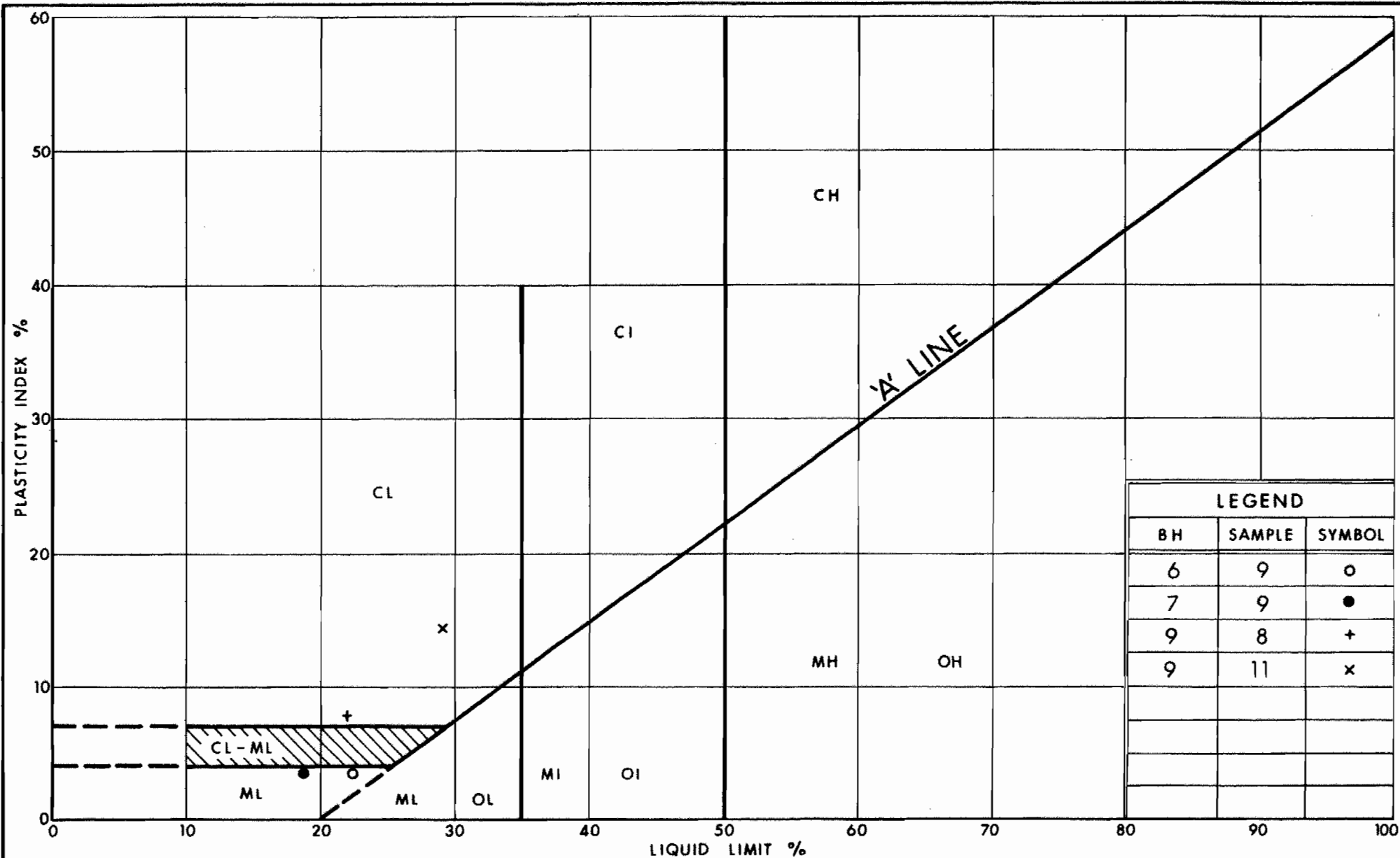
Ontario

Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILT, TO SOME SAND, GRAVEL

FIG No 4

W P 139 - 86 - 03



Ministry of
Transportation and
Communications

PLASTICITY CHART SILT, TRACE OF SAND, OCCASIONAL SILTY CLAY LAYERS

FIG No 5

W P 139-86-03



RECORD OF BOREHOLE No 6

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 377.8 E 411 370.6
DIST 2 HWY 401 BOREHOLE TYPE Continuous Flight Auger
DATUM Geodetic DATE 86 11 17

ORIGINATED BY DC
COMPILED BY DM
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
268.3	Ground Level													GR SA SI CL
0.0	Gravelly Sand some silt Very Loose to Compact (Fill Material)		1	SS	29		268							21 67 (12)
			2	SS	5									
265.6			3	SS	3		266							30 55 (15)
2.7	Silty Clay some sand trace of gravel Very Stiff to Hard		4	SS										
			5	SS	22									
			6	SS	25									6 31 (63)
262.7			7	SS	43		264							
5.6														
	Silt some gravel trace sand Dense to Very Dense		8	SS	82		262							
			9	SS	86									
			10	SS	77		260							16 2 (82)
			11	SS	49		258							
255.6			12	SS	55		256							
12.7	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 7

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 415.5 E 411 369.8 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (HS) COMPILED BY PM
DATUM Geodetic DATE 86 11 18 to 86 11 19 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
268.0	Ground Level													
0.0	Silty Sand some gravel Very Loose to Compact (Fill Material)		1	SS	23									18 46 (36)
266.2			2	SS	6									
1.8	Organic Clay		3	SS	11									
2.3			4	SS	27									
	Silty Clay trace of sand		5	SS	24									0 1 (99)
	occ. silt layers		6	SS	20									0 0 (100)
	Stiff to Very Stiff		7	SS	19									
260.8														
7.2	Silt		8	SS	47									
	Dense		9	SS	37									0 0 (100)
	Occ. Silty Clay Layers		10	SS	43									
256.7														
11.3	Silty Clay		11	SS	35									
	occ. silt layers		12	SS	33									0 0 (100)
	Hard		13	SS	32									
250.9														
17.1	Silt													
249.3	Dense		14	SS	33									
18.7														

OFFICE REPORT ON SOIL EXPLORATION

METRIC

W P 139-86-03

LOCATION Co-ords: N 4 755 363.7 E 411 373.7

ORIGINATED BY DC

DIST 2 HWY 401

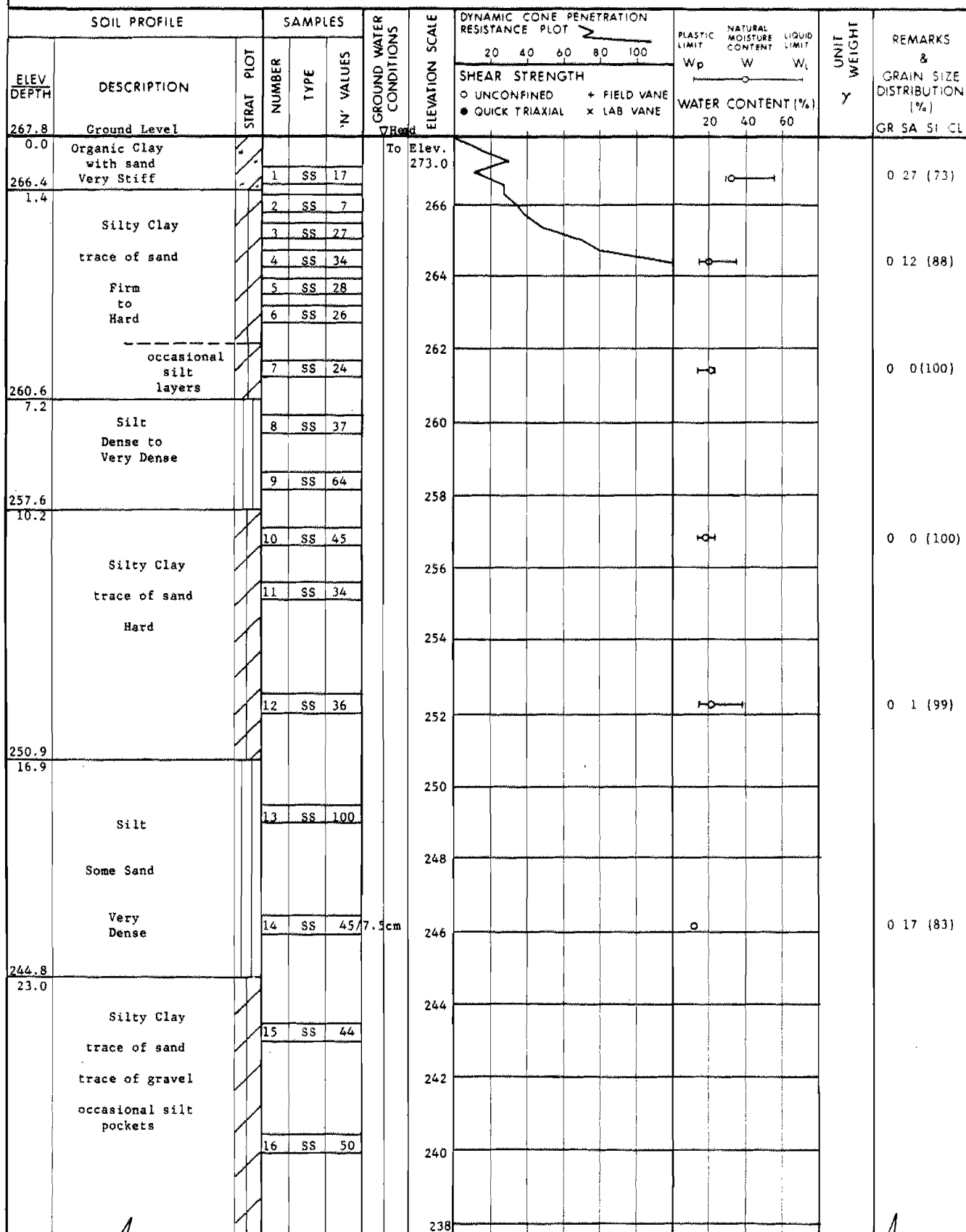
BOREHOLE TYPE Continuous Flight Auger (HS) & BX Casing

COMPILED BY DM

DATUM _____ Geodetic

DATE 86 11 19 and 86 11 20

CHECKED BY _____



Continued

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

Continued



RECORD OF BOREHOLE No 8 Cont

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 363.7 E 411 373.7 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Continuous Flight Auger (HS) & BX Casing COMPILED BY PM
DATUM Geodetic DATE 86 11 19 and 86 11 20 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
237.6	Continued															
30.2			17	SS	90	15 cm										
	Hard		18	SS	70		236									
							234									2 10 (88)
							232									
230.6			19	SS	67											
37.2	End of Borehole					Artesian Encountered El. 280.6										

+3', x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 9

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 357.5 E 411 359.2 ORIGINATED BY DC
 DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (HS) COMPILED BY PM
 DATUM Geodetic DATE 86 11 28 to 86 12 01 CHECKED BY JA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
268.3	Ground Level														GR SA SI CL
0.0	Organic Clay trace of sand traces of wood fibres Stiff		1	SS	15		268								
266.2			2	SS	13		266								
2.1	Silty Clay traces of organics trace of sand trace of gravel Very Stiff		3	SS	18		264								0 8 (92)
			4	SS	25										
			5	SS	30										
			6	SS	26										
	Occasional Silt Pockets		7	SS	20		262								
261.1			8	SS	21		260								0 0 (100)
7.2	Silt Occasional Silty Clay Layers		9	SS	39		258								
			10	SS	29		256								0 0 (100)
			11	SS	32		254								
			12	SS	38		252								
	Compact to Very Dense		13	SS	52/15 cm		250								
249.7	End of Borehole														
18.6															

+3, x5: Numbers refer to
Sensitivity

20
15
10


5 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 10

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 429.4 E 411 357.8 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Continuous Flight Auger (HS) COMPILED BY DM
DATUM Geodetic DATE 86 12 01 and 86 12 02 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
269.0	Ground Level												GR SA SI CL
0.0	Silty Clay trace/with sand traces of organics Very Stiff to Hard		1	SS	20		268						0 30 (70)
			2	SS	35		266		120/28 cm				0 1 (99)
			3	SS	50								
			4	SS	46								
			5	SS	45								
	occasional silt pockets		6	SS	55								
263.4			7	SS	49			264					
5.6			8	SS	46			262					
	Silt		9	SS	44			260					
	trace of sand		10	SS	61			258					
	Compact to Very Dense		11	SS	101			256					
			12	SS	38			254					0 3 (97)
								252					
							250						
							248						
247.2			13	SS	22								
21.8	End of Borehole												

+3, x5 : Numbers refer to
Sensitivity

20
15 \div 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 103

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 384.9 E 411 378.2 ORIGINATED BY DM
DIST 2 HWY 401 BOREHOLE TYPE Washbore - BX & NX Casings COMPILED BY DM
DATUM Geodetic DATE 86 11 25 to 86 11 28 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
								SHEAR STRENGTH						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL x LAB VANE						
								WATER CONTENT (%)						
276.2	Ground Level													
0.0														
	Gravelly Sand		1	SS	17		276							
	some silt		2	SS	20									
	Compact		3	SS	58		274							
	to		4	SS	41								22 62 (16)	
	Very Dense						272							
	(Fill Material)		5	SS	68									
			6	SS	57		270							
			7	SS	68									
268.7			8	SS	32									
7.5	Silty Clay with/some		9	SS	24		268						0 28 (72)	
	sand trace of gravel		10	SS	30								1 42 (57)	
267.2	Very Stiff													
9.0	Gravelly Sand some		11	SS	56								35 44 (21)	
266.6	Silt Very Dense													
9.6	End of Borehole													

+3, x5: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 110

METRIC

W P 139-86-03 LOCATION Co-ords: N 4 755 418.0 E 411 379.6 ORIGINATED BY PD
 DIST 2 HWY 401 BOREHOLE TYPE Continuous Flight Auger (H.S.) COMPILED BY PM
 DATUM Geodetic DATE 89 02 28 CHECKED BY SD

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
276.4	Ground Level													
0.0	Shoulder Material		1	SS	36		276							
	Silty Sand		2	SS	18		274							
	Trace of Gravel		3	SS	29		272							0 62 (38)
	occ. Pockets of Clayey Silt (Fill)		4	SS	18		270							2 24 (74)
	Dense to Loose		5	SS	23		268							5 38 (57)
270.9			6	SS	8									6 7 48 39
5.5	Clayey Silt With/Trace Sand Trace of Gravel (Fill)		7	SS	17									
			8	SS	14									
			9	SS	20									
			10	SS	34									
	Sandy Silt (ML)		11	SS	9									
266.2	Stiff to Hard		12	SS	55									
10.2	End of Borehole													

RECORD OF BOREHOLE No 111

METRIC

W P 139-86-03 LOCATION Station: 25+409.7, Offset 14m Lt of C ORIGINATED BY PD
 DIST 2 HWY 401 BOREHOLE TYPE Continuous Flight Auger (H.S.) COMPILED BY PM
 DATUM Geodetic DATE 89 02 28 CHECKED BY SDT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
276.3	Ground Level													
0.0	Shoulder Material						276							
			1	SS	40									
	Silty Sand with Gravel (Fill)		2	SS	33		274							
			3	SS	47									
	Occ. Pockets of Clayey Silt		4	SS	34									
			5	SS	7		272							
	Dense to Loose		6	SS	10									
269.3			7	SS	20		270							
7.0	Clayey Silt with to Some Sand		8	SS	19									
	Trace of Gravel (Fill)		9	SS	8		268							
	Stiff/Very Stiff													
265.2			10	SS	31		266							
11.1	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 112

METRIC

W P 139-86-03 LOCATION Station: 25+400; Offset 14.25m Rt of C
 Co-ords: N 4 755 375.5; E 411 352.3
 DIST 2 HWY 401 BOREHOLE TYPE Continuous Flight Auger (H.S.)
 DATUM Geodetic DATE 89 03 01
 ORIGINATED BY PD
 COMPILED BY PM
 CHECKED BY JDT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
276.0	Ground Level																
0.0	Shoulder Material																
	Silty Sand		1	SS	40												
	Trace of Gravel		2	SS	33												
	Occ. Pockets of Clayey Silt		3	SS	53												
	(Fill)																
272.3	Dense to Very Dense		4	SS	42 *												
3.7	Clayey Silt		5	SS	19												
	Some Sand		6	SS	21												
	Sand With Clayey Silt		7	SS	30												
	(Fill)		8	SS	25												
	Stiff to Very Stiff		9	SS	14												
267.2	Organic (Original Grd.)		10	SS	26												
8.8	End of Borehole																
	* Water Level not Established																

OFFICE REPORT ON SOIL EXPLORATION

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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