May 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 4 (Colonel Talbot Road) Underpass Replacement
Site Number 19-405
Highways 401, 4 and 21 Structural Replacements
GWP 3030-11-00, Assignment No. 1 (3011-E-0046)
Ministry of Transportation, Ontario - West Region

Submitted to:
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DRAFT REPORT

Report Number: 12-1132-0076-1001-R01

Geocres No. _____

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PART A

FOUNDATION INVESTIGATION REPORT

HIGHWAY 4 (COLONEL TALBOT ROAD) UNDERPASS REPLACEMENT
SITE NUMBER 19-405
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS
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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 GWPs 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, two (2) culverts and improvements at five (5) Highway 401 interchanges.

This report addresses the replacement of the Highway 4 (Colonel Talbot Road) Underpass (Site 19-405) for GWP 3030-11-00.

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 (RFP) 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.

Dillon provided Golder Associates with preliminary drawings for this project in digital format.

2.0 SITE DESCRIPTION

2.1 General

The Highway 4 Underpass is located in the City of London, Ontario. The location of the project is shown on the Key Plan, Figure 1.

This section of Highway 401 is currently a four lane divided highway oriented generally northeast-southwest. The existing underpass was constructed in 1956 and consists of a single span, four lane concrete rigid frame structure. The area adjacent to the site consists of relatively flat-lying agricultural and commercial lands.

Site photographs are provided in Appendix D.

For the purposes of this report, Highway 401 and Highway 4 are assumed to be oriented in northeast-southwest and north-south directions, respectively.

Based on the preliminary design information provided by Dillon, it is understood that four preliminary alternatives are being considered for the underpass replacement and interchange improvements. Each of the four alternatives includes demolition of the existing structure and a new structure built about 30 metres to the east of the original structure alignment. It is anticipated that the replacement structure will be a two span structure with approximately 40 metre long spans and skewed supports. The new structure will carry two lanes of Highway 4 and a tapering speed change lane in each direction.

2.2 Site Geology

This project lies within the physiographic region of southern Ontario known as the Westminster Moraine. The physiographic mapping indicates that the Highway 4 Underpass site is situated on a till moraine.[1] The available surficial geology mapping indicates that lacustrine silt and sand as well as silty clay till is present at the site.[2]

The bedrock 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, based on the available mapping, to be at about elevation 160 metres or some 90 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 (West 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 current investigation was carried out on January 16, 2014, during which time 2 boreholes, identified as boreholes 105 and 106, were drilled. Boreholes 1 and 2 drilled for the preliminary foundation investigation in 2004 for the proposed replacement structure (Geocres Report No. 40I14-134) and boreholes 1, 3, 7 and 9 from the investigation for the original structure (Geocres Report No. 40I14-52), have been used to supplement the current data. The Record of Borehole sheets and laboratory test results for these boreholes have been included in Appendices B and C, respectively.

The approximate locations of the current and previous boreholes are 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		
105	4 746 690	404 955	251.65	11.13
106	4 746 771	404 897	252.17	11.13
1 (40I14-134)	4 746 750	404 901	252.27	30.94
2 (40I14-134)	4 746 664	404 895	251.88	51.97
1 (40I14-52)	4 746 715	404 882	251.00	10.82
3 (40I14-52)	4 746 731	404 892	251.97	8.38
7 (40I14-52)	4 746 682	404 905	249.66	9.60
9 (40I14-52)	4 746 697	404 915	250.55	21.34

The current 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 0.76 or 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with ASTM D1586. The results of the standard penetration testing (SPT) as presented on the Record of Borehole sheets and in Section 4 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.).

The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes, including cobbles and boulders, are known to be present in the native soils as discussed in the text of this report.

The current boreholes were terminated 11.1 metres below the existing ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. 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 Associates staff 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 soil 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.

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. Laboratory test results for samples obtained during the previous investigations are shown in Appendices B 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 surficial topsoil and fill materials over layers of silty clay, clayey silt, and silty clay till, which are underlain by an extensive deposit of clayey silt till.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawings 1 to 3. 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.

Materials described as silty clay in the boreholes advanced for Geocres Report No. 40I14-52 have been classified as clayey silt, silty clay, silty clay till or clayey silt till based of the soil descriptions provided on the Record of Borehole sheets, the results of Atterberg limits testing and comparison of the stratigraphy in adjacent boreholes. This has been reflected in the following report sections and the inferred profiles, Drawings 2 and 3.

4.1.1 Topsoil

About 240 to 610 millimetres of topsoil was encountered at the ground surface in boreholes 105, 106, 1 (40I14-134) and 2 (40I14-134). In addition, a 450 millimetre thick layer of buried topsoil was encountered beneath the fill materials in borehole 1 (40I14-134) at elevation 250.6 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.

4.1.2 Fill

Stiff to very stiff clayey silt fill materials were encountered beneath the surficial topsoil in boreholes 105, 106, 1 (40I14-134) and 2 (40I14-134). The fill materials were about 0.9 to 2.4 metres thick at the borehole locations. The fill material had N values ranging from 10 to 21 blows per 0.3 metres. Samples of the fill material had water contents of 16 to 32 per cent. An Atterberg limits determination carried out on a sample of the clayey silt fill indicated plastic and liquid limits of 19 and 30 per cent, respectively, and a plasticity index of 11 per cent, indicating a clay of low plasticity. The Atterberg limits data are shown on Figure A-5. A grain size distribution curve for a sample of the clayey silt fill material is shown on Figure A-1.

4.1.3 Silty Clay

Firm to hard silty clay was encountered beneath the fill in borehole 106, the buried topsoil in borehole 1 (40I14-134), and at the ground surface in boreholes 1 (40I14-52) and 3 (40I14-52) between elevations 250.1 and 251.0 metres. The silty clay layers were between 0.8 and 2.3 metres thick. The silty clay had N values ranging from 5 to 44 blows per 0.3 metres. Samples of the silty clay had water contents ranging from 18 to 30 per cent. The silty clay had plastic and liquid limits ranging from 16 to 22 per cent and 35 to 46 per cent, respectively, with corresponding plasticity indices ranging from 16 to 24 per cent, indicating clays of intermediate plasticity. The Atterberg limits data are shown on Figure A-5 and in Appendices B and C. A grain size distribution curve for a sample of the silty clay from borehole 106 is shown on Figure A-2.

In addition, a layer of hard silty clay was encountered within the clayey silt till in borehole 2 (40I14-134) at elevation 205.4 metres. The silty clay layer was about 1.4 metres thick at the borehole location. The silty clay had an N value of 38 blows per 0.3 metres with a water content of about 18 per cent.

4.1.4 Clayey Silt

Layers of soft to stiff clayey silt about 0.7 to 1.2 metres thick were encountered beneath the fill in borehole 2 (40I14-134) and at the ground surface in boreholes 7 (40I14-52) and 9 (40I14-52) between elevations 249.7 and 250.6 metres. The clayey silt had N values of 2 to 8 blows per 0.3 metres. A sample of the clayey silt had a water content of 22 per cent.

4.1.5 Silt

Layers of compact to dense silt about 0.3 to 0.8 metres thick were encountered below the clayey silt in borehole 2 (40I14-134) and within the clayey silt till in boreholes 1 (40I14-134), 2 (40I14-134) and 9 (40I14-52) between elevations 229.8 and 249.0 metres. Borehole 9 (40I14-52) was terminated in a silt layer after exploring it for about 0.6 metres. The silt had N values of 16 to 48 blows per 0.3 metres. The water content of a sample of the silt was 20 per cent.

4.1.6 Silty Fine Sand

A 0.6 metre thick layer of silty fine sand was encountered at elevation 229.1 metres within the clayey silt till in borehole 1 (40I14-134).

4.1.7 Sandy Silt Till

Layers of dense to very dense sandy silt till, 1.2 and 1.4 metres thick, were encountered in borehole 2 (40I14-134) within the clayey silt till at elevations 202.2 and 243.5 metres, respectively. Cobbles were noted within the sandy silt till layers. The upper sandy silt till and lower layers of sandy silt till had N values of 34 and greater than 100 blows per 0.3 metres, respectively. Water contents of samples the sandy silt till were 11 per cent. Cobbles and boulders should be expected in the sandy silt till.

4.1.8 Silty Clay Till

Stiff to hard silty clay till was encountered beneath the fill material in borehole 105, the silty clay in boreholes 1 (40I14-134) and 3 (40I14-52) and beneath the upper layer of clayey silt till in borehole 2 (40I14-134). The silty clay till was encountered between elevations 246.7 and 249.4 metres. The silty clay till had N values generally ranging from 14 to 33 blows per 0.3 metres. Samples of the silty clay till had water contents between 18 and 21 per cent. Atterberg limits determinations carried out on samples of the silty clay till yielded plastic and liquid limits of 15 to 20 per cent and 35 to 36 per cent, respectively, with plasticity indices of 16 to 20 per cent indicating clays of intermediate plasticity. The Atterberg limits data from boreholes 105 and 106 are provided on Figure A-5 and the data from the previous boreholes are provided in Appendices B and C. A grain size distribution curve for a sample of the silty clay till from borehole 105 is shown on Figure A-3. Cobbles and boulders should be expected in the silty clay till.

4.1.9 Clayey Silt Till

An extensive deposit of stiff to hard clayey silt till was encountered in each of the boreholes. The clayey silt till was encountered beneath the silty clay till in boreholes 105, 1 (40I14-134), 2 (40I14-134) and 3 (40I14-52), beneath the silty clay in boreholes 106 and 1 (40I14-52) and beneath the clayey silt in boreholes 7 (40I14-52) and 9 (40I14-52). The surface of the clayey silt till was encountered between elevations 244.4 and 250.0 metres. The clayey silt till in boreholes 105, 106, 1 (40I14-134), 2 (40I14-134), 1 (40I14-52), 3 (40I14-52) and 7 (40I14-52) was explored for between 4.6 and 44.5 metres prior to terminating the boreholes. In borehole 9 (40I14-52), the clayey silt till was 19.6 metres thick.

In addition, a 1.8 metre thick layer of clayey silt till was encountered beneath the upper layer of silt in borehole 2 (40I14-134) at elevation 248.2 metres.

The clayey silt till had layers of silt, silty fine sand, sandy silt till, sand and sand and gravel, as noted in the above sections. Boulders were encountered within the clayey silt till in boreholes 106 and 2 (40I14-134) at elevations 210.3 and 213.7 metres. Cobbles and boulders should be expected throughout the clayey silt till deposit.

The clayey silt till had N values of 8 to 125 blows per 0.3 metres. Water contents of samples of the clayey silt till ranged from 8 to 22 per cent. The results of twenty Atterberg limits determinations carried out on samples of the clayey silt till yielded plastic limits ranging from 12 to 20 per cent, liquid limits ranging from 25 to 34 per cent and plasticity indices of between 9 and 19 per cent, indicating a clay of low plasticity. The Atterberg limits data from boreholes 105 and 106 are provided on Figure A-5 and the data from the previous boreholes are provided in Appendices B and C. Grain size distribution curves for samples of the clayey silt till from boreholes 105 and 106 are shown on Figure A-4.

4.1.10 Sand and Gravel

A 0.3 metre thick layer of sand and gravel was encountered within the clayey silt till in borehole 2 (40I14-134) at elevation 202.5 metres.

4.1.11 Sand

A layer of very dense sand was encountered beneath the sandy silt till in borehole 2 (40I14-134) at elevation 201.0 metres. The sand had an N value of 125 blows per 0.3 metres, for a test partially completed in the layer, with a water content of about 14 per cent.

4.2 Groundwater Conditions

Groundwater conditions were observed during and on completion of drilling and sampling. Groundwater was encountered in boreholes 105, 1 (40I14-134), 2 (40I14-134), 1 (40I14-52), 3 (40I14-52), 7 (40I14-52) and 9 (40I14-52). A summary of the encountered groundwater levels is provided in the table below.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level	
		Depth (m)	Elevation (m)
105	251.7	2.6	249.1
106	252.2	Dry	Dry
1 (40I14-134)	252.3	8.9	243.4
		23.2	229.1
2 (40I14-134)	251.9	3.7	248.2
		49.4	202.5
		50.9	201.0
1 (40I14-52)	251.0	0.8	250.2
3 (40I14-52)	251.0	0.9	250.1
7 (40I14-52)	249.7	0.9	248.8
9 (40I14-52)	250.6	0.6	250.0

The encountered water levels are not considered to be representative of the long-term stabilized groundwater conditions. Based on the encountered groundwater levels and the change in colour from brown to grey, the groundwater level is inferred to be at about elevation 248 metres. Groundwater levels should be 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 under the direction of the Field Investigation Manger, Mr. David J. Mitchell.

Routine laboratory tests were 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 Mr. Michael E. Beadle, P.Eng., an Associate with Golder Associates. An independent quality review of this report was carried out by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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PART B

FOUNDATION DESIGN REPORT

HIGHWAY 4 (COLONEL TALBOT ROAD) UNDERPASS REPLACEMENT
SITE NUMBER 19-405
HIGHWAYS 401, 4 AND 21 STRUCTURAL REPLACEMENTS
GWP 3030-11-00, ASSIGNMENT No. 1 (3011-E-0046)
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION

May 2015
Report No. 12-1132-0076-1001-R01

6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design for the replacement of the Highway 4 Underpass (Site 19-405). 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, it is understood that four preliminary alternatives are being considered for the underpass replacement and interchange improvements. Each of the four alternatives includes demolition of the existing structure and a new structure built about 30 metres to the east of the original structure alignment. It is anticipated that the replacement structure will be a two span structure with approximately 40 metre spans and skewed supports. The new structure will carry two lanes of Highway 4 and a tapering speed change lane in each direction.

6.1 Existing Structure

The existing underpass was constructed in 1956 and consists of a single span, four lane, concrete rigid frame structure. The structure has a clear span of about 37 metres and is about 17 metres wide constructed on a skew of 26 degrees. The superstructure is comprised of nine reinforced concrete Tee girders at a spacing of about 2 metres on centre.

Based on the original design drawings, the structure was designed with full height abutment walls parallel to Highway 401 with frame legs aligned with each Tee girder and cast monolithically with the abutment wall. Wing walls are parallel to Highway 4 which are attached to the abutment walls and supported vertically by discrete columns and horizontally by a reinforced concrete buried tie beam located approximately 7 metres behind the abutment wall. Retaining walls are located at all four corners of the bridge aligned with the face of the abutments (parallel to Highway 401) and curved at the ends. The abutment walls, columns and retaining walls were founded on conventional spread footings between elevations 248.4 and 249.0 metres. The abutment foundations were designed with keys cut to elevations 248.1 and 247.8 metres at the north and south abutments, respectively. The concrete cantilever retaining walls extend between 2.1 and 3.3 metres above the highway elevation and are between 5.0 and 15.4 metres long.

The Highway 401 pavements at the site are at about elevation 252 metres and the Highway 4 pavements are at about elevations 259.2 and 258.8 metres at the north and south abutments, respectively.

As indicated in the RFP, this bridge has been recommended for replacement due to a combination of condition deficiency and functional obsolescence. The girders have severe spalls and exposed reinforcing. Further, the existing span arrangement cannot accommodate the future widening of Highway 401.

6.2 Bridge Foundations

The subsurface soil conditions at the site typically consist of topsoil and fill materials underlain by layers of silt, silty clay, clayey silt and silty clay till which overly an extensive deposit of clayey silt till. The inferred groundwater level is expected at about elevation 248 metres.

Integral, semi-integral and conventional abutments are considered to be viable alternatives for design of the replacement structure. 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. Provided the abutment heights and wingwall lengths 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. Integral abutments are typically supported by driven steel H-piles. Consideration may also be given to supporting integral abutments on concrete filled steel tube piles provided the increased stiffness can be accommodated in the design. Shallow spread footings are not suitable for integral abutments. Conventional or semi-integral abutments may be founded on spread footings bearing on native soils, steel H-piles, concrete filled steel tube piles or drilled concrete shafts (caissons).

If a conventional or semi-integral design is selected, shallow foundations are the preferred alternative for support of the abutments for this bridge. Shallow foundations are also preferred for support of any piers and the retaining walls. A comparison of foundation alternatives is presented in Table I following the text of this report.

6.2.1 Shallow Foundations

The abutments, piers and retaining walls for the replacement bridge may be founded on conventional spread/strip footings. Assuming the new footings are constructed at similar elevations as the existing footings, a factored geotechnical resistance at Ultimate Limit States (ULS) of 450 kilopascals and a geotechnical reaction at Serviceability Limit States (SLS) of 300 kilopascals may be used for footings founded on the very stiff to hard silty clay to clayey silt till below elevations 248.5 and 248.0 metres on the north and south sides of Highway 401, respectively.

Alternatively, it has been indicated that perched abutments founded on engineered fill may be used. The engineered fill should be constructed on properly prepared very stiff to hard silty clay to clayey silt glacial till at or below elevation 248.5 metres and should consist of a minimum of 2 metres of Ontario Provincial Standard Specifications (OPSS) Granular A placed in maximum 300 millimetre thick loose lifts and compacted to at least 98 per cent of the standard Proctor maximum dry density. The engineered fill should extend a minimum of 1 metre plus the fill thickness beyond the footing and should be sloped at an inclination of 1 horizontal to 1 vertical to intersect the subgrade. Abutment footings founded on engineered fill as described above may be designed using a factored geotechnical resistance at ULS of 600 kilopascals and a geotechnical reaction at SLS of 400 kilopascals.

The geotechnical resistances and reactions provided above are net values. As such, these values do not include the overburden pressures associated with the existing side hill fill above the founding elevation. The SLS values correspond to an estimated total settlement of 25 millimetres. A footing width of 4 metres has been assumed.

Resistance to Lateral Forces

Resistance to lateral forces/sliding between the concrete spread/strip footings and the native, undisturbed subsoil should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC). Assuming that the founding soils are not loosened or disturbed during excavation and footing construction, an angle of friction between the mass cast-in-place concrete and the founding soils of 30 degrees and corresponding unfactored coefficient of friction, tan δ, of 0.58 may be used.

Frost Protection

All footings should be provided with a minimum of 1.2 metres of earth cover or thermal equivalent for frost protection purposes.

6.2.2 Deep Foundations

Geotechnical Axial Resistance - Driven Piles

For the design of HP 310 x 110 piles and 324 millimetre diameter, 9.5 millimetres wall thickness concrete filled, steel tube piles driven to practical refusal, the factored geotechnical axial resistances at ULS and geotechnical reactions at SLS provided in the following table may be used. Based on the conditions encountered in borehole 2 (40I14-134) it is anticipated that refusal will occur on the very dense sand and gravel at about elevations 201.0 to 202.5 metres. This will result in pile lengths of about 48 to 49 metres.

Pile Type	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	250.8	Very dense sand and gravel	201.0	1,600	1,200
324 mm OD x 9.5 mm tube	250.8	Very dense sand and gravel	202.5	2,000	1,400

Alternatively, steel H-piles and tube piles driven into the very stiff to hard clayey silt till at elevation 225 metres may be designed using the factored geotechnical axial resistances at ULS and geotechnical reactions at SLS provided in the following table.

Pile Type	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	250.8	Very stiff to hard clayey silt till	225.0	1,000	750
324 mm OD x 9.5 mm tube	250.8	Very stiff to hard clayey silt till	225.0	1,000	750

The SLS values provided in the tables 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.

The above cut-off elevations have been assumed based on the existing ground surface elevation at the site of 252.0 metres and a soil cover of 1.2 metres for frost protection. Higher cut-off elevations will likely be required for piles supporting integral abutments.

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 the CSPs detailing the sand gradation should be included in the Contract Documents.

Piles should be installed and monitored in accordance with 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 native silty clay till, clayey silt till and sand and gravel strata is known to contain cobbles and boulders which may interfere with advancement of the piles or cause damage to pile tips. Appropriate driving shoes should be used to minimize 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. 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. It is recommended that, following the selection of the driving equipment, the piling contractor submit for review his proposed pile driving criteria based on the characteristics of the hammer and equipment intended for use.

Geotechnical Axial Resistance – Drilled Caissons

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 kilonewtons, 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. 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 kilonewtons, q_{BN} is the nominal unit base resistance in kilopascals 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/m³)
Fill	Where applicable	-	-	19.0
Silty Clay Till	246 to 249	10	360	20.0
Clayey Silt Till	240 to 246	35	1,800	21.0
	235 to 240	60	1,800	21.0
	230 to 235	85	1,800	21.0
	225 to 230	110	1,800	21.0

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. A 1.2 metre diameter caisson founded in the clayey silt till at elevation 235 metres may be designed using a factored geotechnical resistance at ULS of 2,000 kilonewtons.

Downdrag Loads

The new approach embankments will cause consolidation settlement of the underlying clayey silt till deposit as a result of the embankment surcharge. The consolidation settlement is time-dependent and will not completely occur during the construction period unless the embankments are placed well in advance of bridge construction.

Post-construction settlement of the clayey deposit relative to the piles will result in development of negative skin friction acting on the piles. Therefore, negative skin friction, or downdrag loads, will need to be taken into account during design of the piles supporting the abutments.

Based on the results of the investigation, for preliminary design, the unfactored downdrag load acting on the piles may be taken as 300 kilonewtons per pile.

Frost Protection

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

Backfilling of Integral Abutments

It has been indicated that the feasibility of integral abutments at a skew angle of 28.7 degrees to Highway 4 is being reviewed and is considered to be dependent on the interface friction angle that can be achieved between the inside face of the abutment wall and the backfill material. It is considered that an unfactored interface friction angle of 34 degrees may be achieved, provided the backfill material consists of a crushed granular material such as OPSS Granular A, the movement of the abutment is of sufficient magnitude to mobilize shear forces in the backfill and the surface of the abutment wall is sufficiently roughened. Further study should be carried out at a later design and/or pre-engineering stage to determine/confirm the above noted parameters.

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 soil properties.

k_h = coefficient of horizontal subgrade reaction (MPa/m) $= n_h (z/d)$ for cohesionless soils

$= \frac{67\, S_u}{d}$ for cohesive soils

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 non-integral abutments or pier foundations. The stratigraphy presented in the table below has been simplified for the purposes of this report.

Location	Soil Type	Elevation (m)	n_h (MPa/m)	S_u (MPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 – 10	-
North Abutment	Existing fill – Stiff to very stiff clayey silt	Above 249	-	70
	Stiff to hard silty clay till	245 to 249	-	150
	Very stiff to hard clayey silt till	Below 245	-	200
South Abutment	Existing fill - Soft to stiff clayey silt	Above 249	-	-
	Very stiff silty clay till	247 to 249	-	150
	Very stiff to hard clayey silt till	Below 247	-	200

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	35	*
- 324 mm OD x 9.5 mm tube	55	40
Semi-Integral or Conventional abutments		
- HP 310 x 110, strong axis bending	260	60
- 324 mm OD x 9.5 mm tube	195	100
1.2 metre diameter Caisson	1,000	125

* Load to mobilize 10 mm horizontal displacement is greater than or equal to 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-head conditions 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 existing foundation elevation. A 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 top of the pile.

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:

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

6.3 Liquefaction Potential and Seismic Analysis

6.3.1 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 CHDBC Clause 4.4.5.1.

6.3.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.[4] The characteristics of the cohesive soils indicate that they are not susceptible to liquefaction. Although layers of saturated granular materials are present, they were found to be relatively thin and have normalized N values 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 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.

6.4 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, the nature of the soils behind the backfill, the freedom of lateral movement of the structure and 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 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

[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.

accordance with OPSS.PROV 501. 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 kilopascals should be included in the lateral earth pressures for the structural design of the abutment walls 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/m³
Coefficient 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 III
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_p	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 CHDBC.

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.

6.5 Construction Considerations

6.5.1 Shallow foundations

The founding soils are sensitive to disturbance and loosening due to water seepage and/or ponding and construction equipment or foot traffic when damp to wet. Placement of a concrete working slab (100 millimetres thick of 20 megapascals concrete) will be required at the base of the excavations for the footing areas. Exposure without protection from the working slab may result in loosening or softening of the founding soils. The cleaned excavation base should be inspected by a Quality Verification Engineer (QVE) qualified in geotechnical engineering prior to placing the working slab. It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the working slab placed immediately after footing inspection.

6.5.2 Deep Foundations

Cobbles and boulders should be expected in the soils at the site and which may impact pile driving or caisson drilling operations. An NSSP should be added to the Contract Documents to alert the contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile or caisson installation.

Deep foundations should be installed and monitored in accordance with OPSS 903 and Ontario Provincial Standard Drawing (OPSD) 3000.150 and 3001.150, and SS103-11 (Pile Driving Control). The H-piles and steel tube piles should be equipped with Type I driving shoes as shown in OPSD 3000.100 and 3001.100, respectively.

6.6 Embankments

It is anticipated that new approach embankments will be required for the new structure. Alternatively, if the current alignment is maintained, or if the new alignment is sufficiently close to the existing, the current embankments may be widened. All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankments. Prior to placement of embankment fill material, the exposed subgrade should be proofrolled under the direction of the geotechnical QVE. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 to 1.0 metres where pavement structure will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments, as appropriate, and adequately compacted. Embankments shall be constructed in accordance with OPSS 206 and OPSD 208.010.

It is anticipated that the approach embankments will be about 6 metres in height with the crests about 30 metres wide. A preliminary settlement analysis was carried out using the results of the boreholes and estimates of soil properties from similar soils in the surrounding area. Settlement at the embankment crests was estimated to be 75 millimetres. Ninety per cent of the long-term settlement is expected to occur about 4 years following embankment construction with 50 per cent settlement within 1 year. These settlement estimates are very preliminary and should be refined during detail design using site specific soil properties.

In order to minimize the effect of settlement of the approach embankments relative to a pile supported bridge, consideration may be given to constructing the embankments well in advance of bridge construction, utilizing a

2 metre surcharge prior to embankment construction to accelerate consolidation or the use of lightweight expanded polystyrene (EPS) fill. Early construction of the embankments is preferred.

Embankments constructed with SSM and founded on the stiff silty clay till or 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 with side slopes 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 mid-height.

6.7 Excavations and Temporary Cut Slopes

Excavations for pile caps and/or abutments will penetrate the existing fill and organic materials into the silty clay till and clayey silt till. The groundwater level is expected to be at about elevation 248.0 metres and will fluctuate seasonally. The excavations may extend below the groundwater level; however, seepage volumes from the cohesive founding soils are expected to be low. If necessary, groundwater control may be achieved by pumping from properly constructed and filtered sumps in the base of the excavation. 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 would be classified as Type 2 soils.

6.7.1 Temporary Roadway Protection

In order to support the excavation sides and permit the use of vertical cuts, temporary road protection systems will be required where space is restricted and will not permit the use of open 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 system(s) could be in the form of struts and walers in the case of pile cap/abutment excavations or rakers and anchors. The support system must be designed to accommodate the loads applied from pressures and surcharge pressures from area line or point loads as well as address the impact(s) of sloping ground behind the system. The lateral movement of the temporary support system should meet Performance Level 2 as specified in OPSS.PROV 539.

7.0 RECOMMENDATIONS FOR DETAIL DESIGN

Depending on the selection of integral, semi-integral or conventional abutments, both driven piles and shallow foundations are considered to be feasible alternatives for support of the replacement structure. A Foundation Investigation and Design Report should 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, requiring one borehole at each end of each abutment, one borehole at each approach, and a borehole at any proposed pier location. The number and location of required boreholes will be dependent on the location and alignment of the proposed structure selected for Detail Design and may be reduced if the location of any part of the of the proposed structure coincides with the location of existing boreholes. In addition, if embankment settlement is of concern, odometer testing would be required to determine site specific soil properties for refinement of the settlement analysis.

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, wingwalls and any proposed piers. Embankment stability and settlement should be evaluated. Further, if staged construction is to be used, the discussion on temporary roadway protection should include lateral earth pressures and effect of ground conditions on shoring design and construction.

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 Mr. Michael E. Beadle, P.Eng., an Associate with Golder Associates. An independent quality review of this report was carried out by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES – UNDERPASS REPLACEMENT

Middlesex Road 73, Site 19-304
Highway 401 Structural Replacements
GWP 3053-11-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS[1]	RISKS/ CONSEQUENCES
Spread footings supported on compact to very dense sand to sand and gravel	• Feasible for abutments and central pier(s).	• Least expensive option. • Ease of construction.	• Not compatible with integral abutments. • More settlement expected than with deep foundations. • Larger work area required compared to caissons or driven piles.	• Low	• 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	• Preferred technical alternative for abutments. • May be considered for central pier(s).	• High bearing resistance. • Negligible settlement. • Compatible with integral abutments.	• More expensive than shallow foundations. • Can be damaged and deflected by cobbles and boulders within sand and sand and gravel deposits. • More construction noise and vibration compared to shallow foundations or caissons. • H-piles cannot be visually inspected at depth.	• Moderate	• Possible pile tip damage if piles are not adequately protected while driving through sand and sand and gravel deposits. • Variation in pile tip elevations.

COMPARISON OF FOUNDATION ALTERNATIVES – UNDERPASS REPLACEMENT

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS[1]	RISKS/ CONSEQUENCES
			• Integrity inspection requires specialty dynamic testing.		
• Concrete caissons drilled into dense to very dense granular soils	• Feasible for abutments but not preferred. • May be preferred for pier(s).	• Negligible settlement. • 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.	• 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.	• High	• Cleaning of base could be problematic or overlooked during construction.

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

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.)

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, 60º 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 cm^2 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.

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

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u 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

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
v	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 = σ'_p/σ'_{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 δ
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)

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 105

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00
LOCATION N 4746690.3 , E 404954.5
ORIGINATED BY BT
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE January 16, 2014
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
251.65	GROUND SURFACE							
0.00	TOPSOIL, clayey Brown							
0.24	FILL, clayey silt, trace to some sand, trace to some topsoil, with topsoil layers Soft to stiff Brown and grey	1	SS	12	0	8	85	7
		2	SS	10				
249.06		3	SS	4				
2.59	SILTY CLAY TILL, trace sand, trace gravel Soft to very stiff Brown becoming grey below about elev. 247.7m	4	SS	22	1	6	54	39
		5	SS	20				
247.23								
4.42	CLAYEY SILT TILL, trace to some sand Very stiff to hard Grey	6	SS	27	0	6	54	40
		7	SS	30				
		8	SS	22				
		9	SS	32				
		10	SS	34	1	10	46	43
		11	SS	29				
240.52								
11.13	END OF BOREHOLE Groundwater encountered at about elev. 249.1m during drilling on January 16, 2014.							

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 251, 250, 249, 248, 247, 246, 245, 244, 243, 242, 241

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

○ 3% STRAIN AT FAILURE

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London, Ontario

RECORD OF BOREHOLE No 106

METRIC

PROJECT 12-1132-0076
W.P. 3030-11-00
LOCATION N 4746771.3 , E 404896.8
ORIGINATED BY BT
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE January 16, 2014
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
252.17	GROUND SURFACE								
0.00 / 251.65	TOPSOIL, clayey Brown								
0.52 / 250.80	FILL, clayey silt, some topsoil, trace sand, with sandy silt layers Stiff Brown	1	SS	11					
1.37 / 250.04	SILTY CLAY, trace sand Stiff Brown and grey mottled	2	SS	14		0	7	49	44
2.13	CLAYEY SILT TILL, trace to some sand Very stiff to hard Brown becoming grey below about elev. 247.8m	3	SS	38		0	7	51	42
		4	SS	33					
		5	SS	41					
		6	SS	22		0	6	56	38
		7	SS	26					
		8	SS	22					
		9	SS	23		1	18	46	35
		10	SS	31					
241.04		11	SS	31					
11.13	END OF BOREHOLE Borehole dry during drilling on January 16, 2014.								

GROUND WATER CONDITIONS

ELEVATION SCALE: 252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

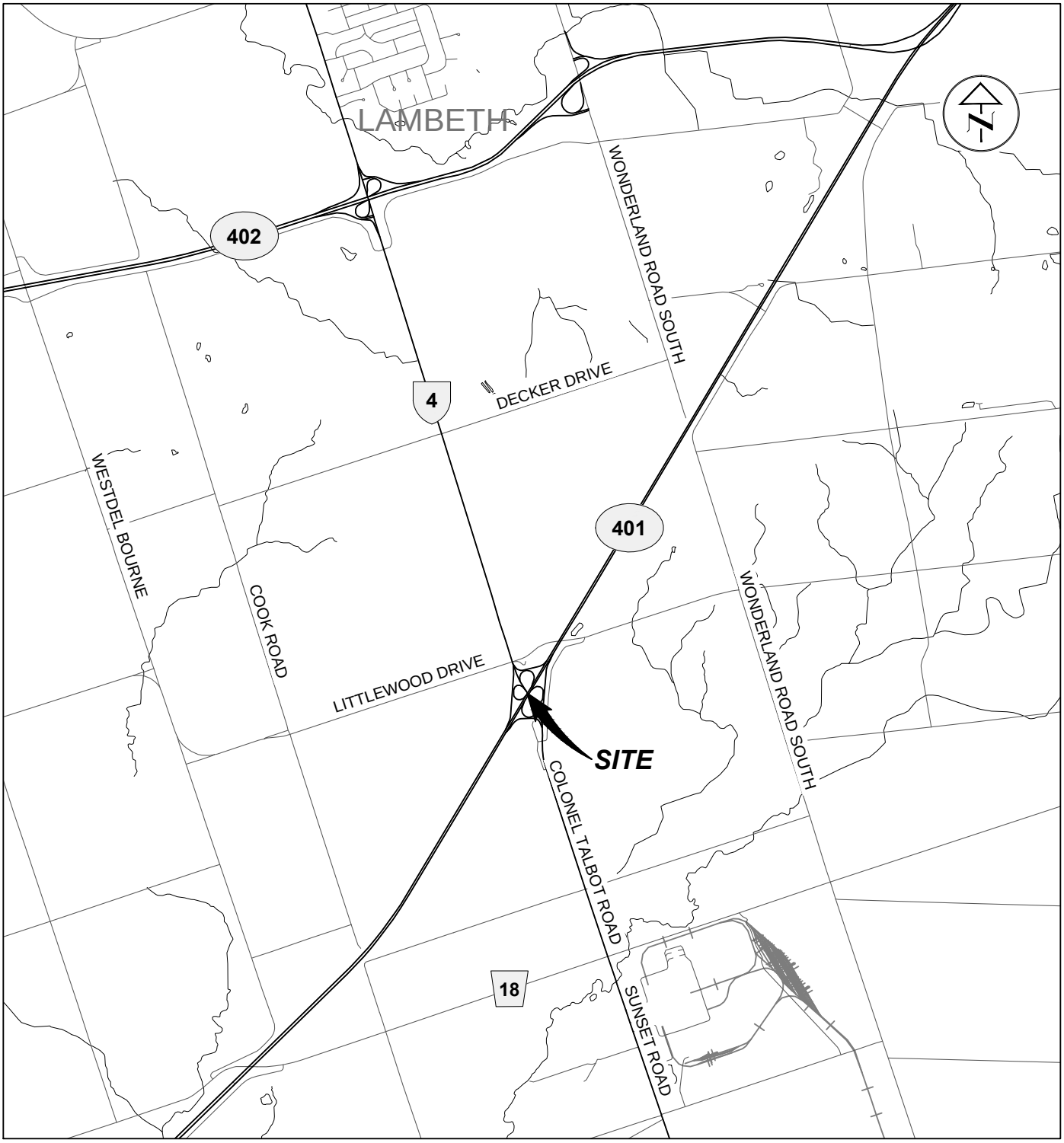
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

46

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



SCALE
0 1000 2000m
1:50,000

REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3054-11-00

TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0076	FILE No.	1211320076-1001-F01001
		SCALE	AS SHOWN REV. 0
CADD	LMK/WDF	Feb. 4/14	
CHECK			

FIGURE 1

Drawing file: 1211320076-1001-F01001.dwg Jan 26, 2015 - 11:53am

MINISTRY OF TRANSPORTATION, ONTARIO

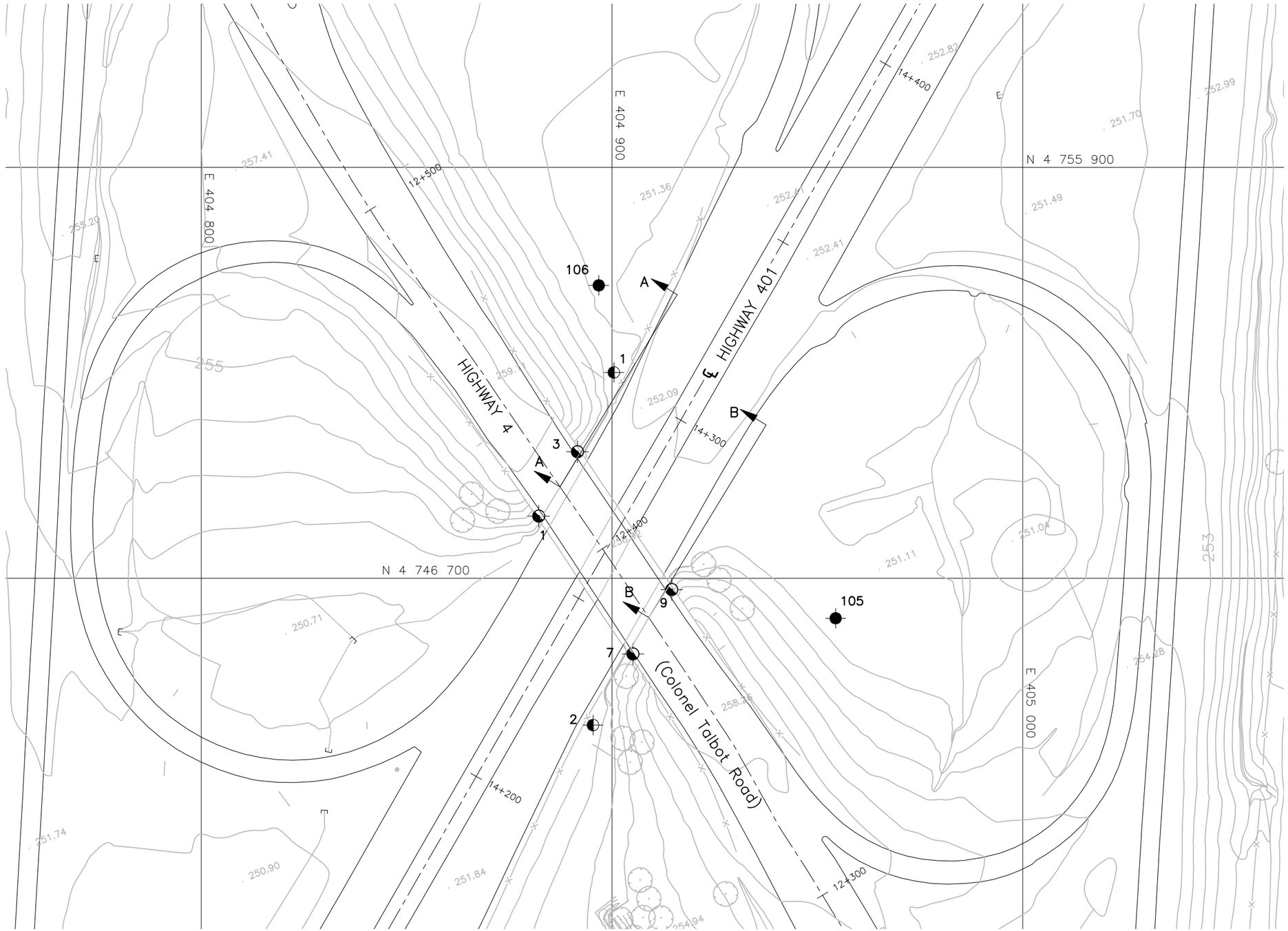
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 3030-11-00

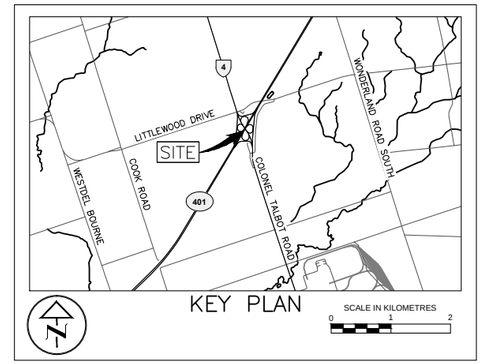
SHEET

HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



PLAN
SCALE
10 0 10 20 m



KEY PLAN

LEGEND

- Borehole – Current Investigation
- Borehole (Geocres 40I14–134)
- Borehole (Geocres 40I14–52)

No.	ELEVATION	CO-ORDINATES (NAD 83, MTM ZONE 11)	
		NORTHING	EASTING
105	251.65	4 746 690.3	404 954.5
106	252.17	4 746 771.3	404 896.8
Geocres 40I14–134			
1	252.27	4 746 750.1	404 900.5
2	251.88	4 746 664.3	404 895.4
Geocres 40I14–52			
1	251.00	4 746 715.1	404 882.2
3	250.97	4 746 730.8	404 891.6
7	249.66	4 746 681.6	404 905.1
9	250.55	4 746 697.3	404 914.6

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.

HWY. 401	PROJECT NO. 12-1132-0076		DIST.
SUBM'D. BT	CHKD. NAG	DATE: Feb. 13/14	SITE: 19-405
DRAWN: LMK\WDF	CHKD.	APPD.	DWG. 1

PLOT DATE: January 26, 2015
FILENAME: N:\active\2012\1132 - Geo\1132-0000\12-1132-0076 DILLON-11 STRUCTURES-3011-E-0046\Drafting\AutoCAD files\211320076-1001-001001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

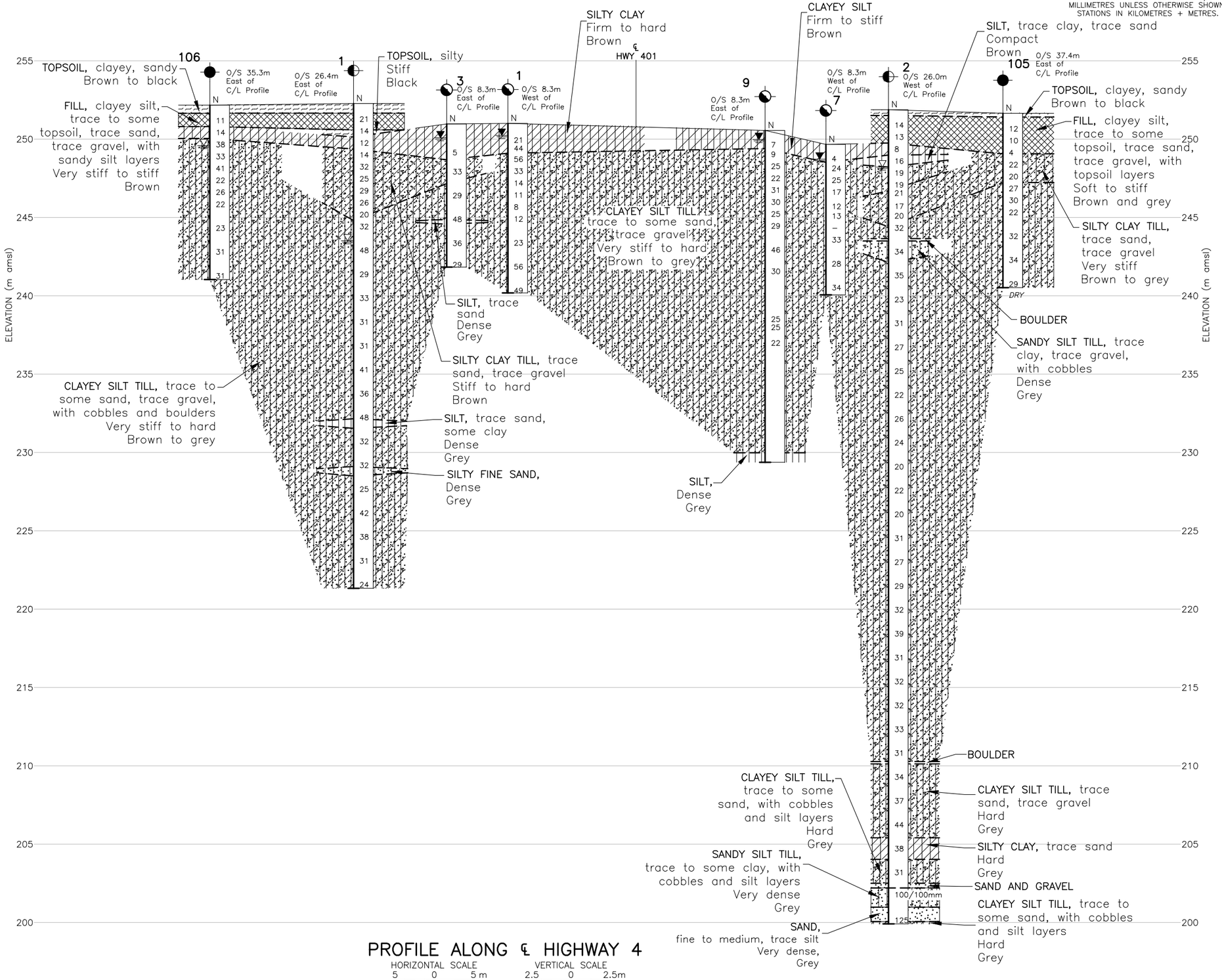
CONT No.
WP No. 3030-11-00

SHEET

HIGHWAY 4 UNDERPASS REPLACEMENT

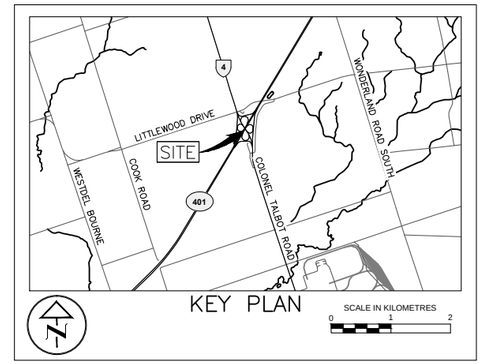
HIGHWAY 401 IMPROVEMENTS
SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



PROFILE ALONG ℄ HIGHWAY 4

HORIZONTAL SCALE 5 0 5 m
VERTICAL SCALE 2.5 0 2.5m



KEY PLAN
SCALE IN KILOMETRES 0 1 2

LEGEND

- Borehole – Current Investigation
- Borehole (Geocres 40I14–134)
- Borehole (Geocres 40I14–52)
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on Jan. 1955.
- WL encountered during drilling
- DRY — Water level not established

No.	ELEVATION	CO-ORDINATES (NAD 83, MTM ZONE 11) NORTHING	EASTING
105	251.65	4 746 690.3	404 954.5
106	252.17	4 746 771.3	404 896.8
Geocres 40I14–134			
1	252.27	4 746 750.1	404 900.5
2	251.88	4 746 664.3	404 895.4
Geocres 40I14–52			
1	251.00	4 746 715.1	404 895.2
3	250.97	4 746 730.8	404 891.6
7	249.66	4 746 681.6	404 905.1
9	250.55	4 746 697.3	404 914.6

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)

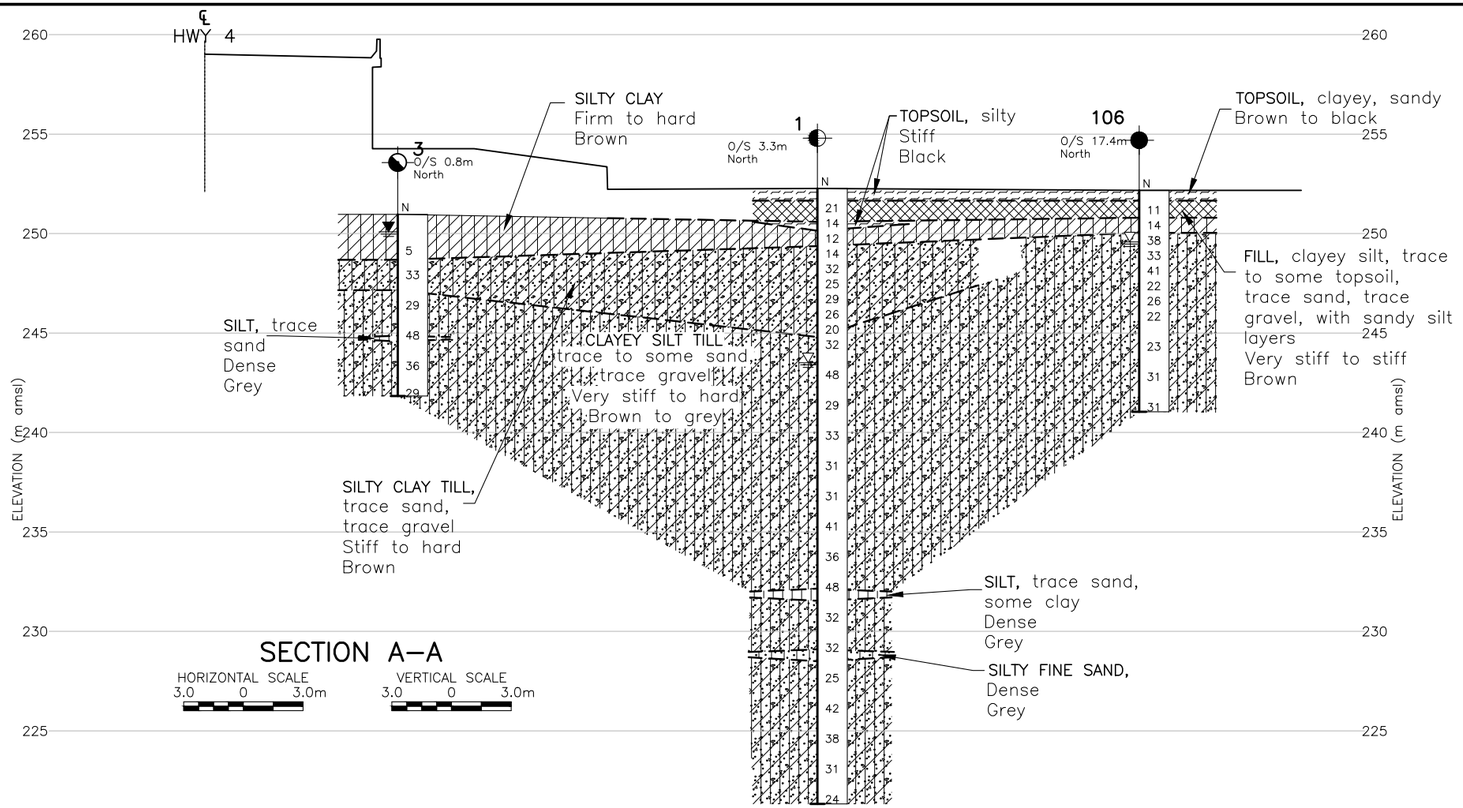
NO.	DATE	BY	REVISION

Geocres No.

HWY. 401	PROJECT NO. 12-1132-0076		DIST.
SUBM'D. BT	CHKD. NAG	DATE: Feb. 13/14	SITE: 19-405
DRAWN: LMK	CHKD.	APPD.	DWG. 2

PLOT DATE: January 26, 2015
FILENAME: N:\active\2012\1132 - Geo\1132-0000\12-1132-0076 DILLON-11 STRUCTURES-3011-E-0046\Drafting\AutoCAD files\1211320076-1001-001001.dwg

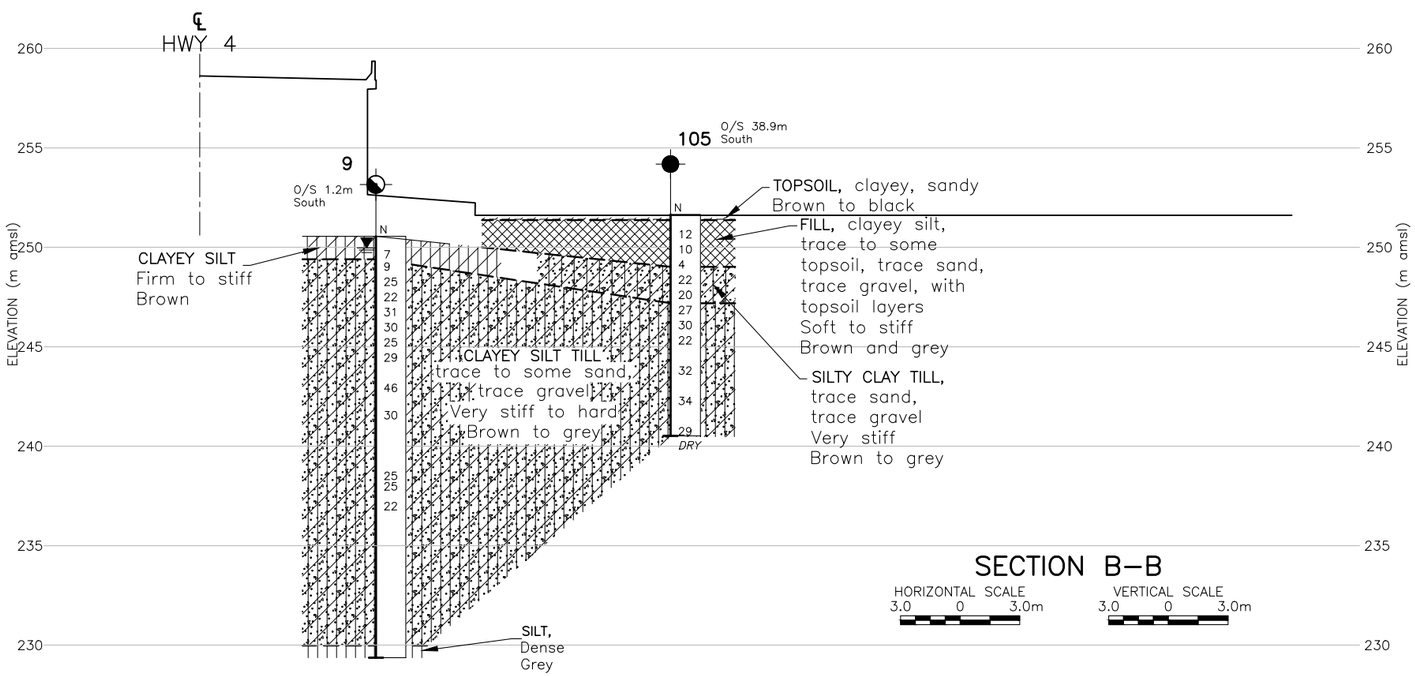
MINISTRY OF TRANSPORTATION, ONTARIO



SECTION A–A

HORIZONTAL SCALE 3.0 0 3.0m

VERTICAL SCALE 3.0 0 3.0m



SECTION B–B

HORIZONTAL SCALE 3.0 0 3.0m

VERTICAL SCALE 3.0 0 3.0m

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.

WP No. 3030–11–00

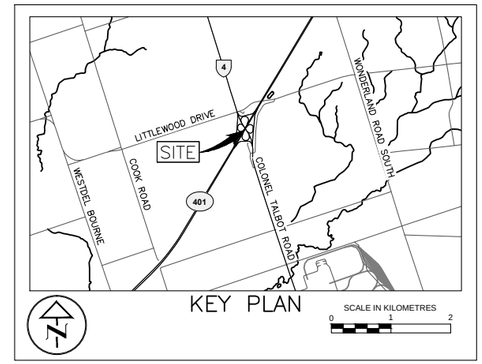
SHEET

HIGHWAY 4 UNDERPASS REPLACEMENT

HIGHWAY 401 IMPROVEMENTS

SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole – Current Investigation
- Borehole (Geocres 40I14–134)
- Borehole (Geocres 40I14–52)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on Jan. 1955.
- WL encountered during drilling
- *DRY* Water level not established

No.	ELEVATION	CO–ORDINATES (NAD 83, MTM ZONE 11)	
		NORTHING	EASTING
105	251.65	4 746 690.3	404 954.5
106	252.17	4 746 771.3	404 896.8
Geocres 40I14–134			
1	252.27	4 746 750.1	404 900.5
Geocres 40I14–52			
3	250.97	4 746 730.8	404 891.6
9	250.55	4 746 697.3	404 914.6

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

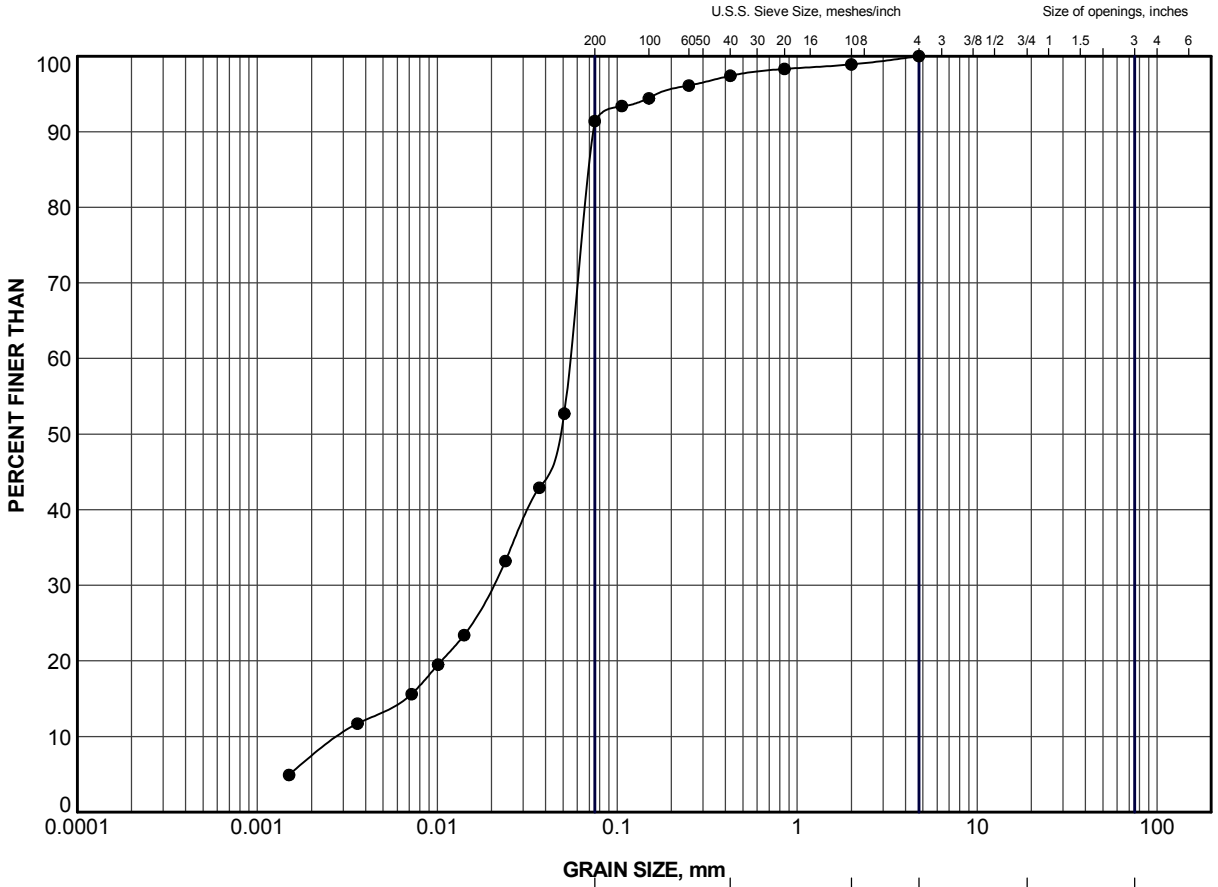
Geocres No.

HWY. 401	PROJECT NO. 12–1132–0076		DIST.
SUBM'D. BT	CHKD. NAG	DATE: Feb. 13/14	SITE: 19–405
DRAWN: WDF	CHKD.	APPD.	DWG. 3

PLOT DATE: January 26, 2015
FILENAME: N:\active\2012\1132 - Geo\1132-0000\12-1132-0076 DILLON-11 STRUCTURES-3911-E-0046\Drafting\AutoCAD files\1211320076-1001-001001.dwg

APPENDIX A

Laboratory Test Data



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	1	250.7

PROJECT

HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3030-11-00

TITLE

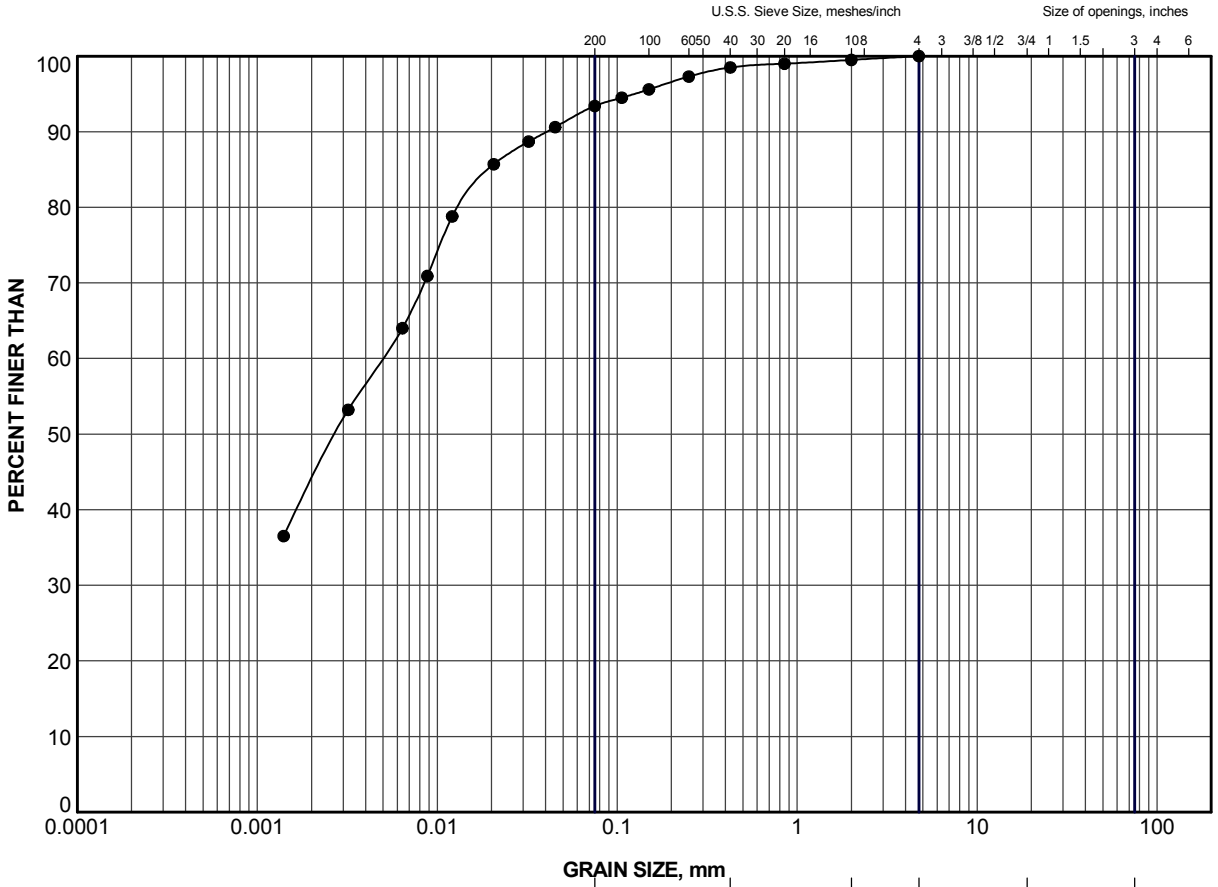
GRAIN SIZE DISTRIBUTION

FILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0076		FILE No. 1211320076-1001-F010A1
			SCALE N/A REV.
DRAWN	LMK	Mar 04/14	FIGURE A-1
CHECK			

LDN_MTO_GSD_GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	106	2	250.4

PROJECT

HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3030-11-00

TITLE

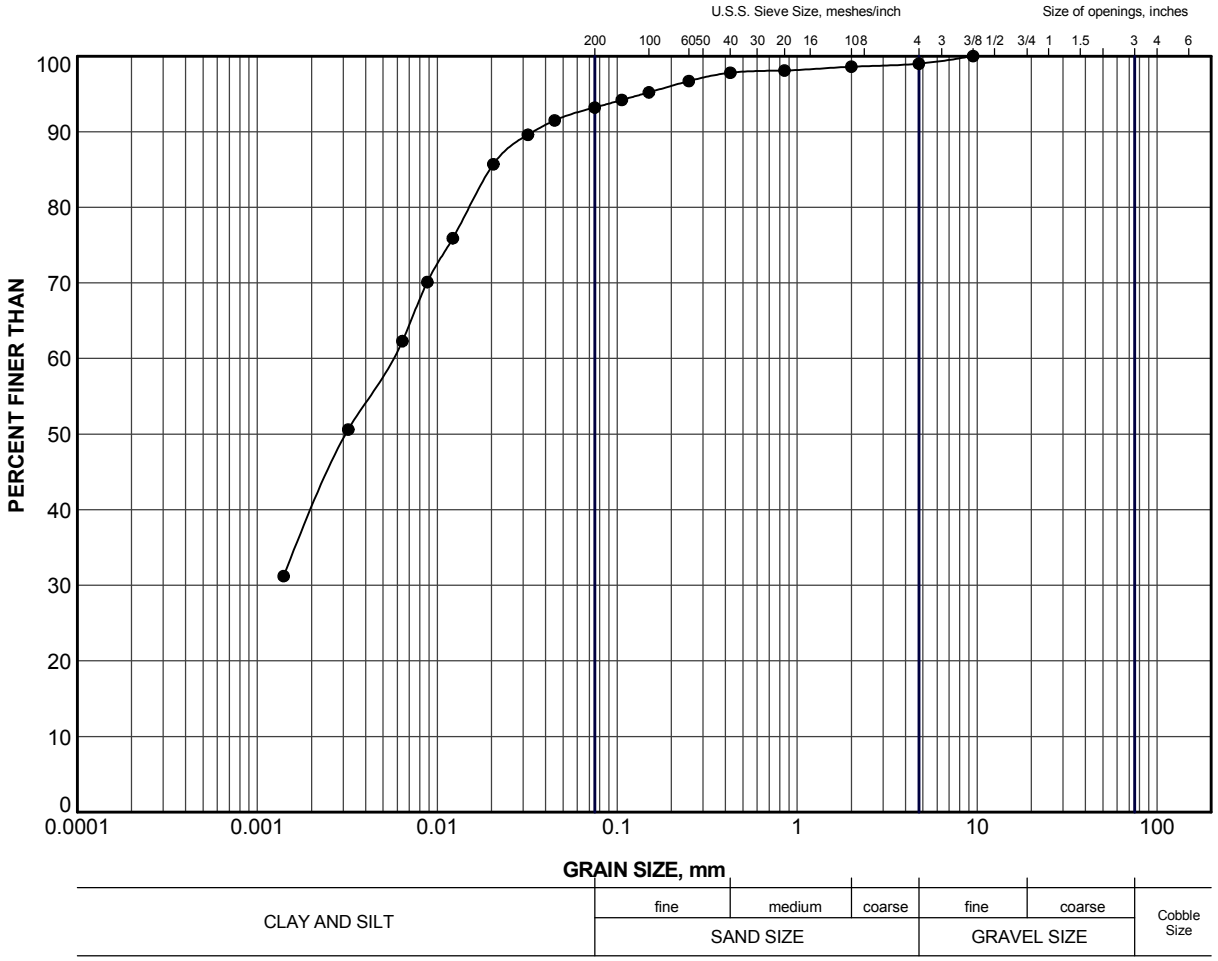
GRAIN SIZE DISTRIBUTION

SILTY CLAY

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0076		FILE No. 1211320076-1001-F010A2
			SCALE N/A REV.
DRAWN	LMK	Mar 04/14	FIGURE A-2
CHECK			

LDN_MTO_GSD_GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	4	248.4

PROJECT

HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3030-11-00

TITLE

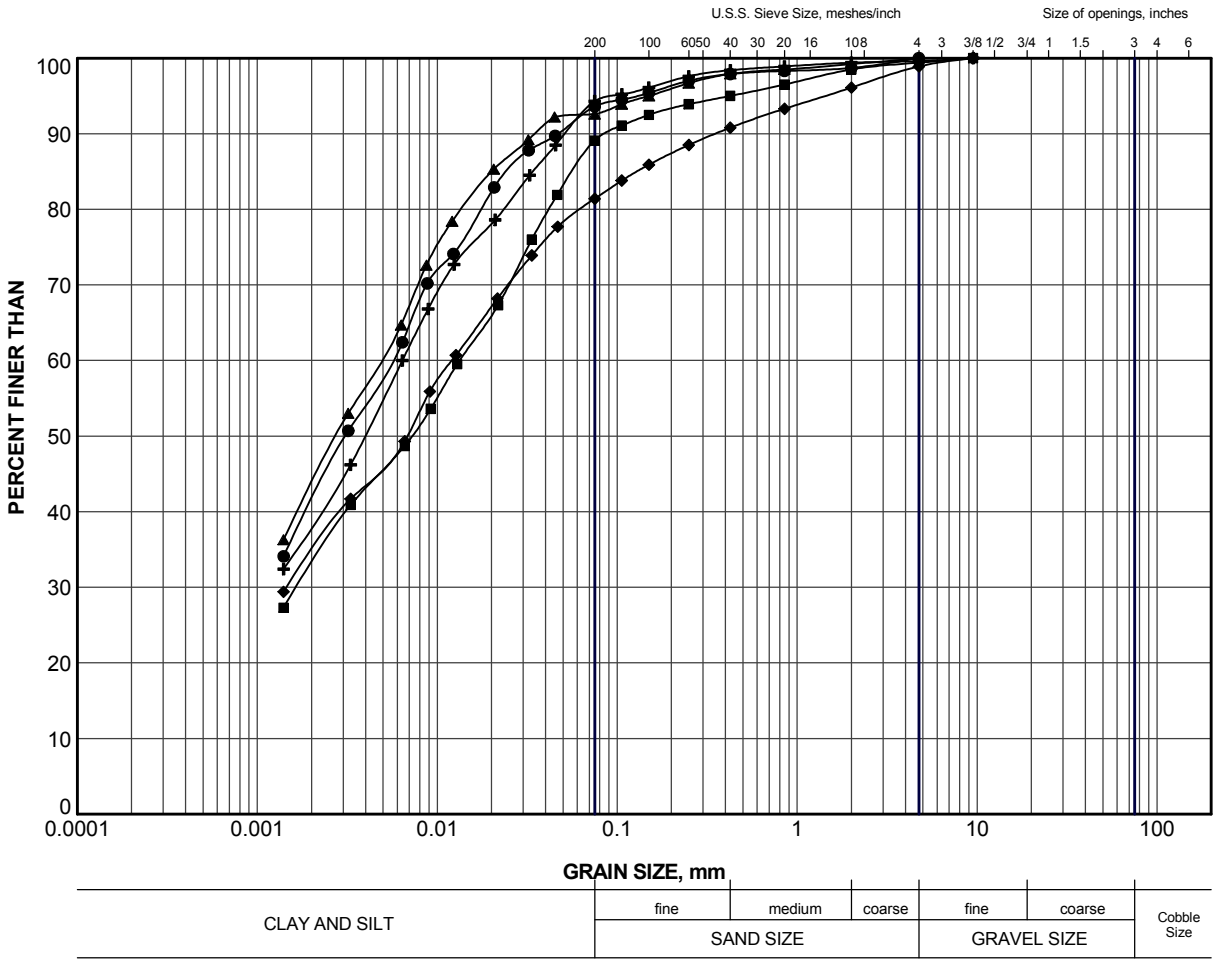
GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0076		FILE No. 1211320076-1001-F010A3
			SCALE N/A REV.
DRAWN	LMK	Mar 04/14	FIGURE A-3
CHECK			

LDN_MTO_GSD_GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	6	246.9
■	105	10	242.3
▲	106	3	249.7
✚	106	6	247.4
◆	106	9	244.3

PROJECT

HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3030-11-00

TITLE

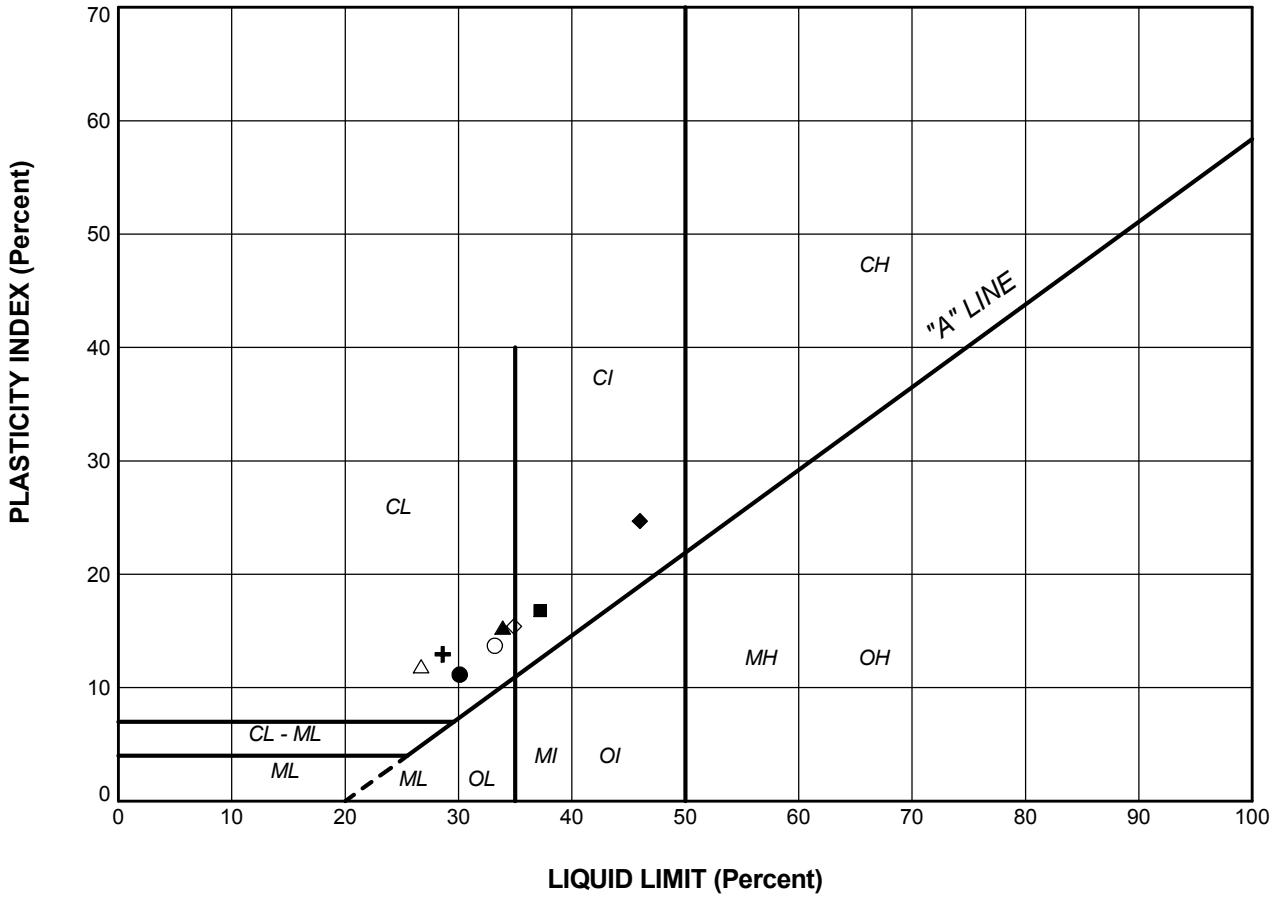
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0076		FILE No. 1211320076-1001-F010A4	
			SCALE N/A	REV.
DRAWN	LMK	Mar 04/14	FIGURE A-4	
CHECK				

LDN_MTO_GSD_GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	105	1	30.1	19.0	11.2
■	105	4	37.2	20.4	16.8
▲	105	6	33.9	18.6	15.3
✚	105	10	28.6	15.7	13.0
◆	106	2	46.0	21.3	24.7
◇	106	3	34.9	19.5	15.4
○	106	6	33.2	19.5	13.7
△	106	9	26.7	14.8	11.9

PROJECT
HIGHWAY 4 UNDERPASS REPLACEMENT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3030-11-00

TITLE
PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	12-1132-0076		FILE No. 1211320076-1001-F010A5
			SCALE N/A REV.
DRAWN	LMK	Mar 04/14	FIGURE A-5
CHECK			

LDN_MTO_PI GLDR_LDN.GDT

APPENDIX B

Record of Borehole Sheets, Geocres Report No. 40I14-134

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 1

METRIC

PROJECT	001-3225-3				
W.P.	476-89-00	LOCATION	N 4746750.113; E 404900.478	ORIGINATED BY	MR
DIST		HWY	HIGHWAY 4 & 401	BOREHOLE TYPE	POWER AUGER (HOLLOW STEM)	COMPILED BY	BG
DATUM	GEODETIC	DATE	March 15, 2004 - March 17, 2004	CHECKED BY	DJM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
252.27	GROUND SURFACE								
0.00 / 251.66	TOPSOIL, silty, Black								
0.61 / 250.59	FILL, clayey silt, trace sand, trace gravel, trace topsoil, Very stiff to stiff, Brown	1	SS	21					
1.68 / 250.14	TOPSOIL, silty, Stiff, Black	2	SS	14					
2.13 / 249.37	SILTY CLAY, trace sand, trace gravel, Stiff, Mottled brown and grey	3	SS	12					
2.90	SILTY CLAY (TILL), trace sand, trace gravel, Stiff to hard, Brown	4	SS	14					
		5	SS	32					
		6	SS	25		1	6	50	43
		7	SS	29					
		8	SS	26					
		9	SS	20					
7.47 / 244.80	CLAYEY SILT (TILL), trace sand, trace gravel, with cobbles and boulders, Hard, Grey	10	SS	32					
		11	SS	48					
10.06 / 242.21	CLAYEY SILT (TILL), trace sand, trace gravel, Hard, Grey	12	SS	29					
		13	SS	33					
		14	SS	31					

GROUND WATER CONDITIONS: ▽

ELEVATION SCALE: 252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 1

METRIC

PROJECT 001-3225-3
W.P. 476-89-00
LOCATION N 4746750.113; E 404900.478
ORIGINATED BY MR
DIST
HWY HIGHWAY 4 & 401
BOREHOLE TYPE POWER AUGER (HOLLOW STEM)
COMPILED BY BG
DATUM GEODETIC
DATE March 15, 2004 - March 17, 2004
CHECKED BY DJM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CLAYEY SILT (TILL), trace sand, trace gravel Hard Grey		15	SS	31		237		○		
			16	SS	41		236				
			17	SS	36		235		○		
232.15			18	SS	48		234				
20.12 / 231.54	SILT, trace sand, some clay, Dense Grey						233				
20.73	CLAYEY SILT (TILL), trace sand, trace gravel Hard Grey		19	SS	32		232		○		
229.10			20	SS	32	▽	231				
23.17 / 228.50	SILTY FINE SAND, Dense Grey						230				
23.77	CLAYEY SILT (TILL), trace sand, trace gravel Very stiff to hard, Grey		21	SS	25		229		○		
			22	SS	42		228				
			23	SS	38		227		○		
			24	SS	31		226				
							225				
							224				
							223				

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 1

METRIC

PROJECT 001-3225-3
W.P. 476-89-00
LOCATION N 4746750.113; E 404900.478
ORIGINATED BY MR
DIST
HWY HIGHWAY 4 & 401
BOREHOLE TYPE POWER AUGER (HOLLOW STEM)
COMPILED BY BG
DATUM GEODETIC
DATE March 15, 2004 - March 17, 2004
CHECKED BY DJM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CLAYEY SILT (TILL), trace sand, trace gravel Very stiff to hard, Grey						222				
221.33			25	SS	24				○		
30.94	**END OF BOREHOLE** Groundwater encountered in borehole at elev. 243.43m and 229.10m during drilling Mar 15 to 17, 2004										

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 2

METRIC

PROJECT	001-3225-3				
W.P.	476-89-00	LOCATION	N 4746664.26; E 404895.405	ORIGINATED BY	MR
DIST		HWY	HIGHWAY 4 & 401	BOREHOLE TYPE	POWER AUGER (MUD ROTARY) NW CASING	COMPILED BY	BG
DATUM	GEODETIC	DATE	March 18, 2004 - March 23, 2004	CHECKED BY	DJM

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
251.88	GROUND SURFACE
0.00 / 251.51	TOPSOIL, silty, Black
0.37 / 249.75	FILL, clayey silt, trace sand, trace gravel, with topsoil layers, Stiff, Brown
2.13 / 248.98	CLAYEY SILT, some sand, Stiff, Grey
2.90 / 248.22	SILT, trace clay, trace sand, Compact, Brown
3.66 / 246.70	CLAYEY SILT (TILL), trace sand, trace gravel, Very stiff, Grey
5.18 / 244.41	SILTY CLAY (TILL), trace sand, trace gravel, Very stiff, Grey
7.47 / 243.65	CLAYEY SILT (TILL), trace sand, trace gravel, Hard, Grey
8.23	BOULDER
8.38 / 242.13	SANDY SILT (TILL), trace clay, trace gravel, with cobbles, Dense, Grey
9.75	CLAYEY SILT (TILL), trace sand, trace gravel, Very stiff to hard, Grey

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	14
2	SS	13
3	SS	8
4	SS	16
5	SS	19
6	SS	19
7	SS	21
8	SS	17
9	SS	20
10	SS	32
11	SS	34
12	SS	35
13	SS	23
14	SS	31

GROUND WATER CONDITIONS: ∇

ELEVATION SCALE: 251, 250, 249, 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)	GR	SA	SI	CL
	2	11	57	30
	1	9	57	33
	2	13	50	35

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 2

METRIC

PROJECT 001-3225-3
W.P. 476-89-00
LOCATION N 4746664.26; E 404895.405
ORIGINATED BY MR
DIST
HWY HIGHWAY 4 & 401
BOREHOLE TYPE POWER AUGER (MUD ROTARY) NW CASING
COMPILED BY BG
DATUM GEODETIC
DATE March 18, 2004 - March 23, 2004
CHECKED BY DJM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
	CLAYEY SILT (TILL), trace sand, trace gravel Very stiff to hard Grey		15	SS	27						
							236				
			16	SS	25						
							235				
							234				
			17	SS	22						
							233				
			18	SS	26						
							232				
							231				
			19	SS	24						
							230				
			20	SS	20						
							229				
							228				
			21	SS	22						2 11 52 34
							227				
			22	SS	20						
							226				
225.36											
26.52	CLAYEY SILT (TILL), some sand, Hard Brown						225				
			23	SS	31						
223.84							224				
28.04	CLAYEY SILT (TILL), trace sand, trace gravel Very stiff hard, Grey										
			24	SS	27		223				
							222				

Continued Next Page

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

Golder Associates
London, Ontario

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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	CLAYEY SILT (TILL), trace sand, trace gravel Very stiff hard, Grey		25	SS	29		221				
			26	SS	32		220				
			27	SS	39		219				
			28	SS	31		218				
			29	SS	32		217				
			30	SS	32		216				
			31	SS	33		215				
			32	SS	31		214				
210.27							213				
41.61	BOULDER						212				
41.76	CLAYEY SILT (TILL), trace sand, trace gravel Hard, Grey		33	SS	34		211				
			34	SS	37		210				
							209				
							208				
							207				

Continued Next Page

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 2

METRIC

PROJECT 001-3225-3
W.P. 476-89-00
LOCATION N 4746664.26; E 404895.405
ORIGINATED BY MR
DIST
HWY HIGHWAY 4 & 401
BOREHOLE TYPE POWER AUGER (MUD ROTARY) NW CASING
COMPILED BY BG
DATUM GEODETIC
DATE March 18, 2004 - March 23, 2004
CHECKED BY DJM

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
	CLAYEY SILT (TILL), trace sand, trace gravel, Hard, Grey	35	SS	44		206							
205.40 / 46.48	SILTY CLAY, trace sand, Hard, Grey	36	SS	38		205				0	1	45	54
204.03 / 47.85	CLAYEY SILT (TILL), trace to some sand, with cobbles and silt layers, Hard, Grey	37	SS	31		204							
202.50 / 49.38	SAND AND GRAVEL				▽	203							
49.68	SANDY SILT (TILL), trace to some clay, with cobbles and silt layers, Very dense, Grey	38	SS	100/100mm		202				11	26	40	23
200.98 / 50.90	SAND, fine to medium, trace silt, Very dense, Grey				▽	201							
200.06 / 51.82	CLAYEY SILT (TILL), trace to some sand, with cobbles and silt layers, Hard, Grey	39	SS	125		200				0	90	10	0
51.97	**END OF BOREHOLE**												

Groundwater encountered in borehole at elev. 248.22m, 202.50m and 200.98m during drilling Mar 18 to 23, 2004

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

APPENDIX C

Record of Borehole Sheets, Geocres Report No. 40I14-52

Order No.: S-500-509/54, T-46 RACEY, MacCALLUM AND ASSOCIATES Limited — F. LUSK, Driller

Dated 31/12/54 (Day Month Year) — Foundation Engineering Division

Hole Begun 6/1/55

Hole Ended 8/1/55 — Engineering Data Sheet for Borehole: 1 — Helper

Job Name: SOIL INVESTIGATIONS, PROP. WESTMINSTER TOWNSHIP BRIDGE #19 — K. TUBBESING, Engineer

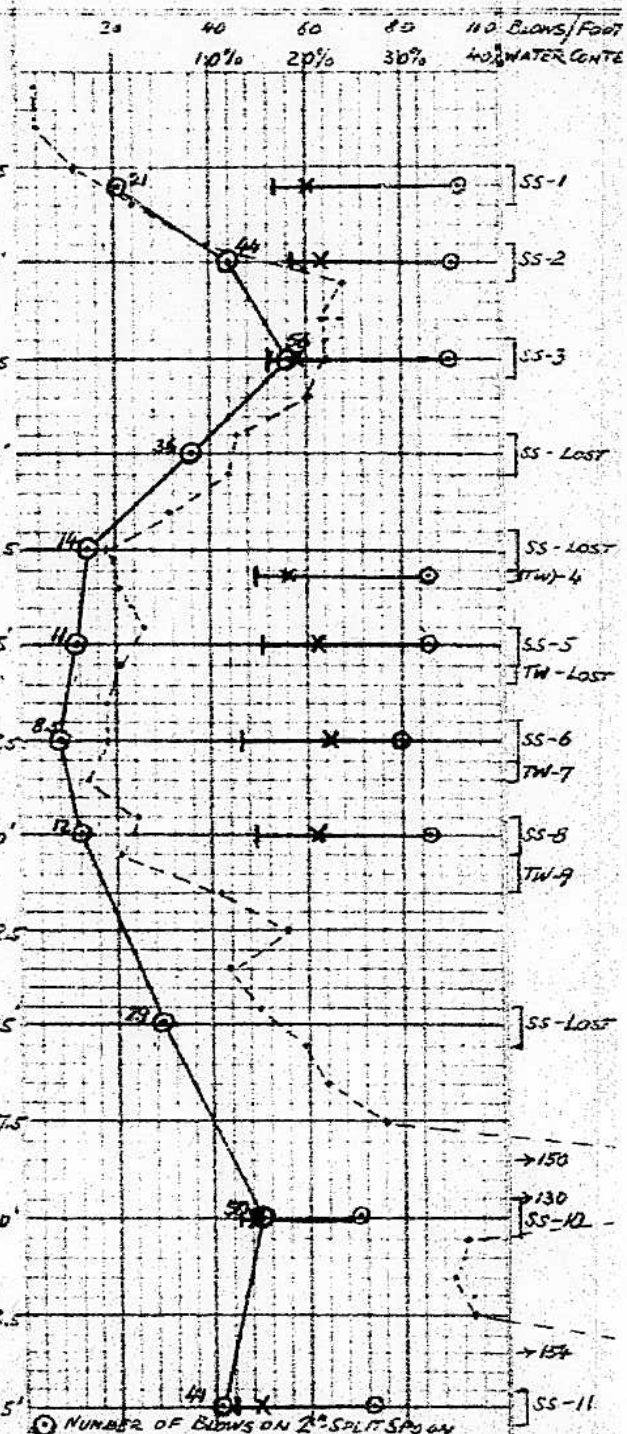
Job Located: OVERHEAD CROSSING AT JUNCTION OF HIGHWAYS NO. 4 AND NO. 401

Hole Located: SEE ATTACHED PORTION OF OHD - PLAN

Hole Elevation: 823.5' Datum: M.S.L. — 8/1/55 (Date Month Year)

251.00m

Depth	El.	Thickness	W	Symbol	Narrative
0	823.5				GRADED GROUND SURFACE
2.5'	821.0				STIFF SLIGHTLY SANDY, BROWN CLAY. WATER LEVEL IN PIPE JAN. 7TH, 55 (DEPTH OF BOREHOLE 11.5').
					EXTREMELY STIFF, BROWN SILTY CLAY, OF MED. PLASTICITY
					HARD SILTY CLAY, LIGHT BROWN OF MEDIUM PLASTICITY
					STIFF SILTY BROWN CLAY OF MEDIUM PLASTICITY
					STIFF SILTY BROWN CLAY OF MEDIUM PLASTICITY
					MEDIUM, BROWNISH-GREY, SLIGHTLY SILTY CLAY OF LOW TO MEDIUM PLASTICITY
					STIFF SANDY & SILTY CLAY OF MEDIUM PLASTICITY
					VERY STIFF CLAY
					EXTREMELY STIFF GREY CLAY WITH SOME SAND & SMALL GRAVEL CLAY OF LOW PLASTICITY
					GREY CLAY + SANDY GRAVEL EXTREMELY STIFF, LOW PLASTICITY
35.5'	788.0				END OF HOLE (35.5')



ABBREVIATIONS:
W. WATER; B. BAILED; M. MOIST; H. HUMID
D. DRY
S.S., SPLIT SPOON, TW. THIN WALL SHELBY

Order No. S-500-319/54/T-46 RACEY, MacCALLUM AND ASSOCIATES Driller F. LUSK

Dated 31/12/54 Limited

Day Month Year Foundation Engineering Division

Hole Began 18/1/55

Hole Ended 19/1/55 Engineering Data Sheet for Borehole: 3 Helper

Job Name: SOIL INVESTIGATIONS, PROP. WESTMINSTER THR. BRIDGE #19 K. TUBBESING

Job Located: OVERHEAD CROSSING AT JUNCTION HIGHWAYS NO. 4 AND NO. 401 Checked by

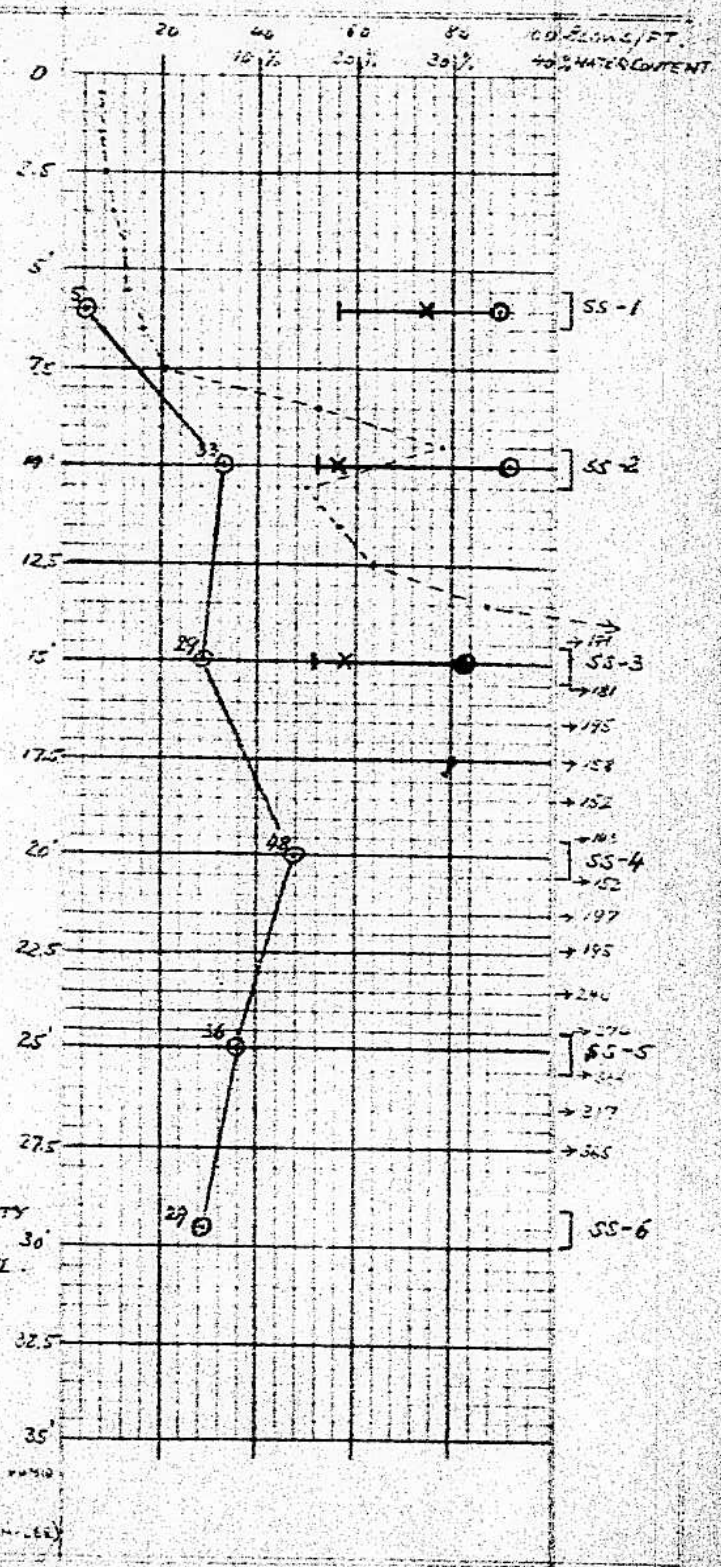
Hole Location: SEE ATTACHED PORTION OF OHD. PLAN

Hole Elevation: 823.4 Datum: M.S.L. 22/1/55

Date

251.97m

Depth	El.	Thickness	W	Symbol	Narrative
0	823.4				SURFACE
2.9	820.5		▽		WATER LEVEL IN HOLE, JAN. 22, 55.
					SOFT, BROWN SILTY CLAY OF MEDIUM PLASTICITY
					EXTREMELY STIFF CALCAREOUS, SANDY AND SILTY CLAY OF MEDIUM PLASTICITY
					VERY STIFF CALCAREOUS, SILTY CLAY OF MEDIUM PLASTICITY
					DENSE SLIGHTLY CALCAREOUS, MEDIUM TO COARSE PURE SILT, GREY.
					EXTREMELY STIFF CALCAREOUS, SLIGHTLY SANDY AND SILTY CLAY, GREY.
					VERY STIFF VERY CALCAREOUS, SANDY AND SILTY CLAY WITH SOME ANGULAR GRAVEL.
30'	783.4				END OF HOLE.



ABBREVIATIONS

W WATER; S SATURATED; M MOIST; H HUMID; D DRY

S S; SPLIT SPOON, TW THIN WALL (SAMPLER)

Order No.: S-500-509/54, T-46
Dated 31/12/54
Day Month Year

RACEY, MacCALLUM AND ASSOCIATES
Limited
Foundation Engineering Division

A. McLADDEN
Driller

Hole Begun 17/1/55
Hole Ended 19/1/55

Engineering Data Sheet for Borehole: 7

M. CHEVRIER
Helper

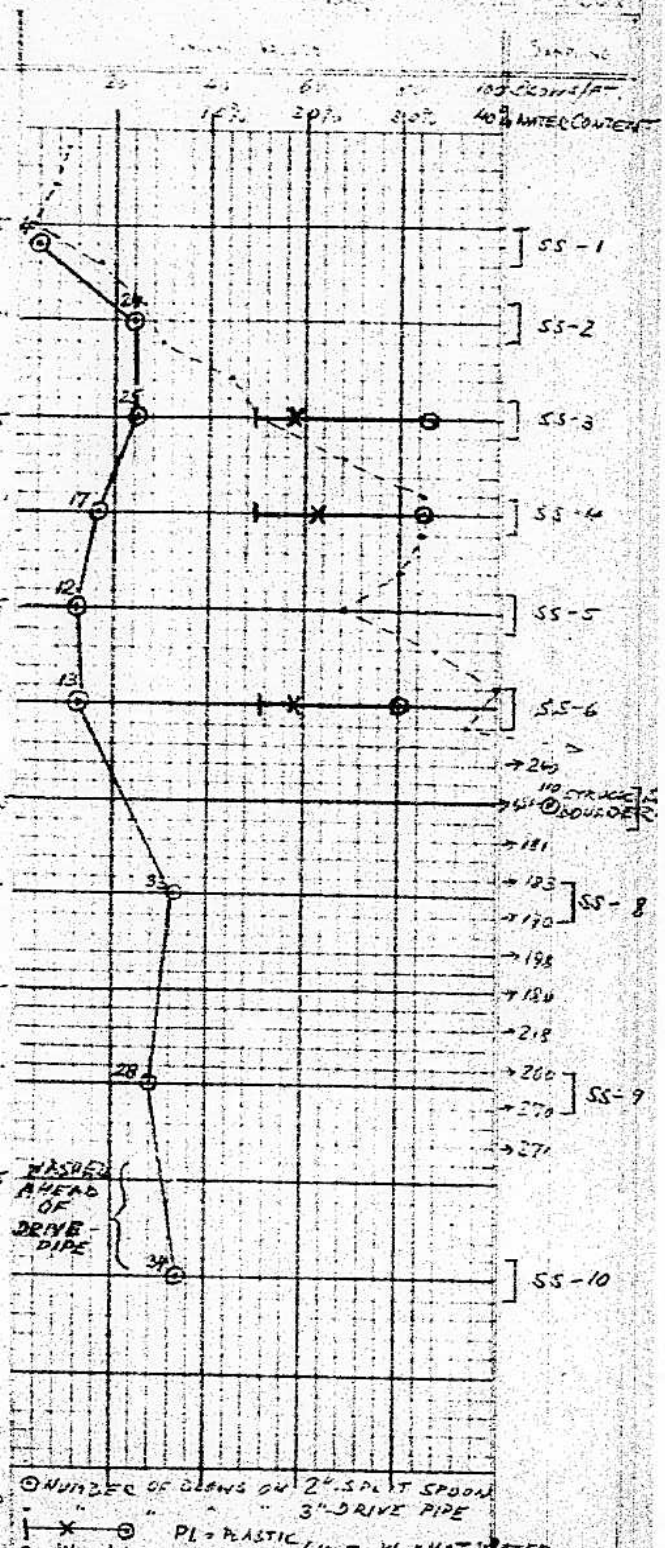
Job Name: SOIL INVESTIGATIONS, PROP. WESTMINSTER TWP. BRIDGE #19
Job Located: OVERHEAD CROSSING AT JUNCTION HIGHWAYS NO. 4 AND NO. 401
Hole Located: NEAR EDGE OF CREEK (SEE ATTACHED O.D.R. - PLAN)
Hole Elevation: 819.1' Datum: M.S.L.
~2.5' ABOVE BED OF CREEK

K. TURBERVILLE
Checked by

24/1/55
Date

249.66m

Depth	El.	Description
0	819.1	SURFACE
3.1'	816.0	SOFT BROWN SILTY CLAY
		WATER LEVEL IN HOLE, JAN. 22, 55.
		VERY STIFF VERY CALCAREOUS, BROWN CLAY
		VERY STIFF, BROWN SILTY CLAY OF MEDIUM PLASTICITY
		STIFF, CALCAREOUS, SILTY BROWN CLAY OF MEDIUM PLASTICITY
		STIFF, VERY CALCAREOUS, SILTY CLAY
		STIFF, CALCAREOUS, BROWN SILTY CLAY
		STIFF, GREY SILTY CLAY, SOME BOULDERS
		VERY STIFF VERY CALCAREOUS, GREY SILTY CLAY WITH SOME ANGULAR GRAVEL
		VERY STIFF, VERY CALCAREOUS, GREY SILTY CLAY WITH SOME GRAVEL
		EXTREMELY STIFF VERY CALCAREOUS, GREY SILTY CLAY
31.5	787.6	END OF HOLE.



Abbreviations
* Water: S. Saturated, M. Moist, H. Humid
D. Dry
S.S.: Split Spoon, T.W.: Thin Wall Sampler

Order No. S-500-SUM54T-46 F. LUCIK
Dated 31/12/54
Date Commenced 5/1/55
Date Completed 15/1/55 Page 9
Job Name: SOIL INVESTIGATIONS, PARK WESTMINSTER TWR. BRIDGE F19 K. TURCZYNOWICZ
Job Location: OVERHEAD CROSSING AT JUNCTION HIGHWAYS NO. 4 AND NO. 401
Hole Location: SEE ATTACHED DRAWING OF OVD-PLAN
Hole Elevation: 822' Datum: M.S.L. 15/1/55

250.55m

SURFACE

WATER LEVEL IN HOLE, JAN. 22, 55

SOFT BROWN CLAY

STIFF, MEDIUM-BROWN CLAY

VERY STIFF
SLIGHTLY SANDY AND SILTY, CALCAREOUS CLAY OF MEDIUM PLASTICITY

VERY CALCAREOUS, BROWN CLAY, SLIGHTLY SANDY & SILTY

VERY STIFF,
VERY CALCAREOUS, BROWN, SILTY CLAY, SOME ANGUL. GRAVEL

VERY STIFF
VERY CALCAREOUS, MEDIUM-BROWN CLAY WITH SOME SAND AND GRAVEL

VERY STIFF
VERY CALCAREOUS CLAY, MANY PEBBLES UP TO 1½" DIA.

VERY STIFF
CALCAREOUS, BROWNISH-GREY SILTY CLAY WITH SILT LAYERS AND SOME GRAVEL

VERY STIFF,
SILTY GREY CLAY CHANGING TO DENSE, SILTY FINE SAND WITH GRAVEL (LAYER)

VERY STIFF
CLAY

VERY STIFF
VERY CALCAREOUS, SILTY CLAY WITH FINE SAND & SMALL GRAVEL

VERY STIFF, CALCAREOUS, SILTY CLAY

VERY STIFF
VERY CALCAREOUS, GREY CLAY WITH SOME FINE ANGULAR GRAVEL

CHANGING TO:

GREYISH BROWN CLAYEY SILT
(OBSERVATION OF HIGH WATER)

WASHED AHEAD OF DRIVE PIPE

VERY CALCAREOUS, BROWN TO COARSE, GREY SILT.

END OF HOLE

SS-LOST
SS-1
SS-2
SS-3
SS-4
SS-5
SS-6
SS-7
SS-8
SS-LOST
SS-10
SS-LOST
SS-11
ROD-12 DIA.

Numbers on scale on 2" plot are on 3 cycle semi-log paper

PL = PLASTIC LIMIT, W = NAT. WATER CONTENT, LL = LIQUID LIMIT

APPENDIX D

Site Photographs

Photograph 1: Existing north embankment, east side.

Photograph 2: Existing deck, looking south.

Photograph 3: Proposed north abutment location, looking southeast.

Photograph 4: Proposed south abutment location, looking northeast.

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\r01 highway 4 site 19-405\1211320076-1001-r01 may 21 15 (draft) app d-site photos.docx

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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