



June 2015

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Pond Mills Road Overpass Replacement
Site Number 19-372
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 1 – 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

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Site Photographs

APPENDIX C

Record of Boreholes - Geocres Report No. 40114-111



PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

**POND MILLS ROAD OVERPASS REPLACEMENT
SITE NUMBER 19-372
HIGHWAY 401 INTERCHANGE IMPROVEMENTS/STRUCTURAL REPLACEMENTS
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 that includes ten (10) bridges, two (2) culverts, and improvements at five (5) Highway 401 interchanges.

This report addresses the replacement of the Pond Mills Road Overpass (Site 19-372) for GWP 3054-11-00.

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 Highway 401 Pond Mills Road Overpass is located in the City of London, Ontario. The location of the project is shown on the Key Plan, Figure 1. For the purposes of this report, Highway 401 and Pond Mills Road are assumed to be oriented in an east-west direction and a north-south direction, respectively. This section of Highway 401 is currently a six lane divided highway and the existing bridge structure at the Pond Mills Road overpass was constructed in 1955 and consists of a single span, concrete tee beam 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 new structure erected at the same location as the existing structure. Site Photographs are 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 Pond Mills Road Overpass site is situated on an undrumlinized till plane.¹ Geology mapping indicates that the surficial material consists of Port Stanley silty clay till and clayey silt till, in places covered by thin patches of lacustrine silt.²

The rock formation in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation which belongs to the Hamilton Group of Middle Devonian Age.³ The bedrock surface is estimated to be at about elevation 210 metres, some 65 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 6 and 19, and on May 14 and 15, 2013, during which time four boreholes were drilled at the 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		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
13-101	4 755 729	411 549	269.72	21.43
13-102	4 755 673	411 594	270.72	24.54
13-103	4 755 704	411 540	275.58	12.65
13-104	4 755 694	411 595	275.10	11.13

The investigation was carried out using all-terrain drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.75 to 1.5 metre intervals of depth using 50 millimetre outside diameter split-spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of ASTM D 1586. The recorded SPT N values are noted on the Record of Borehole sheets. According to ASTM D1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres after first having penetrated 150 millimetres. 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 in situ field tests (i.e., SPT N-values) as presented on the Record of Borehole sheets and in Section 4 of this report are field values and have not been adjusted for depth, hammer energy, or other factors. The boreholes were terminated between 11.1 and 24.5 metres below the existing pavement or ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. The boreholes were backfilled in general 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, obtained utility locates, 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, Atterberg limits determinations, and grain size distribution analyses were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

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. Data from boreholes 1 through 5, 101, and 102 from Geocres Report No. 40I14-111 entitled "Foundation Investigation Report for Pond Mills Road Overpass Widening" was used to supplement the current data.

The Record of Borehole sheets for 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. The locations as presented below have been converted to current MTM format.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
1	4 755 658	411 564	268.3	4.3
2	4 755 647	411 558	269.0	40.1
3A	4 755 727	411 564	269.0	3.5
3B	4 755 733	411 563	269.0	18.6
4	4 755 736	411 579	269.5	18.7
5	4 755 666	411 583	269.5	15.7
101	4 755 676	411 558	274.7	15.7
102	4 755 723	411 586	274.5	8.1

Boreholes 1, 2, and 5 were drilled south of Highway 401 in the vicinity of Pond Mills Road and boreholes 3A, 3B, and 4 were drilled north of Highway 401 in the vicinity of Pond Mills Road. Boreholes 101 and 102 (40I14-111) were drilled on the shoulders of the Highway 401 east and westbound lanes, respectively, through the existing abutment backfill. The 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 Appendix A. 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 the existing pavement structure, topsoil, and fill materials underlain by layers of clayey silt till, silt, sandy silt, clayey silt, and sand.

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 boreholes are provided on the Record of Borehole sheets and are summarized in the following sections. For the purpose of this report, the descriptions of soils encountered during the original geotechnical investigation, Geocres Report No. 40114-111, have been adjusted to reflect current soil classification procedures.

4.1.1 Pavement Structure

Boreholes 13-103 and 13-104 were advanced through the paved west and east bound shoulders of Highway 401, respectively. Approximately 120 and 110 millimetres of asphalt was encountered from the ground surface in boreholes 13-103 and 13-104, respectively. Underlying the asphalt approximately 180 to 230 millimetres of crushed sand and gravel road base was encountered, underlain by about 670 millimetres of sand and gravel sub-base. Cobbles were found to be present within the sand and gravel sub-base.

4.1.2 Topsoil

Approximately 300 and 460 millimetres of topsoil was encountered from the ground surface in boreholes 13-101 and 13-102, respectively. Also, approximately 210 and 240 millimetre thick buried layers of topsoil were encountered in borehole 13-103 at elevations 270.0 and 269.3 metres. The buried topsoil in borehole 13-103 had measured N values, as determined by the standard penetration testing, of 18 and 20 blows per 0.3 metres.

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.



4.1.3 Fill

Fill materials were encountered in boreholes 13-101 and 13-102 underlying the surficial topsoil, in boreholes 13-103 and 13-104 underlying the pavement structure, in borehole 13-103 underlying a buried topsoil layer, and from the ground surface in Geocres Report No. 40I14-111 boreholes 4, 101, and 102. The fill in boreholes 13-101, 13-102, and 4 (40I14-111) were 0.5, 2.9, and 3.7 metres thick, respectively. Boreholes 13-103, 13-104, 101 (40I14-111), and 102 (40I14-111) were advanced through the existing embankments and encountered some 4.2 to 6.7 metres of fill materials. Also, about 0.5 metres of fill was encountered between the buried topsoil layers in borehole 13-103. The fill materials were encountered to between elevations 265.8 and 269.9 metres.

The fill materials varied from loose to very dense sand, silty sand, and silt, and firm to hard clayey silt to silty clay. The granular fill materials had measured N values ranging from 4 to greater than 100 blows per 0.3 metres. Samples of silt and sand fills exhibited water contents of 6 to 22 per cent. The firm to hard clayey silt to silty clay fill materials had measured N values ranging from 4 to 40 blows per 0.3 metres. Samples of the cohesive fill materials exhibited water contents ranging from 6 to 21 per cent. Select samples of the cohesive fill materials had liquid limits of between 22 and 31 per cent, plastic limits of 14 and 15 per cent, and plasticity indices of between 8 and 16 per cent, indicating low plasticity. The results of the current Atterberg limits determinations are shown on Figure A-1. The results of grain size determinations carried out on samples of the clayey fill materials are shown on Figure A-2.

4.1.4 Clayey Silt Till

In all but one case, materials described as silty clay in the boreholes advanced for Geocres No. 40I14-111 have been classified as clayey silt till based on comparison of the results of the grain size analyses and Atterberg limits determinations and comparison of the stratigraphy in adjacent 2013 boreholes.

Soft to hard clayey silt glacial till was encountered in boreholes 13-101 to 13-104, 101 (40I14-111), and 102 (40I14-111), underlying the fill and topsoil materials, from the ground surface in boreholes 1 (40I14-111), 2 (40I14-111), 3A (40I14-111), and 5 (40I14-111), in borehole 13-102 underlying a layer of sandy silt, and in borehole 13-104 underlying a layer of silt. The clayey silt till layers were between about 0.8 to 10.7 metres thick where fully penetrated and were encountered at between elevations 265.8 to 269.9 metres. Geocres Report No. 40I14-111 boreholes 1, 3A, 101, and 102 were terminated in the clayey silt till after exploring some 2.9 to 9.0 metres.

The clayey silt till had measured N values ranging from 3 to 86 blows per 0.3 metres, more typically in the range of 14 to 50 blows per 0.3 metres. Samples of the clayey silt till exhibited water contents of between 11 and 20 per cent. Select samples of clayey silt till had liquid limits of between 17 and 39 per cent but typically below 32 per cent, plastic limits of between 11 and 19 per cent, and plasticity indices of between 2 and 17 per cent but typically above 7 per cent, generally indicating low plasticity. The results of the current Atterberg limits determinations are shown on Figure A-1. The results of grain size determinations carried out on samples of the clayey silt till are shown on Figure A-3.



4.1.5 Sandy Silt

Layers of compact to very dense sandy silt were encountered in borehole 13-101 beneath the clayey silt till at about elevation 265.5 metres, and in borehole 13-102 between clayey silt till layers and beneath a layer of silt at about elevations 266.6 and 262.5 metres, respectively. The sandy silt layers in boreholes 13-101 and 13-102 were between about 0.8 and 0.9 metres thick. Sandy silt layers were also encountered in borehole 4 (40I14-111) beneath a silt layer at elevation 261.6 metres, in borehole 5 (40I14-111) beneath the clayey silt till, at about elevation 263.1 metres, and in borehole 2 (40I14-111) beneath a lower layer of silt at about elevation 247.5 metres. The sandy silt layer in borehole 2 (40I14-111) was about 4.4 metres thick. Boreholes 4 and 5 (40I14-111) were terminated in the sandy silt after exploring some 9.3 and 10.8 metres, respectively.

Measured N values from the sandy silt ranged from 13 to greater than 100 blows per 0.3 metres, more typically in the range of 13 to 59 blows per 0.3 metres. Samples of the sandy silt had water contents of between 13 and 21 per cent. The results of grain size determinations carried out on samples of the sandy silt are shown on Figure A-4.

4.1.6 Silt

Layers of very loose to very dense silt were encountered in each of the boreholes. Layers of silt were encountered in borehole 13-101 beneath the sandy silt and clayey silt; in borehole 13-102 beneath the lower layer of clayey silt till, a layer of sandy silt, and the upper layer of clayey silt; in boreholes 13-103 and 13-104 beneath the clayey silt till layers; in borehole 2 (40I14-111) beneath the clayey silt till and upper clayey silt layer; in borehole 3B (40I14-111) from the initial sampling depth, and in borehole 4 (40I14-111) beneath the fill. The silt layers were encountered between elevations 251.7 and 267.2 metres and were between 0.9 and 6.7 metres thick where fully penetrated. Boreholes 3B (40I14-111), 13-103 and 13-104 were terminated in layers of silt after penetrating about 1.1 to 14.9 metres.

Measured N values from the silt ranged from 0 blows per 0.3 metres (sampler advanced under the weight of rods) to greater than 100 blows per 125 millimetres. However, the shallower deposits of silt had N values typically in the range of 13 to 34 blows per 0.3 metres, and the deeper deposits below elevation 254 metres in boreholes 3B (40I14-111), 13-101 and 13-102 had N values of greater than 100 blows per 0.3 metres. Samples of the silt had water contents ranging from 12 to 24 per cent. It was noted that clayey layers and pockets were observed in the silt in boreholes 3B (40I14-111) and 4 (40I14-111). This was supported by the results of an Atterberg limits determination carried out on a sample of silt which yielded a liquid limit of 21 per cent, plastic limit of 14 per cent, and plasticity index of 7 per cent. The results of grain size determinations carried out on samples of the silt are shown on Figure A-5.



4.1.7 Clayey Silt

Layers of stiff to hard clayey silt were encountered in borehole 13-101 beneath the upper layer of silt, in borehole 13-102 beneath a layer of silt and a layer of sand, and in borehole 2 (40114-111) beneath the upper layer of silt and the layer of sandy silt. Where fully penetrated, the clayey silt layers ranged in thickness from 3.1 to 5.6 metres. The upper layers of clayey silt were encountered between about elevations 255.3 and 257.7 metres and the lower layers were encountered at about elevations 246.8 and 243.1 metres. Boreholes 13-102 and 2 (40114-111) were terminated in a layer of clayey silt after penetrating about 0.6 and 14.2 metres, respectively.

Measured N values from the clayey silt ranged from 13 to 84 blows per 0.3 metres, more typically in the range of 13 to 42 blows per 0.3 metres. Samples of the clayey silt had water contents ranging from 11 to 22 per cent. Select samples of clayey silt had liquid limits of between 20 and 30 per cent, plastic limits of between 11 and 18 per cent, and plasticity indices of between 6 and 14 per cent, indicating low plasticity. The results of the current Atterberg limits determinations are shown on Figure A-1. The results of grain size determinations carried out on samples of the clayey silt are shown on Figure A-6.

4.1.8 Sand

Sand was encountered beneath the lower layers of silt in boreholes 13-101 and 13-102 at about elevations 249.0 and 247.3 metres. Borehole 13-101 was terminated in the sand after exploring for about 0.7 metres. The sand in borehole 13-102 was about 0.5 metres thick.

Measured N values from the sand were 13-102 blows per 225 millimetres and 100 blows per 250 millimetres. A sample of the sand had a water content of 20 per cent.

4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. Also, a standpipe groundwater observation well and a piezometer were installed in borehole 13-101, as shown on the Record of Borehole sheets. Groundwater elevations are summarized in the table below.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
POND MILLS ROAD OVERPASS, SITE NUMBER 19-372**

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Elevation (m)	Installation	Measured Groundwater Elevation (m)				
				February 11, 2013	March 8, 2013	April 3, 2013	April 25, 2013	June 5, 2013
13-101	269.7	262.8	Standpipe	266.5	266.6	266.8	266.8	267.0
			Piezometer	Piezometer obstructed	266.5	266.7	266.8	266.6
13-102	270.7	269.5	-	-	-	-	-	-
13-103	275.6	*	-	-	-	-	-	-
13-104	275.1	272.2	-	-	-	-	-	-
1 (40I14-111)	268.3	268.3	-	-	-	-	-	-
2 (40I14-111)	269.0	268.2	-	-	-	-	-	-
3A (40I14-111)	269.0	268.8	-	-	-	-	-	-
3B (40I14-111)	269.0	268.7	-	-	-	-	-	-
4 (40I14-111)	269.5	269.3	-	-	-	-	-	-
5 (40I14-111)	269.5	*	-	-	-	-	-	-
101 (40I14-111)	274.7	268.8	-	-	-	-	-	-
102 (40I14-111)	274.5	273.9	-	-	-	-	-	-

*Groundwater level not established.

The above-noted water levels encountered during drilling are not considered to be representative of the long-term, stabilized groundwater conditions as the readings were taken only during the relatively short duration of drilling. Based on the measured and encountered groundwater levels and the soil colour change from brown to grey, the inferred groundwater level is at elevation 266 metres. Groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.




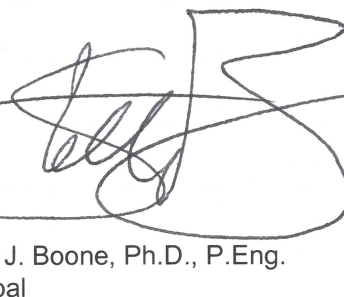
5.0 MISCELLANEOUS


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 and Mr. Michael Arthur 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. This report was reviewed by the Team Leader, Dr. Storer J. Boone, P.Eng. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, carried out an independent review of this report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT

**POND MILLS ROAD OVERPASS REPLACEMENT
SITE NUMBER 19-372**

**HIGHWAY 401 INTERCHANGE IMPROVEMENTS/STRUCTURAL REPLACEMENTS
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 preliminary design of the replacement Pond Mills Road Overpass (Site 19-372). 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 in order 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, and scheduling.

Based on the preliminary design information provided by Dillon, the replacement overpass will be a single span structure supported by integral abutments.

6.2 Existing Structure

For the purposes of this report, Highway 401 and Pond Mills Road are assumed to be oriented in an east-west direction and a north-south direction, respectively. This section of Highway 401 is currently a six lane divided highway oriented generally northeast-southwest. The original Pond Mills Road bridge structure constructed in 1955 is a concrete tee beam structure with a single span of 12 metres. In 1989 the structure was extended by widening the deck only to a total 33.6 metres between barrier walls to accommodate a total of six lanes of Highway 401 traffic. It is understood that the original bridge abutments and retaining walls were founded on 3.7 metre wide spread footings at elevations 267.8 and 267.6 metres for the west and east abutments, respectively.

6.3 Bridge Foundations

The subsurface soil conditions at the site generally consisted of fill and topsoil materials to about elevations 269.4 and 266.6 metres overlying layers of clayey silt till, silt, sandy silt, clayey silt, and sand. The inferred groundwater surface is at about elevation 266 metres.

It is understood that the new structure will be widened to accommodate 8 or 10 lanes of traffic. Consideration is being given to designing the replacement structure with integral abutments.

The suitability of integral or semi-integral abutments is influenced by the length, type, and geometry of the structure, abutment and wingwall heights, number of spans, and the subsurface soil conditions. Based on the preliminary design options being considered, it is understood that the overall length of the structure will be a



single span of less than 100 metres and the structure will be constructed with a minimal skew to the roadway. If integral or semi-integral abutments are to be used, it is recommended that they are supported on driven pile foundations since the near surface fill materials encountered during the investigation are not considered suitable for load bearing. Provided the abutment height and wingwall length are limited to a maximum of 6 and 7 metres, respectively, the use of integral or semi-integral abutments at the site is considered geotechnically feasible.

Preliminary integral abutment designs utilize false abutment walls constructed as reinforced soil system (RSS) walls. Integral abutments constructed with RSS walls may be founded on driven steel H-piles or concrete-filled steel tube piles. Steel H-piles will achieve greater driving depths than steel tube piles; however, compared to steel H-piles loaded in the direction of the weak axis, concrete-filled steel tube piles provide greater resistance to lateral loads and may therefore be less suitable for integral abutments. Both pile types are considered geotechnically feasible.

Conventional abutments may be founded on shallow spread footings bearing on native soils or on new engineered fill materials supported by the native soils, or driven steel H-piles or concrete filled steel tube piles; however, the depth of the existing fill materials and construction staging limitations may preclude full removal of the existing embankment fill and use of shallow foundations for support of conventional abutments. Further, shallow spread footings are not suitable for integral bridge abutments. Therefore, deep foundations in the form of driven piles are the preferred alternative for support of the abutments for this bridge. A comparison of foundation alternatives is presented in Table I, following the text of this report.

6.3.1 Deep Foundations

Geotechnical Axial Resistance

For design of HP 310 x 110 piles and 324 millimetre diameter concrete filled steel tube piles driven to practical refusal at or below the elevations shown, the factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical reaction at Serviceability Limit States (SLS) provided in the following table may be used. The SLS values correspond to an estimated total of 25 millimetres of settlement. The pile driving hammer should be selected such that the energy delivered to the piles is sufficient to achieve the termination criteria (ultimate geotechnical resistances and tip elevations) stated in the Contract Documents.

Pile Type and Location	Assumed Cut-off Elevation (m)	Founding Strata	Maximum Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
HP 310 x 110					
- North Half of Bridge	268	Dense to very dense silt	251	1,200	850
- South Half of Bridge			250	1,300	900
324 mm O.D. x 9.5 mm tube					
- North Half of Bridge	268	Dense to very dense silt	251	1,100	750
- South Half of bridge			251	1,200	800



The above cut-off elevations have been assumed based on the existing elevations at the site, a soil cover of about 1.2 metres for frost protection, and a stick up of 3 metres for integral abutments. Lower cut-off elevations will likely be used if other abutment types are selected resulting in shorter piles. However, the consequent reduction in capacity is expected to be minimal.

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.

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.

It should be noted that the existing embankment fill is known to contain cobbles which may interfere with advancement of the piles or cause damage to pile tips. Appropriate pile shoes should be used to prevent damage to the piles.

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.

Downdrag Loads

Considering that grade increases are not currently proposed, the character of the embankment fill, and the very stiff to hard or compact to dense native soils, negative skin friction should not develop on the new piles at both abutments. If, however, significant grade raises might be proposed for the site, the potential for downdrag loads must be reconsidered.

Frost Protection

Pile caps should be provided with a frost cover of 1.2 metres of soil or thermal equivalent.

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 stratigraphy presented in the table below has been simplified for the purposes of this report.

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



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
POND MILLS ROAD OVERPASS, SITE NUMBER 19-372

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

where:

- 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 grade (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 non-integral abutments or pier foundations.

Location	Soil Type	Elevation (m)	n_h (MPa/m)	S_u (kPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 - 10	-
West Abutment, North End	Existing Fill – Firm to hard clayey silt	Above 269	-	100
	Very stiff to hard clayey silt till	265 to 269	-	215
	Very loose to compact silt to sandy silt	258 to 265	7 - 20	-
	Stiff to very stiff clayey silt	252 to 258	-	170
	Very dense silt to sand	Below 252	37 - 38	-
West Abutment, South End	Existing Fill - Loose to very dense sand	Above 268	5 - 55	-
	Very stiff to hard clayey silt till	258 to 268	-	375
	Dense silt	255 to 258	10 - 15	-
	Hard clayey silt	252 to 255	-	330
	Very dense silt to sandy silt	243 to 252	17 - 35	-
East Abutment, North End	Existing Fill - Firm to hard clayey silt	Above 269	-	170
	Very stiff to hard clayey silt till	266 to 269	-	375
	Compact to very dense silt, sandy silt, to silty sand	251 to 266	6 - 22	-
East Abutment, South End	Existing Fill - Firm to very stiff clayey silt, loose to compact silt, sand and gravel and silty sand	Above 269	1 - 6	120
	Stiff to hard clayey silt till	265 to 269	-	310
	Compact to dense silt, sandy silt to silty sand	257 to 265	6 - 17	-
	Very stiff to hard clayey silt	252 to 257	-	275
	Very dense silt to sand	Below 252	28 - 55	-



The lateral resistances for the various foundation options are summarized in the following table.

Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
Integral abutments		
- HP 310 x 110, weak axis bending	30	*
- 324 mm O.D. x 9.5 mm tube	50	*
Semi-Integral or Conventional abutments		
- HP 310 x 110, strong axis bending	230	125
- 324 mm O.D. x 9.5 mm tube	170	100

* Load to mobilize 10mm horizontal displacement is greater than ULS value, therefore ULS value governs.

The lateral resistances are based on Brom's Method from "Design and Construction of Driven Pile Foundations Workshop Manual – Volume 1, FHWA, Pub. No. FHWA H1 97-013, Revised November 1998". Free-headed piles were assumed, with the horizontal load for semi-integral or conventional abutments applied at the ground surface (i.e., underside of abutment footing) and for integral abutments at 3 metres above the ground surface (underside of abutment). An ultimate compressive strength of 32 megapascals was assumed for the concrete within the steel tube piles. The SLS values are based on 10 millimetres of horizontal 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 and factored resistance values 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



6.5 Liquefaction Potential and Seismic Analysis

6.5.1 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) is 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.

6.5.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.⁴ Although saturated granular materials are present, they were found to have a normalized N value of greater than 22 blows per 0.3 metres. The liquefaction potential is considered to be relatively low based on the soil profile type, age of the deposits, relative density and the historically low regional seismicity. 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.

6.6 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wing walls 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 or III but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with SP105S10. 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

⁴ 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.



to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (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 OPSS.PROV 501.
- 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 1 horizontal to 1 vertical slope extending up and back from the rear face of the footing (Case b from Commentary on CHBDC Figure C6.20).
- For Case a, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM).

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
At rest, K_o	0.50

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

	GRANULAR A	GRANULAR B	
		Type II	Type III
Soil unit weight:	22 kN/m ³	21 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:			
Active, K_a	0.27	0.27	0.31
At rest, K_p	3.7	3.7	3.3

- 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 at the back of the wall.



6.7 Embankments

It is understood that consideration is being given to widening the existing embankments up to about 7 metres on each side. All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. Prior to placement of embankment fill material the exposed subgrade should be proofrolled under the direction of a geotechnical QVE. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 metres where pavement base and subbase materials will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments and adequately compacted. Embankments shall be constructed in accordance with OPSS 206 and OPSD 208.010. The settlement resulting from the approaches' widenings is expected to be negligible.

Embankments constructed with SSM and founded on the near surface stiff clayey silt till are expected to be stable and may be designed using a Factor of Safety against slope failure of 1.3 for embankments no steeper than 2 vertical to 1 horizontal. Embankments greater than 8 metres in height should be constructed with a 2 metre wide bench at the mid-height.

6.8 Excavations and Temporary Cut Slopes

6.8.1 General

Excavations for pile caps and/or abutments will penetrate the existing fill and organic materials into the clayey silt till. The groundwater level is expected to be at about elevation 266 metres and will fluctuate seasonally and due to climatic variations. The excavations may extend below the groundwater level; however, seepage volumes from the clayey silt till are expected to be low. If necessary, groundwater control may be achieved by pumping from properly 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.

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 topsoil and 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 1 or Type 2 depending on consistency or relative density.



6.8.2 Temporary Roadway Protection

Temporary road 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. These systems are to be designed by and the limits 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 of the systems could be in the form of struts and walers in the case of pile cap/abutment excavations or rakers and anchors. 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. The lateral movement of the temporary support system should meet Performance Level 2 as specified in SP105S19.



7.0 RECOMMENDATIONS FOR DETAIL DESIGN

Integral abutments are being considered for design of the replacement bridge structure. Deep foundations are considered the preferred foundation alternative. A Foundation Investigation and Design Report will need to be prepared during a future assignment to provide appropriate information for future 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 boreholes 101 and 102(40I14-111). The boreholes should be advanced to 3 metres below refusal, defined as material for which the SPT N value is greater than 100 blows per 0.3 metres.

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

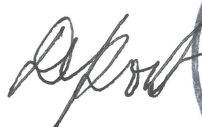
The recommendations given in this Preliminary Foundation Design Report should be expanded upon and updated in the Foundation Design Report for detail design in accordance with MTO's standard requirements for foundation engineering assignments. Detailed recommendations should be provided for foundations for the abutments and wingwalls. If the vertical alignment of the highway or the approach embankments is to be significantly altered, embankment stability and settlement should be evaluated. Also, 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 construction design.

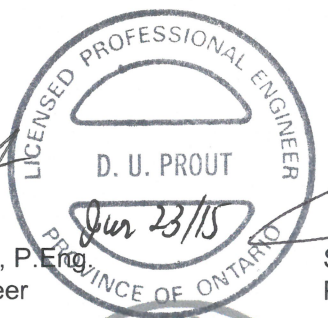


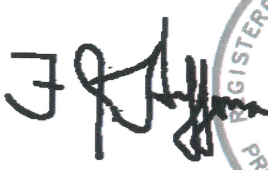
8.0 MISCELLANEOUS


This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by the Team Leader, Dr. Storer J. Boone, P. Eng. Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, carried out an independent review of this report.

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NG/DUP/SJB/FJH/cr

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

Pond Mills Road Overpass, Site 19-372
 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 new engineered fill	<ul style="list-style-type: none"> • Not feasible due to depth of existing fill. • 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-pile or steel tube pile foundations driven to refusal into very dense 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. • Can be damaged and deflected by cobbles and boulders within fill and glacial till deposits. • More construction noise and vibration compared to shallow foundations or caissons. • Cannot be visually inspected at depth. • Integrity inspection requires specialty dynamic testing. 	<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. • Variation in pile tip elevations.

- 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: DUP

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.)

(b) Cohesive Soils

Consistency

	c_u, s_u	
	kPa	psf
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

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)

RECORD OF BOREHOLE No 13-101

1 OF 2

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00 LOCATION N 4755728.8 , E 411548.9 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF/LMK
DATUM GEODETIC DATE February 6 - 7, 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 (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
269.72	GROUND SURFACE							20	40	60	80	100					
0.00	TOPSOIL, silty Brown																
0.30	FILL, silty sand, trace topsoil Brown																
268.96																	
0.76	CLAYEY SILT TILL, trace sand, trace gravel Very stiff Brown		1	SS	18												
			2	SS	21												
			3	SS	25												
			4	SS	23												
			5	SS	15												
265.45																	
4.27	SANDY SILT, trace clay Compact to dense Grey		6	SS	31												
			7	SS	28												
264.33																	
5.39	SILT, some sand, trace clay, with clayey silt layers Compact to dense Grey		8	SS	31												
			9	SS	20												
262.80																	
6.92	SILT, trace sand, some clay Very loose to compact Grey		10	SS	19												
			11	SS	WR												
259.97																	
9.75	SILT, trace clay, with clayey silt layers Dense Grey		12	SS	30												
257.65																	
12.07	CLAYEY SILT, with silt seams and layers Stiff to very stiff Grey		13	SS	19												
			14	SS	25												

Continued Next Page

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

PROJECT 12-1132-0076

W.P. 3030-11-00

LOCATION N 4755728.8 . E 411548.9

ORIGINATED BY BT

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY WDF/LMK

DATUM GEODETIC

DATE February 6 - 7, 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 (%)	
								○ UNCONFINED	+ FIELD VANE							
								● QUICK TRIAXIAL	× LAB VANE							
								20 40 60 80 100	20 40 60 80 100						10 20 30	
								20 40 60 80 100	20 40 60 80 100							
	CLAYEY SILT, with silt seams and layers Stiff to very stiff Grey		15	SS	16		254									
			16	SS	13		253									
252.04							252									
17.68	SILT, trace clay, trace to some sand Very dense Grey		17	SS	101/ 200mm		251									
			18	SS	100		250									
248.99							249									
20.73	SAND, fine, trace silt, with silt layers Very dense Grey		19	SS	102/ 225mm											
248.29																
21.43	END OF BOREHOLE															
	Groundwater encountered at about elev. 262.8m during drilling on February 6, 2013.															
	Water level measured in standpipe at elev. 266.52m on February 11, 2013.															
	Water level measured in standpipe at elev. 266.62m on March 8, 2013.															
	Water level measured in piezometer at elev. 266.52m on March 8, 2013.															
	Water level measured in standpipe at elev. 266.77m on April 3, 2013.															
	Water level measured in piezometer at elev. 266.72m on April 3, 2013.															
	Water level measured in standpipe at elev. 267.05m on June 5, 2013.															
	Water level measured in piezometer at elev. 266.62m on June 5, 2013.															

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

RECORD OF BOREHOLE No 13-102

1 OF 2

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00 LOCATION N 4755673.4 , E 411594.4 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF/LMK
DATUM GEODETIC DATE February 14 - 19, 2013 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							W _P W W _L			
270.72	GROUND SURFACE							20	40	60	80	100						
0.00	TOPSOIL, silty, trace gravel Brown							20	40	60	80	100						
270.26																		
0.46	FILL, silt, some sand, trace topsoil Compact Brown		1	SS	15		270											
269.50																		
1.22	FILL, clayey silt, trace to some sand, trace gravel, with cobbles Loose to compact Brown		2	SS	8		269											
			3	SS	18		268											
			4	SS	25		267											
267.37																		
3.35	CLAYEY SILT TILL, trace sand, trace gravel Very stiff Brown		5	SS	19		266											
266.61																		
4.11	SANDY SILT, Compact Brown becoming grey at about elev. 266.1m		6	SS	20		265											
265.84																		
4.88	CLAYEY SILT TILL, trace sand, trace gravel Stiff Grey		7	SS	14		264											
265.08																		
5.64	SILT, trace to some sand, trace clay, with clayey silt layers Compact Grey		8	SS	25		263											
			9	SS	13		262											
262.49																		
8.23	SANDY SILT Dense Grey		10	SS	31		261											
261.55																		
9.17	SILT, trace sand, trace clay Dense Grey						260											
260.81																		
9.91	SILT, some sand, with silt layers Compact Grey		11	SS	23		259											
			12	SS	21		258											
			13	SS	27		257											
256.39							256											
14.33	CLAYEY SILT, trace sand, with silt layers Very stiff to hard Grey																	

LDN_MTO_06 12-1132-0076-1001.GPJ LDN_MTO.GDT 22/05/15

Continued Next Page

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

PROJECT <u>12-1132-0076</u>		RECORD OF BOREHOLE No 13-102		2 OF 2	METRIC
W.P. <u>3030-11-00</u>	LOCATION <u>N 4755673.4 , E 411594.4</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 14 - 19, 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/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	W _p	W	W _L		
	CLAYEY SILT, trace sand, with silt layers Very stiff to hard Grey		14	SS	42												
						255											
			15	SS	26	254										0 1 51 48	
						253											
			16	SS	20											0 1 48 51	
						252											
251.67																	
19.05	SILT, trace to some sand, trace gravel Very dense Grey		17	SS	132	251											
			18	SS	100/ 225mm	250											
			19	SS	100/ 125mm											0 12 77 11	
			20	SS	100/ 275mm	249											
			21	SS	103	248											
247.31																	
23.41	SAND, fine, some silt Very dense Grey		22	SS	100/ 250mm	247											
246.79																	
23.93	CLAYEY SILT, trace sand, trace gravel Very stiff Grey		23	SS	20												
246.18																	
24.54	END OF BOREHOLE																
	Groundwater encountered at about elev. 269.5m during drilling on February 14, 2013.																

LDN_MTO_06 12-1132-0076-1001.GPJ LDN_MTO.GDT 22/05/15

RECORD OF BOREHOLE No 13-103

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00 LOCATION N 4755703.8 , E 411539.8 ORIGINATED BY MA
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY AMG/LMK
DATUM GEODETIC DATE May 14, 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20	40	60	80	100					
								○ UNCONFINED		+	FIELD VANE						
								● QUICK TRIAXIAL		×	LAB VANE						
								20	40	60	80	100					
275.58	PAVEMENT SURFACE																
0.00	ASPHALT																
0.12	FILL, sand and gravel, trace silt, crushed																
0.30	Brown																
274.60	FILL, sand and gravel, trace silt, cobbles		1	SS	12		275										
0.98	Compact Brown																
	FILL, clayey silt, trace to some sand, trace gravel, trace to some topsoil, with silt seams and layers		2	SS	65		274										
	Firm to hard Brown																
			3	SS	7		273									0 25 45 30	
			4	SS	6		272										
			5	SS	5		271									0 17 54 29	
			6	SS	12		270										
270.00			7	SS	18		269										
5.58	FILL, topsoil, silty, trace sand, trace gravel		8	SS	20		268									7 11 44 38	
5.79	Very stiff Black		9	SS	28		267										
269.27	FILL, clayey silt, some sand, some topsoil, trace gravel		10	SS	43		266										
6.31	Very stiff Brown		11	SS	28		265									20 11 37 32	
6.55	FILL, topsoil, silty, some sand		12	SS	19		264										
	Very stiff Black		13	SS	17		263										
	CLAYEY SILT TILL, trace to some sand, trace to some gravel																
	Very stiff to hard Brown becoming grey at about elev. 266.4m																
264.00																	
11.58	SILT, trace clay																
	Compact Grey																
262.93	END OF BOREHOLE																
12.65	Borehole dry during drilling on May 14, 2013.																

RECORD OF BOREHOLE No 13-104

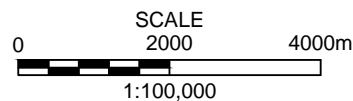
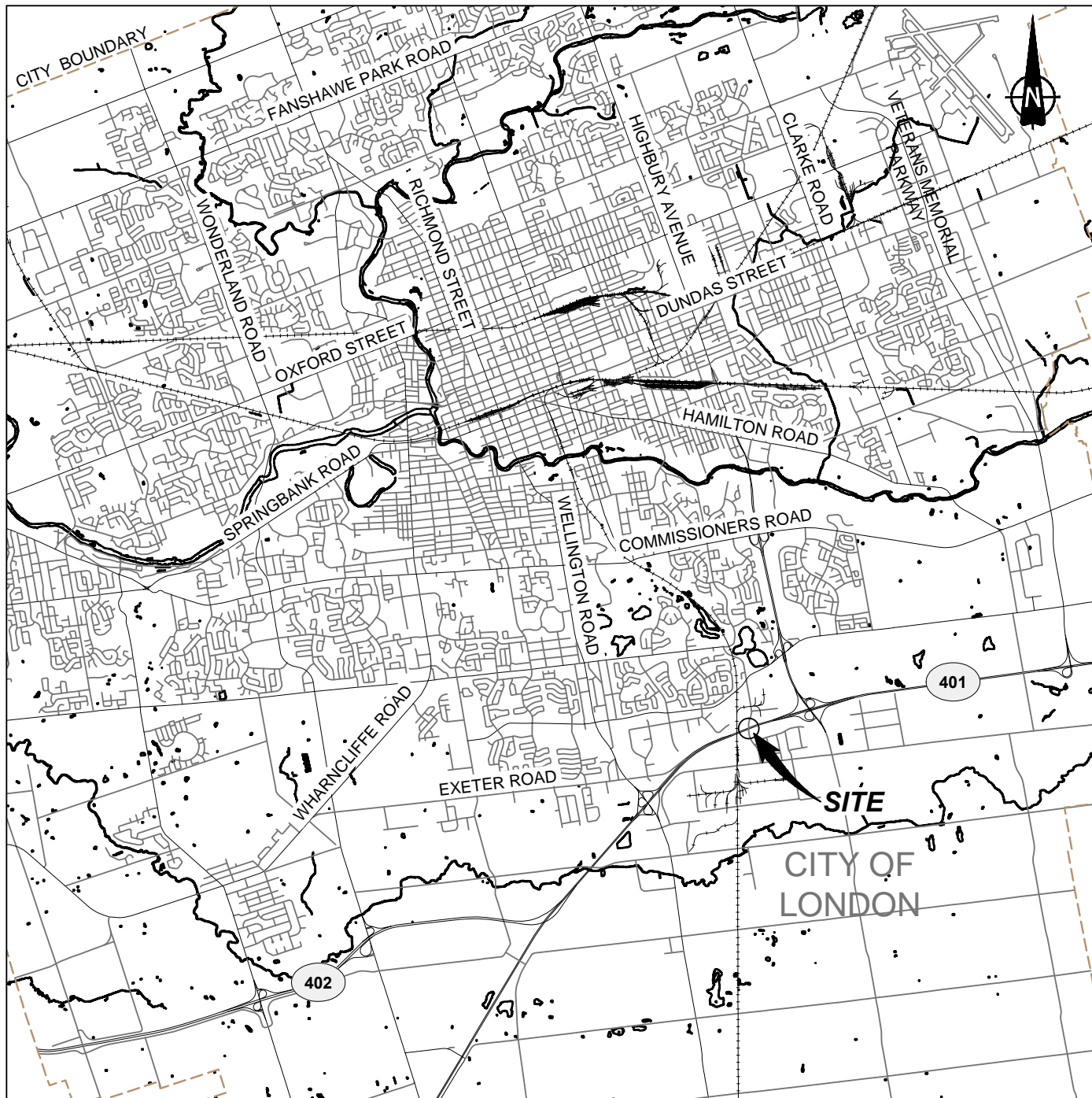
1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00 LOCATION N 4755694.3 , E 411594.5 ORIGINATED BY MA
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY AMG/LMK
DATUM GEODETIC DATE May 15, 2013 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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	W _P W W _L	10 20 30	GR SA SI CL		
275.10	PAVEMENT SURFACE					▽	275							
0.11	ASPHALT													
0.34	FILL, sand and gravel, trace silt, crushed Brown													
274.09	FILL, sand and gravel, trace silt, cobbles Compact Brown		1	SS	14									
273.73	FILL, clayey silt, trace sand, trace gravel, trace topsoil Stiff Brown		2	SS	5									
273.18	FILL, sandy clayey silt, some topsoil Firm Brown		3	SS	6									
272.20	FILL, silty fine sand Loose Brown		4	SS	4									
271.08	FILL, clayey silt, with silt and sand seams and layers Firm Brown		5	SS	7									
269.92	FILL, clayey silt, trace sand, trace topsoil Firm to very stiff Brown		6	SS	23									
267.18	CLAYEY SILT TILL, some sand, trace gravel Very stiff to hard Brown becoming grey at about elev. 268.1m		7	SS	30									
266.26	SILT, trace clay, with clayey silt layers Compact Grey													
265.04	CLAYEY SILT TILL, some sand, trace gravel Very stiff Grey		10	SS	29									
263.97	SILT, with clayey silt layers Dense Grey		11	SS	34									
11.13	END OF BOREHOLE						264							
	Groundwater encountered at about elev. 272.2m during drilling on May 15, 2013.													

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



REFERENCE

PLAN BASED ON CITY OF LONDON CITY CD v.2011.

NOTE

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

ALL LOCATIONS ARE APPROXIMATE.

PROJECT
**POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS**
GWP 3054-11-00

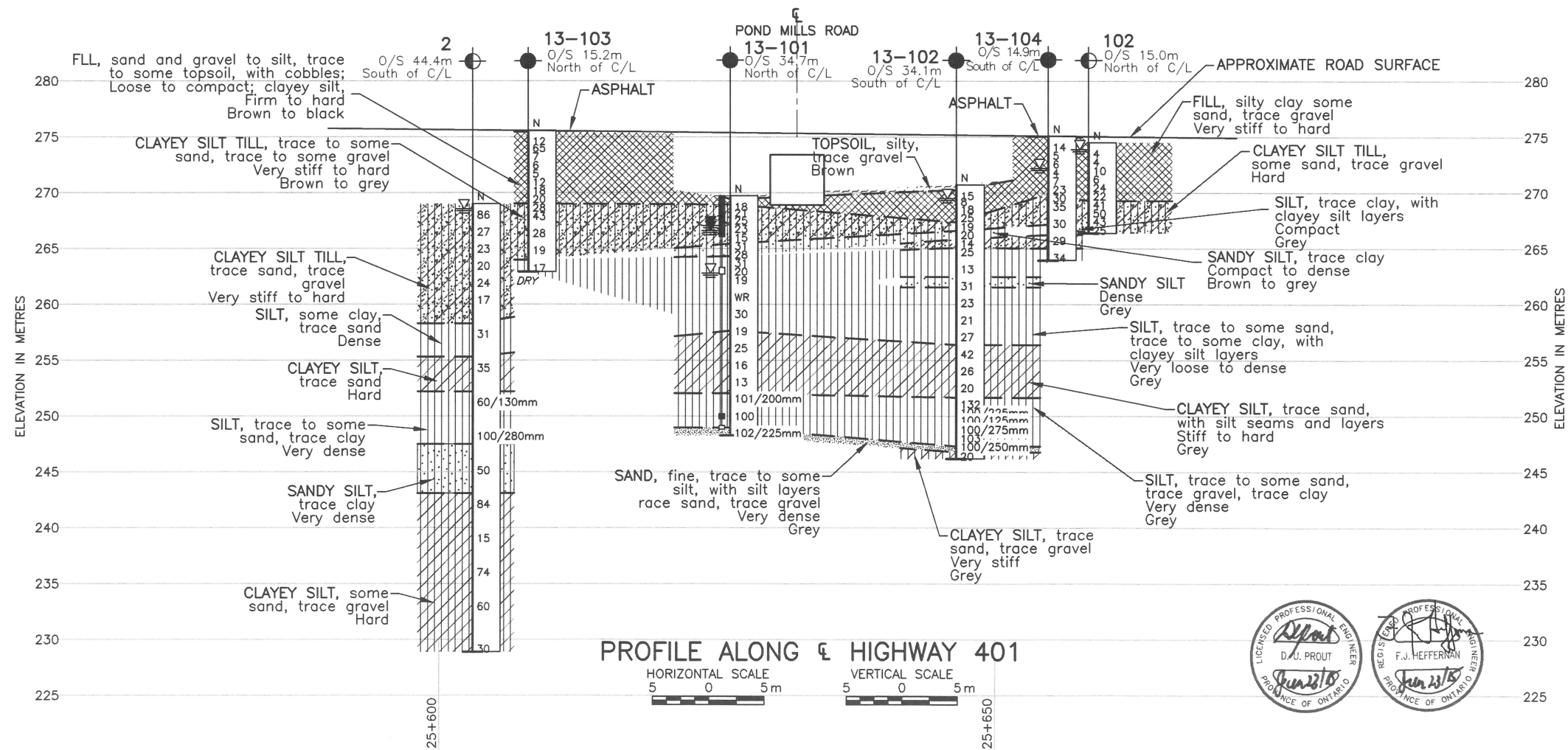
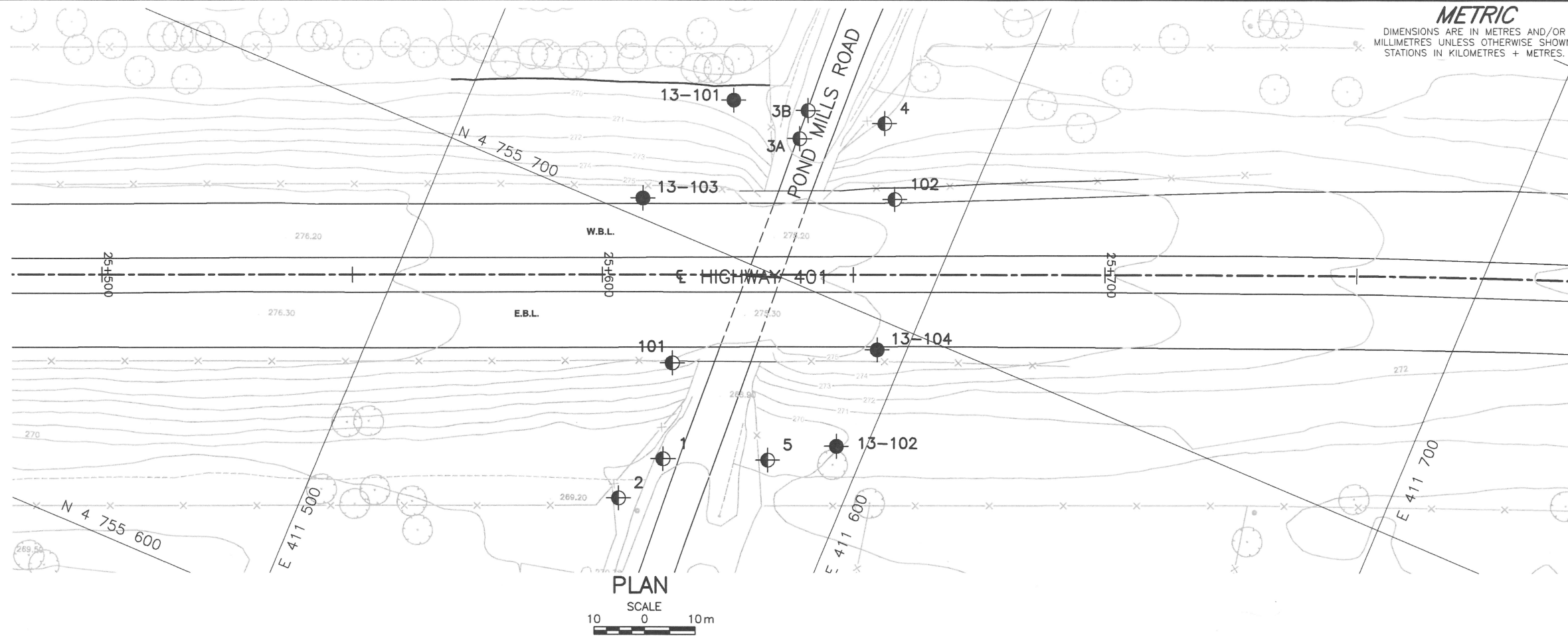
TITLE

KEY PLAN



PROJECT No.	12-1132-0076	FILE No.	1211320076-1001-F04001
CADD	AMG/LMK	June 7/13	SCALE AS SHOWN REV. 0
CHECK			

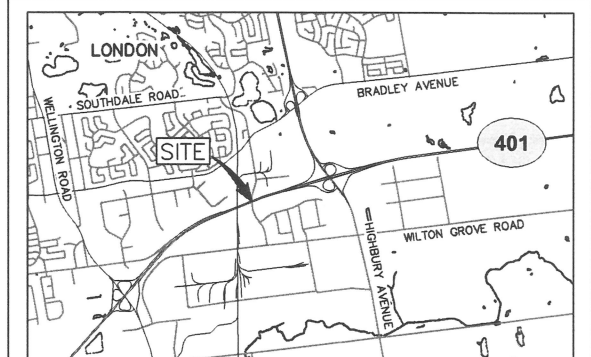
FIGURE 1

CONT No.
WP No. 3054-11-00POND MILLS ROAD OVERPASS
HIGHWAY 401 IMPROVEMENTS

BOREHOLE LOCATIONS AND SOIL STRATA



SHEET

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

KEY PLAN

SCALE IN KILOMETRES
0 1 2

LEGEND

- Borehole - Current Investigation
- Borehole (Geocres 40114-111)
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on June 5, 2013
- WL encountered during drilling
- DRY Water level not established

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
13-101	269.72	4 755 728.8	411 548.9
13-102	270.72	4 755 673.4	411 594.4
13-103	275.58	4 755 703.8	411 539.8
13-104	275.10	4 755 694.3	411 594.5
Geocres 40114-111			
1	268.3	4 755 657.6	411 563.5
2	269.0	4 755 647.0	411 558.4
3A	269.0	4 755 726.9	411 564.0
3B	269.0	4 755 732.7	411 563.4
4	269.5	4 755 736.3	411 578.5
5	269.5	4 755 665.5	411 582.8
101	274.7	4 755 675.9	411 557.8
102	274.5	4 755 723.2	411 586.2

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)

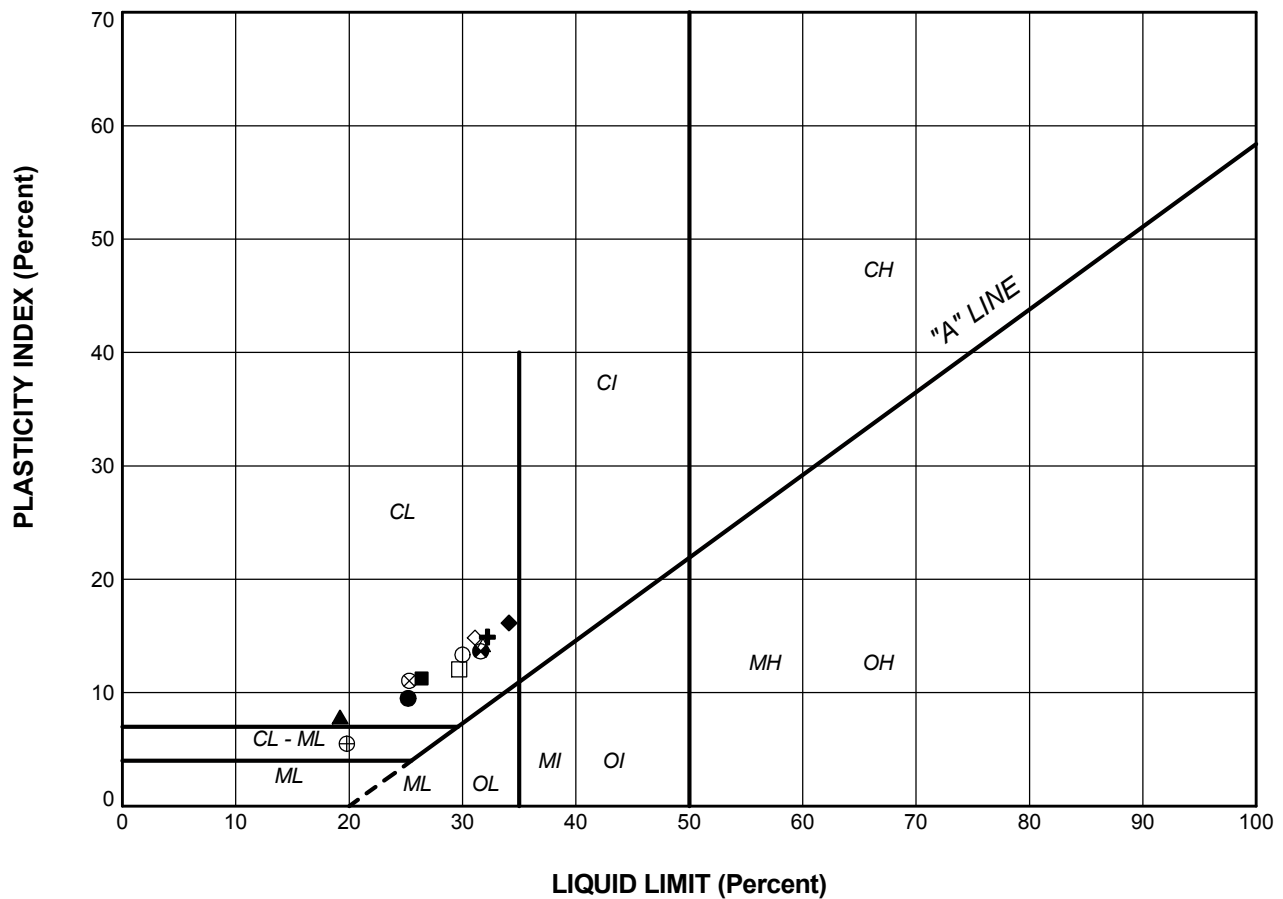


NO.	DATE	BY	REVISION
Geocres No. 40114-157			
HWY.	401	PROJECT NO.	12-1132-0076
SUBM'D.	NG	CHKD.	NAG
DRAWN:	WDF/LMK	CHKD.	DUP
DATE:	June 12/13	APPD.	FJH
SITE:	19-372	DWG.	1



APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
FILL, clayey silt					
●	13-103	3	25.2	15.7	9.5
■	13-103	6	26.4	15.2	11.3
▲	13-104	3	19.2	11.4	7.9
CLAYEY SILT TILL					
+	13-101	4	32.2	17.3	14.9
◆	13-102	5	34.1	18.0	16.2
◇	13-103	10	31.1	16.3	14.9
○	13-103	12	30.0	16.7	13.4
△	13-104	8	31.8	17.6	14.2
⊗	13-104	10	25.3	14.3	11.1
CLAYEY SILT					
⊕	13-101	14	19.8	14.3	5.5
□	13-102	15	29.7	17.7	12.1
⊙	13-102	16	31.6	18.0	13.7

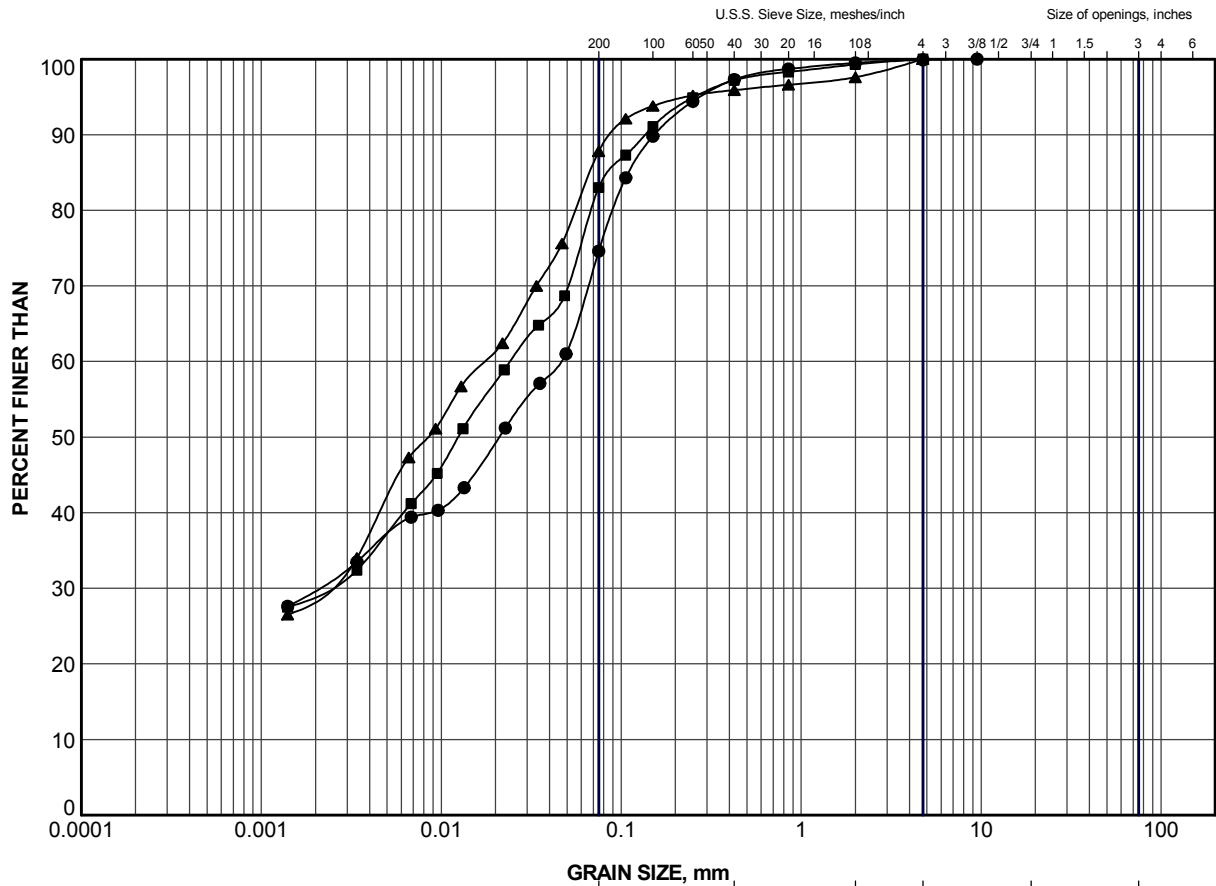
PROJECT **POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00**

TITLE

PLASTICITY CHART




PROJECT No12-1132-0076-1001			FILE No. 1211320076-1001-F040A1		
DRAWN	LMK	Jun 12/13	SCALE	N/A	REV.
CHECK			FIGURE A-1		

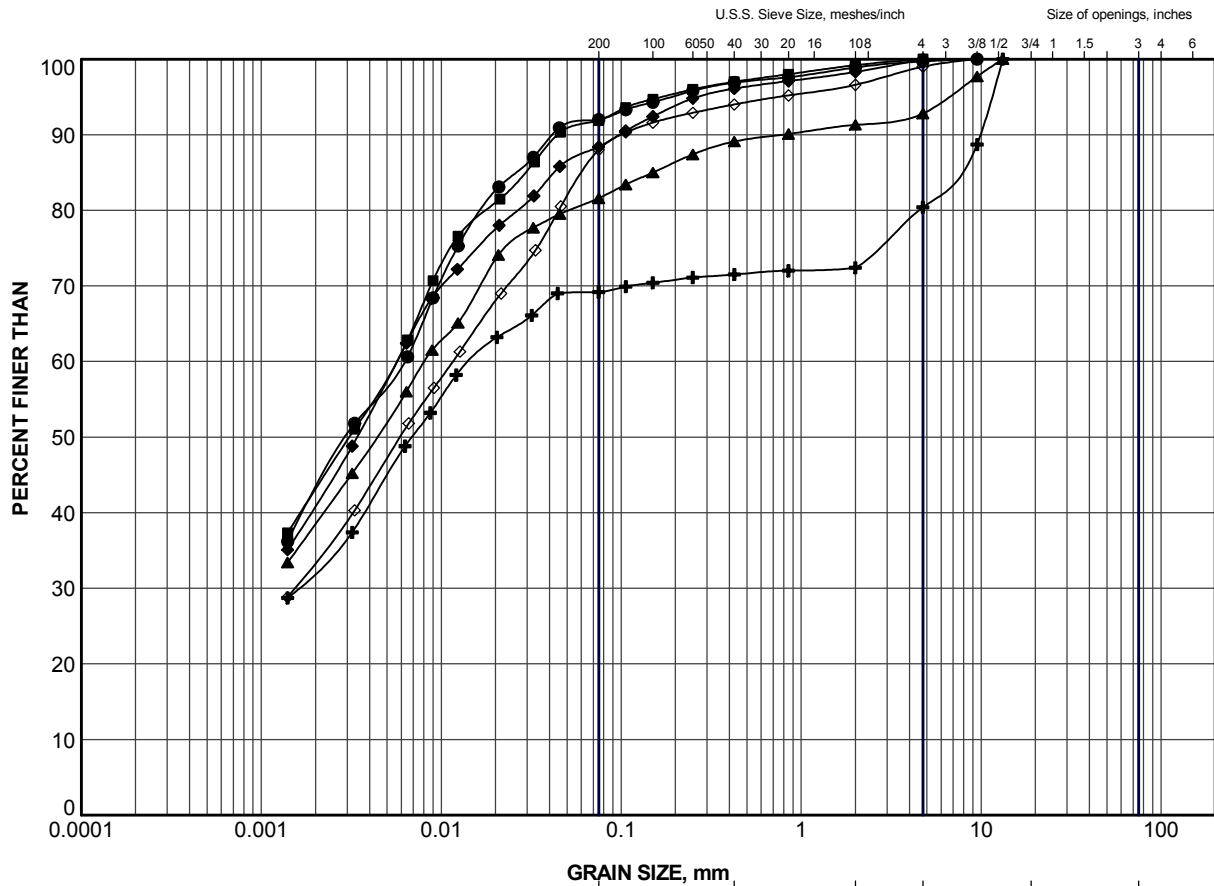


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	13-103	3	273.2
■	13-103	6	270.9
▲	13-104	3	272.6

PROJECT				POND MILLS ROAD OVERPASS REPLACEMENT HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3054-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT FILL			
PROJECT No.12-1132-0076-1001				FILE No. 1211320076-1001-F040A2			
DRAWN LMK Jun 12/13				SCALE N/A REV.			
CHECK				FIGURE A-2			
 Golder Associates LONDON, ONTARIO							



LEGEND

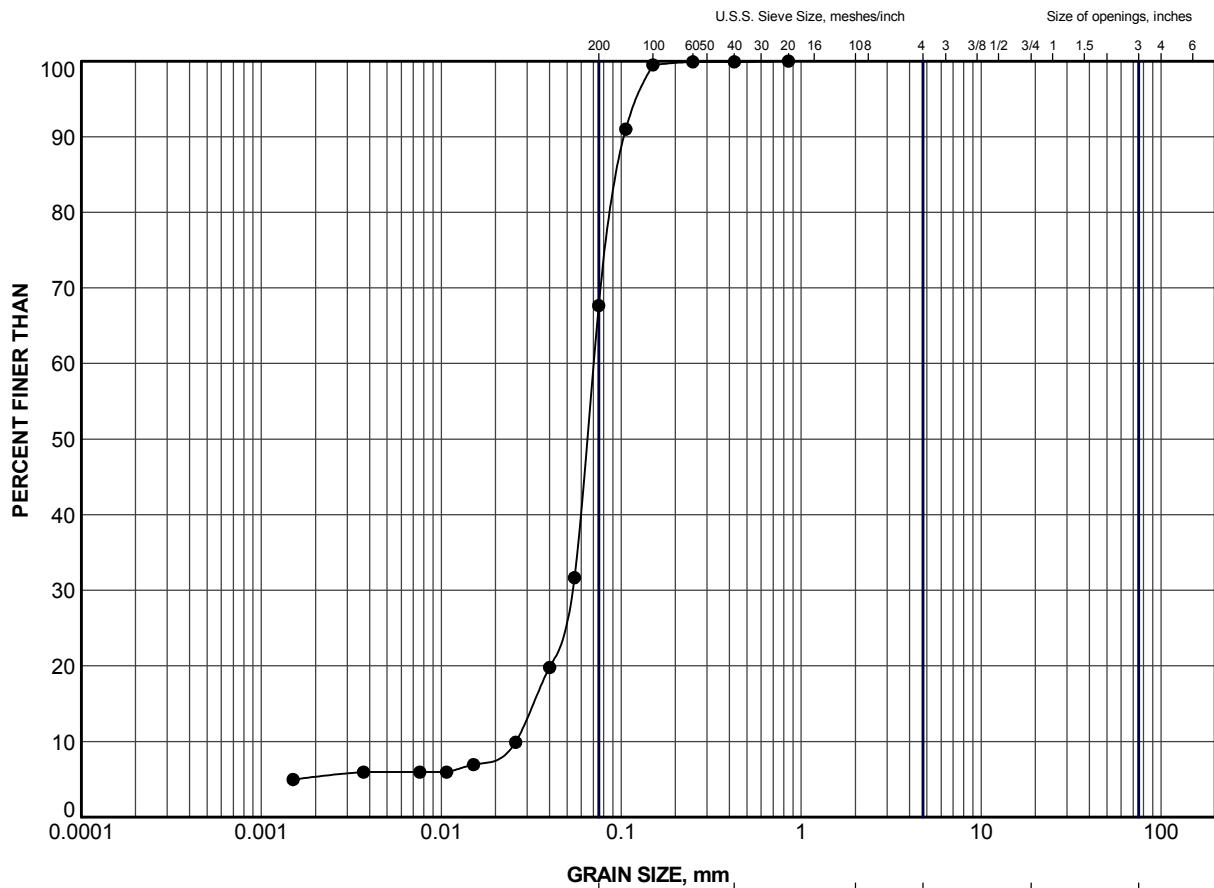
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	13-101	4	266.8
■	13-102	5	267.0
▲	13-103	10	267.9
+	13-103	12	264.8
◆	13-104	8	268.8
◇	13-104	10	265.7

PROJECT
POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE
**GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL**



PROJECT No.12-1132-0076-1001	FILE No. 1211320076-1001-F040A3
DRAWN LMK Jun 12/13	SCALE N/A REV.
CHECK	FIGURE A-3

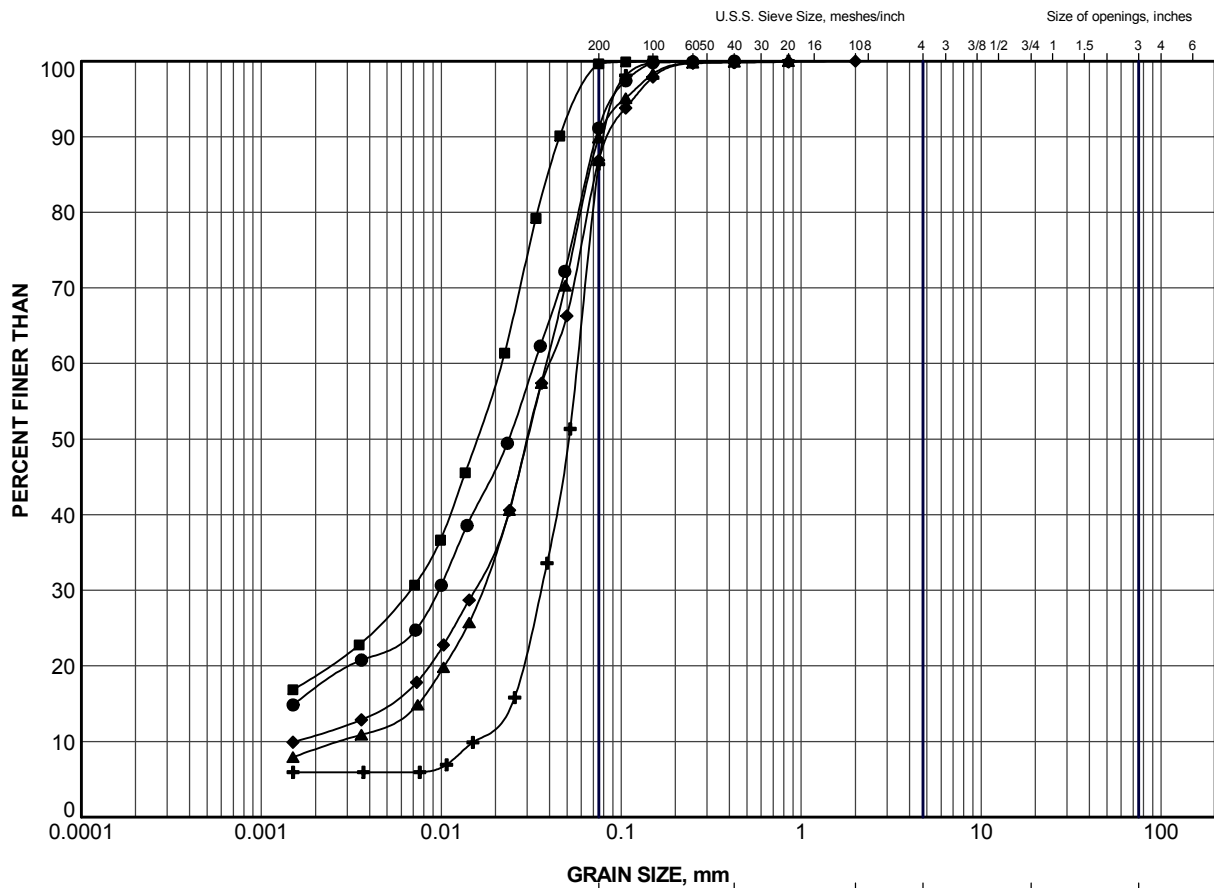


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	13-102	10	261.7

PROJECT	POND MILLS ROAD OVERPASS REPLACEMENT HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3054-11-00		
TITLE	GRAIN SIZE DISTRIBUTION SANDY SILT		
	PROJECT No:12-1132-0076-1001		FILE No. 1211320076-1001-F040A4
	DRAWN	LMK	Jun 12/13
	CHECK		
SCALE N/A			REV.
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)
●	13-101	10	262.2
■	13-101	12	259.1
▲	13-101	17	251.5
+	13-102	12	258.6
◆	13-102	19	249.6

PROJECT POND MILLS ROAD OVERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

GRAIN SIZE DISTRIBUTION SILT



Golder Associates
LONDON, ONTARIO

PROJECT No12-1132-0076-1001			FILE No. 1211320076-1001-F040A5		
DRAWN	LMK	Jun 07/13	SCALE	N/A	REV.
CHECK			FIGURE A-5		



APPENDIX B

Site Photographs



APPENDIX B SITE PHOTOGRAPHS



Photograph 1: Pond Mills Overpass, looking north.



Photograph 2: Pond Mills Overpass, looking south.

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\rvpts\rv04 pond mills site 19-372\1211320076-1001-r04 jun 15
15 (final) app b-site photos.docx



APPENDIX C

Record of Boreholes - Geocres Report No. 40I14-111



RECORD OF BOREHOLE No 1

METRIC

W P 139-86-02 LOCATION Co-ords. N 4 755 438.0; E 411 554.3 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (H.S.) COMPILED BY PP
DATUM Geodetic DATE 86 11 11 CHECKED BY [Signature]

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			SHEAR STRENGTH						
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE					
268.3	Ground Level													
0.0							268							
	Silty Clay traces of sand traces of organics Very Stiff to Hard		1	SS	42									
			2	SS	45									
			3	SS	45									
			4	SS	29									
			5	SS	27									
264.0							266							
4.3	End of Borehole													

+3, x5: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 2

METRIC

W P 139-86-02 LOCATION Co-ords: N 4 755 427.4; E 411 549.2 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (H.S.) & Washbore - BW Casing COMPILED BY PP
DATUM Geodetic DATE 86 11 11 - 86 11 13 CHECKED BY ST

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						
								SHEAR STRENGTH								WATER CONTENT (%)							
								○ UNCONFINED								+ FIELD VANE							
269.0	Ground Level														GR SA SI CL								
0.0																							
	Silty Clay traces of sand traces of gravel		1	SS	86																		
			2	SS	27										0 7 (93)								
			3	SS	23																		
			4	SS	20																		
	Very Stiff to Hard		5	SS	24																		
			6	SS	17										0 5 (95)								
258.3																							
10.7	Silt traces of sand some clay		7	SS	31										0 5 80 15								
255.3	Dense																						
13.7	Silty Clay traces of sand		8	SS	35																		
	Hard																						
252.2																							
16.8	Silt trace/some sand trace clay		9	SS	60	13 cm																	
	Very Dense																						
247.5			10	SS	100	28 cm									0 13 76 11								
21.5	Sandy Silt traces of clay																						
	Very Dense		11	SS	50																		
243.1																							
25.9	Silty Clay		12	SS	84										2 20 (78)								



RECORD OF BOREHOLE No 2 (Cont'd)

METRIC

W P 139-86-02 LOCATION Co-ords. N 4 755 427.4; E 411 549.2 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (H.S.) & Washbore - BW Casing COMPILED BY PP
DATUM Geodetic DATE 86 11 11 - 86 11 13 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES							
238.8	Continued		13	SS	15		238					
	Some Sand						236					
	traces of gravel		14	SS	74		234					
	Hard		15	SS	60		232					
228.9			16	SS	30		230					
40.1	End of Borehole											4 16 (80)

+³, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3A

METRIC

W P 139-86-02 LOCATION Co-ords: N 4 755 507.3; E 411 554.8 ORIGINATED BY DC

DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (S.S.) COMPILED BY DC

DATUM Geodetic DATE 86 12 03 CHECKED BY

[illegible]

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 3B

METRIC

W P 139-86-02 LOCATION Co-ords. N4 755 513.1; E 411 554.2
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (S.S.)
DATUM Geodetic DATE 86 12 04
ORIGINATED BY DC
COMPILED BY PP
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	Probably Silty Clay						268										
265.3							266										
3.7	Silt Occasional Silty Clay Pockets traces of sand Very Dense		5	SS	68		264										0 3 83 14
262.0			6	SS	55												
7.0			7	SS	84		262										0 4 79 17
			8	SS	17		260										9 10 76 5
	Sandy Silt traces of clay traces of gravel Compact to Very Dense		9	SS	31		258										
			10	SS	49		256										0 16 79 5
			11	SS	100	28 cm	254										
							252										
250.4			12	SS	60	15 cm											
18.6	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 4

METRIC

W P 139-86-02 LOCATION Co-ords. N 4 755 516.7; E 411 569.3 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (S.S.) COMPILED BY DC
DATUM Geodetic DATE 86 12 05 CHECKED BY DC

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							
								SHEAR STRENGTH							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL x LAB VANE							
									WATER CONTENT (%)						
									10	20	30				
269.5	Ground Level														
0.0	Silty Clay traces of organics traces of gravel some (Fill Material) sand Very Stiff to Hard		1	SS	15		268							4 21 (75)	
			2	SS	40										
			3	SS	30										
265.8			4	SS	25		266							1 11 (88)	
3.7	Silt traces of gravel traces of sand Occasional Silty Clay Seams Compact to Very Dense		5	SS	45										
			6	SS	61									8 4 79 9	
			7	SS	24		264								
			8	SS	30		262							0 2 (98)	
261.6			9	SS	13		260								
7.9	Sandy Silt to Silty Sand traces of clay Compact to Very Dense		10	SS	71		258								
			11	SS	49		256								
			12	SS	59		254								
250.8							252								
18.7	End of Borehole														

+³, x⁵: Numbers refer to
Sensitivity

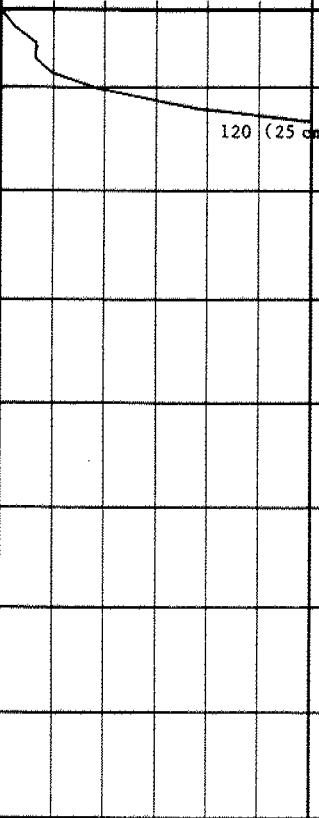
20
15-5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 5

METRIC

W P 139-86-02 LOCATION Co-ords. N 4 755 445.9; E 411 573.6 ORIGINATED BY DC
DIST 2 HWY 401 BOREHOLE TYPE Cont. Flight Auger (S.S.) COMPILED BY PP
DATUM Geodetic DATE 86 12 08 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	SHEAR STRENGTH							WATER CONTENT (%)																													
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE																																				
269.5	Ground Level										10 20 30			GR SA SI CL																															
0.0	Silty Clay trace/some sand traces of gravel		1	SS	32	*	268							2 38 (60)																															
2			SS	40																																									
3			SS	41																																									
4			SS	25																																									
5			SS	54																																									
263.1	Hard		6	SS	55										266							0 2 (98)																							
6.4			Sandy Silt to Silty Sand traces of clay		7																		SS	19	264																				
	Compact to Dense																																												
																																						3	SS	50	262				
						9	SS	45	260																																				
253.8	End of Borehole																																				7 38 (55)								
15.7															* Groundwater Level not observed																														

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (% STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 101

METRIC

W P 139-86-02 LOCATION Co-ords: N 4 755 456.3; E 411 548.6 ORIGINATED BY DM
DIST 2 HWY 401 BOREHOLE TYPE Washbore - NX Casing COMPILED BY PP
DATUM Geodetic DATE 86 11 18 - 86 11 20 CHECKED BY PP

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					
274.7	Ground Level																GR SA SI CL
0.0	Sand Some Gravel Occasional Cobbles traces of silt traces of clay Loose to Very Dense (Fill Material)		1	SS	28		274										19 67 10 4
			2	SS	11												
			3	SS	7		272										
			4	SS	7												
			5	SS	14												14 41 29 16
			6	SS	60/7.5 cm		270										
			7	SS	60/10 cm												
268.0			8	SS	45		268										13 73 10 4
6.7	Silty Clay trace/with sand traces of gravel Occasional Silt Seams and Layers Hard		9	SS	44												1 30 (69)
			10	SS	67												
			11	SS	32		266										
			12	SS	60		264										0 5 (95)
			13	SS	46		262										
259.0			14	SS	46		260										
15.7	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 102

METRIC

W P 139-86-02 LOCATION Co-ords: N 4 755 503.6; E 411 577.0 ORIGINATED BY DM
DIST 2 HWY 401 BOREHOLE TYPE Washbore - NX Casing COMPILED BY DM
DATUM Geodetic DATE 86 11 21 - 86 11 24 CHECKED BY DM

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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100						
								SHEAR STRENGTH						
274.5	Ground Level													
0.0	Silty Clay some/with sand trace/some gravel Firm to Very Stiff (Fill Material)		1	SS	4								13 28 (59)	
			2	SS	4									
			3	SS	10									
			4	SS	6									
			5	SS	24									
269.3			6	SS	22								3 35 (62)	
5.2	Silty Clay trace/some sand traces of gravel Hard		7	SS	41								1 12 (87)	
			8	SS	50									
			9	SS	43									
266.4			10	SS	25								6 16 (78)	
8.1	End of Borehole													
	* Water Level was observed to be 0.6 m below ground level, one day after the removal of casings.													

+³, x⁵: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

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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