July 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

High Fill Ramp Embankments
Highway 401/Highway 40 Interchange Reconfiguration
Chatham-Kent, Ontario
GWP 3093-09-00
Ministry of Transportation, Ontario - West Region

Submitted to:
Mr. Kevin Welker, P.Eng., Partner
Dillon Consulting Limited
130 Dufferin Avenue Suite 1400
London, Ontario, N6A 5R2

REPORT

Report Number: 13-1132-0111-1000-R06

Geocres No. 40J8-61
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PART A

FOUNDATION INVESTIGATION REPORT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
CHATHAM-KENT, ONTARIO
GWP 3093-09-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) 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 detail design work for GWP 3093-09-00. The project involves the detail design for the reconfiguration of the Highway 401 and Highway 40 (Communication Road) interchange as well as the realignment of Pinehurst Line and reconstruction of the Highway 401 eastbound lanes. This report addresses the construction of high fill embankments for new and/or realigned ramps associated with the interchange reconfiguration.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed high fill embankments 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’s proposal P3-1132-0111 dated December 12, 2013. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated February 27, 2014.

Dillon provided Golder with preliminary drawings for this project in digital format. In addition, the Preliminary Design Report (PDR) and Design Build Ready Report (DBRR) package was provided by the MTO.

2.0 SITE DESCRIPTION

2.1 General

The Highway 401/Highway 40 interchange is located in the Municipality of Chatham-Kent, 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 east-west. Highway 40 is oriented in a generally northwest-southeast direction in the area of the site. For the purposes of this report, Highway 40 is assumed to be oriented in a north-south direction and the project site is presumed to be divided into the four quadrants formed by the two highways.

In the northwest quadrant, the area of the proposed N-W Ramp is within the McGregor Creek floodplain where the ground surface elevation ranges from about 180.0 to 183.8 metres. From Station 10+000 to 10+140, the N-W Ramp embankment will be 4.5 to 9.0 metres in height. Between about Stations 10+000 and 10+075, the ramp will form part of the approach embankment to the proposed N-W Ramp bridge over McGregor Creek.

The high fill sections of the S-W and E-N/S Ramps, also within the McGregor Creek floodplain, will be from Station 10+075 to 10+210, and Station 10+230 to 10+430, respectively, and will be up to about 9.1 metres in height. The E-N/S embankment will be restrained on the north side by a structural retaining wall. Foundation recommendations for the design of the retaining wall have been addressed under separate cover for this assignment (Geocres Report No. 40J8-62). The S-W and E-N/S Ramps coincide with the existing Highway 40 embankment along the east side of the highway and will include portions of the existing E-N/S Ramp embankment. The ground surface elevation in the area of the S-W and E-N/S Ramp high fill embankments ranges from about 180.5 to 181.5 metres, except where it coincides with the existing embankments.

The proposed high fill section of the S-E Ramp, in the southeast quadrant, is from Station 10+070 to 10+140 and will be 4.5 to 5.0 metres in height. The proposed S-E Ramp coincides with the existing Highway 40 embankment along the east side of the highway and, at about Stations 10+050 and 10+150, will include portions of the existing W-N/S Ramp embankment. The ground surface elevation in the area of the S-E Ramp high fill embankment is about 183.0 to 183.4 metres.

The proposed W-N/S and N-E Ramp alignments are within an existing agricultural field in the southwest quadrant and coincide with the widened Highway 40 embankment along the west side of the highway. The N-E Ramp will form part of the Highway 401/Highway 40 Underpass approach embankment. High fill embankments are required for the W-N/S and N-E Ramps between Stations 10+280 and 10+500, and 10+000 to 10+160, respectively, and will be up to 9.0 metres in height. The ground surface elevation in the area of the W-N/S and N-E Ramp high fill embankments ranges from about 182.2 to 183.5 metres.

2.2 Site Geology

This project lies within the physiographic region of southwestern Ontario known as the Bothwell Sand Plain which formed a delta from a geologic precursor of the Thames River where it discharged into the former glacial

Lake Warren. The Bothwell Sand Plain primarily consists of a thin layer of sand, approximately 1 metre thick, over the clay floor.[1]

The quaternary geology mapping indicates that the surficial materials consist primarily of glaciolacustrine deposits of clayey silt and silty sand overlain by glaciolacustrine silty sand and sand.[2] Along McGregor Creek and its tributary, Lucas Drain, these deposits are overlain by modern alluvium or young stream deposits of clay, silt, sand and organic soils ("muck"). Based on geologic mapping the underlying bedrock surface may be found at about 25 metres below the ground surface, or near elevation 160 metres.[3] The rock formation is mapped and described as black bituminous shale of the Kettle Point Formation of the Port Lambton Group, upper Devonian age.[4]

The project area is also about 2 kilometres north and 6 kilometres west of an area mapped as "till moraine". Although the mapping provides a general indicator of the geologic conditions of the site, these maps only address the most recent phase of the region's glacial geology based on near-surface materials and may not characterize the geologic complexity of the site at greater depths. In southwestern Ontario, the most significant prehistoric glacial features are associated with the last advance and retreat of ice through the area. As the ice receded from the region, a number of moraines and lakes were formed near the retreating ice front. In some areas, such as Windsor and Wallaceburg, Ontario, the clayey silt or silty clay deposits have a grain size distribution consistent with that of a cohesive glacial till although the density and strength of the materials are not consistent with deposition below a grounded ice sheet as commonly assumed for materials described as glacial till. In the Windsor area, much of the soils described as glacial till were likely deposited from the underside of floating ice through a shallow water depth as a diamict (broadly graded mud) and, therefore, the soil carried little or no weight of the overlying ice. East of Windsor, toward the Chatham area, some areas of the ice sheet may have been grounded and produced hard cohesive glacial till, while in other areas the ice may have been floating or partially floating which has resulted in complex conditions in some areas. Near moraines geologic conditions can be especially complex because of highly localized outwash (sand and gravel) deposits, silt and clay deposited in local ice-proximal lakes and ponds, and comparatively short duration re-advances and retreats of the former ice sheets.

2.3 Previous Site Construction History

This site was extensively altered in the early 1960s when the Highway 401 and Highway 40 interchange was constructed. Prior to construction of the interchange, McGregor Creek and Lucas Drain both existed along previous channel alignments. Aerial photographs from the mid-1950s also indicate that McGregor Creek meandered in the project area producing at least two former and subsequently buried meander channels with one on each of the south and north sides of the present channel. During construction of the interchange, both McGregor Creek and Lucas Drain were realigned to their present positions. The available evidence also

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

[2] Kelly, R.I., 1991: Quaternary geology of the Chatham-Wheatley area; Ontario Geological Survey, Open File Map 163, scale 1:50 000.

[3] Sado, E.V. and Faught, R.B. 1981: Drift Thickness of Chatham Area, Southern Ontario; Ontario Geological Survey Preliminary Map P.2453, Drift Thickness Series. Scale 1:50 000.

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

indicates that the area between the existing highway interchange ramps in the northeast quadrant of the site and the current McGregor Creek channel was in-filled over former low-lying wet areas. In some areas, it also appears that the organic matter may not have been fully removed prior to the in-filling. The degree to which localized soft and wet areas and organic matter was or was not removed beneath the existing highway and ramp embankments remains unknown. Figure 2 illustrates the approximate locations of the former McGregor Creek and Lucas Drain channels that existed within the 5 to 7 years preceding interchange construction as well as the former meander channel. The former meander channel was likely cut-off from flow in McGregor Creek by flood flows at some point in its history and the former channel was filled in with sediments and agricultural reworking of the area.

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between May 6 and June 27, 2014 during which time twenty one (21) boreholes, numbered 605 to 625, were drilled at the locations of the proposed high fill ramp embankments. The boreholes were supplemented with four boreholes, numbered 601 to 604, advanced for the E-N/S Ramp retaining wall portion of this project (Geocres Report No. 40J8-62) and borehole 403 advanced for the McGregor Creek bridge structure (Geocres Report No. 40J8-63). The borehole locations 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		
601	4 693 861	338 356	180.44	14.17
602	4 693 846	338 407	180.40	11.13
603	4 693 847	338 294	180.57	16.92
604	4 693 800	338 254	180.71	19.96
605	4 693 783	338 260	181.54	5.79
606	4 693 848	338 326	180.49	8.08
607	4 693 662	338 158	183.52	6.25
608	4 693 726	338 145	183.31	6.25
609	4 693 640	338 104	183.54	6.55
610	4 693 630	338 064	183.50	5.79
611	4 693 838	338 376	180.34	8.08
612	4 693 803	338 290	180.64	11.13
613	4 693 809	338 369	180.73	5.79
614	4 693 824	338 327	180.43	9.60
615	4 693 587	338 474	181.98	6.55
616	4 693 568	338 435	182.85	11.13
617	4 693 512	338 358	182.90	8.08
618	4 693 491	338 296	183.31	6.55
619	4 693 492	338 399	182.25	9.60
620	4 693 508	338 453	182.48	11.13
621	4 693 610	338 418	183.08	9.60
622	4 693 629	338 375	183.06	6.55
623	4 693 550	338 380	182.72	9.60
624	4 693 594	338 544	183.08	5.03

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
625	4 693 658	338 527	182.54	5.03
403	4 693 779	338 163	180.85	8.08

The investigation was carried out using track mounted drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.76 and 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 D1586. Thin-walled Shelby tube samples were obtained in the boreholes at selected depths in accordance with ASTM D1587. In-situ vane testing was carried out in accordance with ASTM D2573 to determine the undrained shear strength of softer cohesive soils encountered in the boreholes. Piezo-Cone Penetration Testing (CPT) was carried out adjacent to boreholes 603 and 617.

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 an initial 150 millimetres of penetration. In cases where it was not possible to achieve a full 450 millimetres of drive, a penetration resistance representing the number of blows to drive the sampler is recorded on the Record of Borehole. The penetration resistance obtained in the first 150 millimetres is normally neglected unless the sampler could only be driven 150 millimetres or less, in which case SPT testing was terminated after 100 blows. The results of the SPT testing 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 glacial till deposits as discussed in the text of this report.

The boreholes were terminated between 5.0 and 20.0 metres below the existing ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. Groundwater observation piezometers were installed in boreholes 602, 604, 606, 609, 616, 619, 621 and 624 as indicated on the corresponding Record of Borehole sheets. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder staff 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 and Mississauga laboratories 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. Consolidation testing was carried out on selected Shelby tube samples. The results of the testing are shown on the Record of Borehole sheets and in Appendices A and B.

The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.

The CPTs consisted of pushing a 35 millimetre outside diameter cone from a known depth to refusal at an approximate rate of 20 millimetres per second by the drill rig in accordance with ASTM D5778. Electronic sensing elements connected to the CPT probe continuously measure tip resistance, local side friction and pore water pressures with depth. The measurements are shown on the Record of Cone Penetration Test sheets and Figures 3 and 4.

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 given on the attached Record of Borehole sheets following the text of this report and in Appendices A and B. 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. For the purposes of this report, the project site is presumed to be divided into the four quadrants formed by the two highways.

The boreholes drilled at the site generally encountered surficial topsoil and/or fill materials overlying a deposit of silty clay to clayey silt glacial till. In the northeast quadrant up to 4 metres of fill materials was encountered and the cohesive glacial till was underlain by a deposit of sandy silt till interbed with layers of granular soils.

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.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawings 1 to 4. Detailed descriptions of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and are summarized in subsequent report sections.

Boreholes advanced at the location of the Highway 40/Lucas Drain bridge location identified conditions that are different than other areas of the overall interchange site. The three deep boreholes at the Highway 40/Lucas Drain bridge location did not identify the granular layers that dominate the subsurface stratigraphy below about elevation 172 metres at the locations of the Highway 40/McGregor Creek structure, Highway 401/Highway 40 structure, E-N/S Ramp and retaining wall, or the Highway 401/Lucas Drain structure. A distinctly different layer of firm to stiff silty clay was identified at and below approximately elevation 167 metres. These conditions are described in more detail below.

4.1.1 Northwest Quadrant – N-W Ramp

Boreholes 403 and 607 to 610 were advanced in the area of the N-W Ramp. The interpreted stratigraphic profile is shown on Drawing 4.

Topsoil

Between 90 and 270 millimetres of topsoil was encountered at the ground surface in boreholes 607 to 610 and 403.

Silty Sand

A 0.4 metre thick layer of silty sand was encountered beneath the topsoil in borehole 608 at elevation 183.2 metres.

Silt

A 0.2 metre thick layer of loose silt was encountered beneath the silty sand in borehole 608 at elevation 182.9 metres. A single measured N value from SPT testing carried out in the silt layer was 7 blows per 0.3 metres.

Clayey Silt

Firm to stiff clayey silt was encountered beneath the topsoil in boreholes 607 and 610 and beneath the silt in borehole 608 between elevations 182.6 and 183.4 metres. The clayey silt was between 0.5 and 1.3 metre thick. Measured N values from the clayey silt ranged from 7 to 11 blows per 0.3 metres. Samples of the clayey silt had water contents of 17 and 19 per cent. An Atterberg limits determination carried out on a sample of clayey silt yielded liquid and plastic limits of 33 and 18 per cent, respectively, and a plasticity index of 15 per cent, indicating low plasticity. The results of the Atterberg limits determination are provided on Figure A-12. Grain size distribution curves for samples of the clayey silt are shown on Figure A-2.

Silty Clay

Stiff silty clay was encountered beneath the topsoil in borehole 609 and beneath the clayey silt in borehole 610 at elevations 183.4 and 182.1 metres, respectively, and was 2.0 and 0.8 metres thick. Measured N values from the clayey silt ranged from 8 to 14 blows per 0.3 metres. A sample of the silty clay had a water content of 18 per cent, liquid and plastic limits of 40 and 22 per cent, respectively, and a plasticity index of 18 per cent, indicating intermediate plasticity. The results of the Atterberg limits determination are provided on Figure A-12. A grain size distribution curve for a sample of the silty clay is shown on Figure A-3.

Clayey Silt to Silty Clay Glacial Till

A deposit of firm to very stiff cohesive glacial till was encountered to the termination depths of boreholes 607 through 610 and 403. The cohesive till was encountered beneath the clayey silt in boreholes 607 and 608, beneath the silty clay in boreholes 609 and 610, and beneath the topsoil in borehole 403 between elevations 180.7 and 182.3 metres. The boreholes penetrated the cohesive till between 3.7 and 8.0 metres. Based on the Atterberg limits of samples of the cohesive till as described below, the cohesive till was generally classified as clayey silt. A 3.1 metre thick layer of cohesive till in borehole 403 was classified as silty clay. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the glacial till deposit due to its depositional history.

Measured N values in the cohesive till ranged from 5 to 27 blows per 0.3 metres. Samples of the clayey silt till had water contents of 13 to 18 per cent. Seven Atterberg limits determinations carried out on samples of the clayey silt till yielded liquid limits of 32 to 34 per cent, plastic limits of 17 to 20 per cent, and plasticity indices of 12 to 17 per cent, indicating low plasticity. A single Atterberg limits determination carried out on a sample of the silty clay till yielded liquid and plastic limits of 35 and 18 per cent, respectively, and a plasticity index of 17 per cent, indicating intermediate plasticity. The results of the Atterberg limits determinations are provided on Figure A-12. Grain size distribution curves for samples of the silty clay till and clayey silt till are shown on Figures A-4 and A-5, respectively.

4.1.2 Northeast Quadrant – E-N/S and S-W Ramps

Boreholes 601 to 606 and 611 to 614 were advanced in the northeast quadrant. The interpreted stratigraphic profile along the E-N/S and S-W Ramps are shown on Drawing 2.

Topsoil

Between 30 and 460 millimetres of topsoil was encountered at the ground surface in boreholes 601 to 606 and 612 to 614.

Fill

Fill materials were encountered in each of the boreholes advanced in the northeast quadrant of the site, beneath the topsoil in boreholes 601 to 606 and 612 to 614 and from the ground surface in borehole 611. The fill materials were between 2.4 and 4.0 metres thick and extended to elevations between 176.2 and 177.7 metres. The fill typically varied in composition from sand and gravel, silty clay, clayey silt and silt. The lower 0.6 to 2.1 metres of fill materials consisted of dark grey to black sand and gravel to sand with pockets clayey silt and varying amounts of wood, organic materials and other deleterious debris. Based on the site history, this uncontrolled fill material may have been placed during historic rerouting of McGregor Creek and highway interchange construction.

Measured N values in the fill materials ranged from 1 to 16 blows per 0.3 metres. Samples of the fill materials had water contents of between 11 and 33 per cent. Atterberg limits determinations were carried out on four samples of the cohesive fill material, the results of which are provided on Figure A-11. The samples of cohesive fill tested had liquid and plastic limits ranging from 27 to 40 per cent and 16 to 19 per cent, respectively, and plasticity indices of 8 to 21 per cent, indicating low to intermediate plasticity. Grain size distribution curves for samples of the fill materials are shown on Figure A-1.

Clayey Silt Glacial Till

A deposit of firm to very stiff clayey silt glacial till was encountered beneath the fill materials in each of the boreholes drilled in the northeast quadrant of the site between elevations 176.2 and 177.7 metres. Where fully penetrated in boreholes 601 to 604, 611, 612 and 614, the clayey silt till was between 2.8 and 6.1 metres thick. Boreholes 605, 606 and 613 were terminated in the clayey silt till after penetrating the layer for between 1.8 and 4.8 metres. Layers of sand and silty clay till were encountered within the clayey silt till in boreholes 601 and

606, respectively, as described below. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the clayey silt glacial till.

The clayey silt till had measured N values ranging from 6 to 30 blows per 0.3 metres. Samples of the clayey silt till had water contents ranging from 14 to 19 per cent. Nine Atterberg limits determinations carried out on samples of the clayey silt till, the results of which are provided on Figure A-12, yielded liquid limits ranging from 29 to 33 per cent, plastic limits ranging from 16 to 18 per cent, and plasticity indices of 12 to 15 per cent, indicating low plasticity. Grain size distribution curves for samples of the clayey silt till are shown on Figure A-5.

Consolidation testing carried out on a sample of the clayey silt till indicated the following properties. Results of the consolidation testing are provided in Appendix B.

Borehole	Sample	Depth (m)	Effective Overburden Pressure (kPa)	Initial Void Ratio	Recompression Index, C_R	Compression Index, C_C	Preconsolidation Pressure (kPa)
604	7	5.64	69	0.53	0.014	0.136	249

Silty Clay Glacial Till

As noted above, a 1.8 metre thick layer of stiff to very stiff silty clay glacial till was encountered within the clayey silt till in borehole 606 at elevation 175.3 metres. Measured N values in the silty clay till were 13 and 20 blows per 0.3 metres. Cobbles and boulders should be anticipated within the silty clay glacial till.

Clayey Silt

Layers of stiff to hard clayey silt, 2.1 and 0.8 metres thick respectively, were encountered beneath the clayey silt till in boreholes 603 and 604 at elevations 173.7 and 172.5 metres. Measured N values in the clayey silt ranged from 12 to 34 blows per 0.3 metres.

Sandy Silt Glacial Till

Underlying the clayey silt till in boreholes 601, 602, 611 and 614, the clayey silt in boreholes 603 and 604, and a layer of silty sand in borehole 612, a deposit of compact to very dense sandy silt glacial till was encountered. The sandy silt till was between 0.3 and 7.8 metres thick in boreholes 601 to 604 and 614, and was encountered between elevations 170.6 and 173.9 metres. Boreholes 611 and 612 were terminated after penetrating the sandy silt till for 1.6 and 1.1 metres, respectively. The sandy silt till in boreholes 603 and 604 was interbed with layers of sand and sand and gravel, as described below. The sandy silt till varied in gradation from silty sand to sand and silt, with varying amounts of gravel and clay. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the glacial till deposits.

Measured N values from the sandy silt till ranged from 18 to greater than 100 blows per 0.3 metres. Samples of the sandy silt till had water contents of between 8 and 11 per cent. Grain size distribution curves for samples of the sandy silt till are shown on Figure A-7.

Sand

Layers of sand were encountered beneath the clayey silt till in borehole 612, within the clayey silt till in borehole 601, interbed within the sandy silt till in boreholes 603 and 604, and below the sandy silt till in boreholes 602 and 604, between elevations 164.3 and 173.1 metres. Where fully penetrated, the sand layers were between 0.4 and 1.5 metres thick. Borehole 602 was terminated after penetrating a sand layer for 0.3 metres. The sand layers in boreholes 603 and 612 were noted to be silty. Measured N values in the sand ranged from 10 to 92 blows per 0.3 metres and samples of the sand had water contents of between 10 and 12 per cent. Grain size distribution curves for samples of the sand are shown on Figure A-8.

Sand and Gravel

Sand and gravel layers were encountered within the sandy silt till in borehole 603, beneath the lower sand layer in borehole 604, and beneath the clayey silt till in borehole 614, between elevations 163.3 and 172.5 metres. The sand and gravel layer in borehole 603 was 0.9 metres thick. Boreholes 604 and 614 were terminated in the sand and gravel after penetrating the layer for 2.6 and 1.7 metres, respectively. The compact sand and gravel in boreholes 603 and 614 had measured N values of 23 and 26 blows per 0.3 metres, respectively and the very dense sand and gravel at depth in borehole 604 had measured N values of 100 and greater than 100 blows per 0.3 metres. Samples of the sand and gravel had measured water contents ranging from 7 to 11 per cent. Grain size distribution curves for samples of the sand and gravel are shown on Figure A-9.

Silt

Very dense silt layers were encountered beneath the sandy silt till in boreholes 601 and 603 at elevations 167.3 and 163.8 metres, respectively. The silt layers were explored for 1.1 and 0.1 metres prior to terminating boreholes 601 and 603, respectively. Measured N values in the silt layers were 84 and 101 blows per 0.3 metres. A select sample of the silt had a water content of 22 per cent. A grain size distribution curve for a sample of the silt is shown on Figure A-10.

4.1.3 Southeast Quadrant – S-E Ramp

Boreholes 624 and 625 were advanced in the area of the S-E Ramp. The interpreted stratigraphic profile is shown on Drawing 4.

Topsoil

Boreholes 624 and 625 were advanced in the area of the S-E Ramp high fill and encountered 240 and 210 millimetres, respectively, of topsoil at the ground surface.

Silty Clay Glacial Till

A 2.8 metre thick layer of firm to very stiff silty clay glacial till was encountered beneath the topsoil in borehole 625 at elevation 182.3 metres. The silty clay till layer had measured N values ranging from 7 to 18 blows per 0.3 metres. A select sample of the silty clay till had a water content of 17 per cent, liquid and plastic limits of 36 and 19 per cent, respectively, and a plasticity index of 18 per cent, indicating intermediate plasticity. The results of the Atterberg limits determination are provided on Figure A-13. A grain size distribution curve for a sample of the silty clay till is shown on Figure A-4. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the silty clay till.

Clayey Silt Glacial Till

Beneath the topsoil in borehole 624 and the silty clay till in borehole 625 stiff to very stiff clayey silt glacial till was encountered at elevations 182.8 and 179.5 metres. The boreholes penetrated the clayey silt till for 2.0 and 4.8 metres prior to termination of the boreholes. The clayey silt till had measured N values ranging from 11 to 22 blows per 0.3 metres. Samples of the clayey silt till had water contents ranging from 16 to 19 per cent. Three Atterberg limits determinations carried out on samples of the clayey silt till, the results of which are provided on Figure A-13, yielded liquid limits ranging from 27 to 34 per cent, plastic limits ranging from 17 to 19 per cent, and plasticity indices of 15 to 20 per cent, indicating low plasticity. Grain size distribution curves for samples of the clayey silt till are shown on Figure A-6. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the clayey silt till.

Silty Clay to Clayey Silt

Cohesive soils consisting of silty clay to clayey silt were encountered underlying the clayey silt till below approximately elevation 167 metres in boreholes 202, 204 and 205 located at the Highway 40/Lucas Drain structure. Although this deposit was not encountered in boreholes 624 and 625 drilled for the high fill ramp these boreholes did not penetrate below elevation 177.5 metres. Where present, the silty clay to clayey silt deposit was found to be structurally different than the overlying deposit of cohesive glacial till. Standard penetration test results, Atterberg limits determinations, undrained shear strength measurements and grain size distributions also indicated that the material below elevation 167 metres is a geologically different deposit than the overlying glacial till. Borehole and laboratory data indicate that the consistency of this silty clay to clayey silt deposit becomes stiffer from south to north in the vicinity of Lucas Drain. At the location of the south abutment of the Highway 401/Highway 40 underpass structure, this silty clay to clayey silt deposit was not encountered in any of the deep boreholes. The transition from this deep cohesive deposit to the compact to very dense granular soils encountered at similar elevations near and north of Highway 401 is unknown and the transition zone may underlie the S-E Ramp area below elevation 167 meters.

4.1.4 Southwest Quadrant – W-N/S and N-E Ramps

Boreholes 615 to 623 were advanced in the southwest quadrant. The interpreted stratigraphic profiles along the W-N/S and N-E Ramps are shown on Drawing 3.

Fill

Boreholes 615 to 623 were advanced in the southwest quadrant of the site where the current land use is agricultural and the majority of these boreholes were advanced in an area of farm field which contained remnants of the previous year's corn crop at the ground surface. Borehole 615, advanced in the ditch adjacent to the Highway 401/Highway 40 underpass approach embankment, encountered 300 millimetres of surficial vegetation underlain by about 2.4 metres of firm to stiff silty clay fill material. The remainder of the boreholes advanced in the southwest quadrant encountered between 0.2 and 2.1 metres of fill materials from the ground surface. The fill materials were encountered to between elevations 179.5 and 182.8 metres, or 0.2 to 2.7 metres below the existing ground surface. The fill materials varied from silt and sand to silty clay and clayey silt and may be partly associated with the agricultural plowing activities on the site.

Measured N values in the fill materials ranged from 1 to 12 blows per 0.3 metres. Samples of the fill materials had water contents of between 20 and 31 per cent. Atterberg limits determinations were carried out on four samples of cohesive fill materials, the results of which are provided on Figure A-11. Three samples of silty clay fill had liquid and plastic limits ranging from 41 to 49 per cent and 19 to 24 per cent, respectively, and plasticity indices of 22 to 24 per cent, indicating intermediate plasticity. A sample of the clayey silt fill had liquid and plastic limits of 31 and 18 per cent, respectively, and a plasticity index of 13 per cent, indicating low plasticity. Grain size distribution curves for samples of the fill materials are shown on Figure A-1.

Silty Clay

Layers of stiff silty clay were encountered beneath the fill materials in boreholes 620 and 623 at elevations 181.9 and 182.2 metres, respectively, and were 1.6 and 0.5 metres thick. The silty clay layers had measured N values of 10 and 14 blows per 0.3 metres. A select sample of the silty clay from borehole 620 had a water content of 26 per cent, liquid and plastic limits of 51 and 24 per cent, respectively, and a plasticity index of 26 per cent, indicating high plasticity. The results of the Atterberg limits determination are provided on Figure A-13. A grain size distribution curve for a sample of the silty clay is shown on Figure A-3.

Clayey Silt

A 0.8 metre thick layer of stiff clayey silt was encountered beneath the silty clay in borehole 623 at elevation 181.4 metres. A single N value in the clayey silt was 14 blows per 0.3 metres. The corresponding sample had a water content of 20 per cent, liquid and plastic limits of 33 and 19 per cent, respectively, and plasticity index of 13 per cent, indicating low plasticity. The results of the Atterberg limits determination are provided on Figure A-13. A grain size distribution curve for a sample of the clayey silty is shown on Figure A-2.

Silty Clay Glacial Till

Stiff to very stiff silty clay glacial till was encountered beneath the fill materials in boreholes 615 and 622 at elevations 179.5 and 181.7 metres, respectively. The silty clay till in borehole 615 was 1.2 metres thick. The silty clay till in borehole 622 was penetrated for 5.2 metres prior to termination of the borehole. The silty clay till had measured N values ranging from 10 to 18 blows per 0.3 metres. In-situ vane shear strength testing carried out in borehole 622 indicated undrained shear strengths of greater than 144 kilopascals. Samples of the silty clay till had water contents of 18 per cent, liquid limits of 38 and 39 per cent, plastic limits of 19 and 20 per cent, and plasticity indices of 18 and 19 per cent, indicating intermediate plasticity. The results of the Atterberg limits determinations are provided on Figures A-12 and A-13. Grain size distribution curves for samples of the silty clay till are shown on Figure A-4. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the silty clay till.

Clayey Silt Glacial Till

A deposit of stiff to very stiff clayey silt glacial till was encountered beneath the fill materials in boreholes 616 to 619 and 621, beneath the silty clay and clayey silt in boreholes 620 and 623, respectively, and beneath the silty clay till in borehole 615, between elevations 178.3 and 182.8 metres. Each of these boreholes was terminated in the clayey silt till after exploring the layer for between 2.9 and 10.9 metres.

The clayey silt till had measured N values ranging from 10 to 27 blows per 0.3 metres. In-situ vane shear strength testing carried out in the clayey silt till indicated undrained shear strengths of greater than 144 kilopascals. Samples of the clayey silt till had water contents ranging from 14 to 22 per cent. The results of eighteen Atterberg limits determinations carried out on samples of the clayey silt till are provided on Figures A-12 and A-13. The testing yielded liquid limits ranging from 23 to 34 per cent, plastic limits ranging from 14 to 19 per cent, and plasticity indices of 9 to 16 per cent, indicating low plasticity. Grain size distribution curves for samples of the clayey silt till are shown on Figures A-5 and A-6. Cobbles and boulders should be anticipated within the glacial till.

Consolidation testing carried out on a sample of the clayey silt till indicated the following properties. Results of the consolidation testing are provided in Appendix B.

Borehole	**Sample**	**Depth (m)**	**Effective Overburden Pressure (kPa)**	**Initial Void Ratio**	**Recompression Index, C_R**	**Compression Index, C_C**	**Preconsolidation Pressure (kPa)**
616	6	4.9	75	0.47	0.012	0.123	212

Silty Clay to Clayey Silt

As described above for the southeast quadrant (S-E Ramp), cohesive soils consisting of silty clay to clayey silt were encountered underlying the clayey silt till below approximately elevation 167 metres in boreholes 202, 204 and 205 located at the Highway 40/Lucas Drain structure. Although this deposit was not encountered in boreholes 615 through 623 drilled for the high fill ramp, these boreholes did not penetrate below elevation

171.4 metres. Where present, the silty clay to clayey silt deposit was found to be different than the overlying deposit of cohesive glacial till. Borehole and laboratory data indicate that the consistency of this silty clay to clayey silt deposit becomes stiffer from south to north in the vicinity of Lucas Drain and this deposit was not encountered in any of the deep boreholes near the Highway 401/Highway 40 underpass structure. The transition from this deep cohesive deposit to the compact to very dense granular soils encountered at similar elevations near and north of Highway 401 is unknown and the transition zone may underlie the N-E and W-N/S Ramp areas below elevation 167 metres.

4.2 Methane Gas

Pockets of suspected methane gas were encountered at elevations 173.9 and 164.1 metres during drilling at borehole 603 as evidenced by an initial high pressure return of drilling fluids and bubbling of the drilling fluids at the ground surface. Methane gas has been reported in exploratory borings or within or near the Kettle Point Formation which underlies the overburden at this site. [5]

4.3 Piezo-Cone Penetration Testing

During the CPTs carried out adjacent to boreholes 603 and 617, the cone was pushed from the ground surface through existing fill materials and native cohesive soils, and achieved refusal at depths of 8.1 and 5.7 metres, respectively, or elevations 171.9 and 177.2 metres. The tip resistance, local side friction and pore water pressures were measured over the depths penetrated. This data was correlated to the preconsolidation pressure and undrained shear strength, water content and oedometer data from the borehole samples as shown in Figures 3 and 4 and discussed in further report sections.

4.4 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling and, in some cases, the water level in open boreholes was measured immediately following drilling. Piezometers were installed in boreholes 602, 604, 606, 609, 616, 619, 621 and 624 as shown on the Record of Borehole sheets. Encountered and measured groundwater levels are summarized in the following tables.

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[5] Dusseault, M.B. and Loftsson M., 1985: The Mechanical Properties of the Kettle Point Oil Shale, Ontario Geological Survey Open File Report 5560, 93p. 36 figures, 8 tables.

Quadrant	Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level		Measured Groundwater Level Immediately Following Drilling	
			Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
Northwest	607	183.5	*	*	-	-
	608	183.3	*	*	-	-
	609	183.5	*	*	-	-
	610	183.5	*	*	-	-
Northeast	601	180.4	2.1	178.3	-	-
	602	180.4	2.1	178.3	-	-
	603	180.6	5.8	174.8	-	-
	604	180.7	*	*	-	-
	605	181.5	*	*	4.0	177.5
	606	180.5	*	*	-	-
	611	180.3	*	*	0.4	179.9
	612	180.6	3.0	177.6	1.8	178.8
	613	180.7	*	*	0.5	180.2
	614	180.4	*	*	0.6	179.8
Southwest	615	182.0	*	*	3.2	178.8
	616	182.9	*	*	-	-
	617	182.9	*	*	-	-
	618	183.3	*	*	-	-
	619	182.3	*	*	-	-
	620	182.5	*	*	-	-
	621	183.1	*	*	-	-
	622	183.1	*	*	-	-
	623	182.7	*	*	-	-
Southeast	624	183.1	*	*	-	-
	625	182.5	*	*	-	-

* Groundwater level not established during drilling.

Quadrant	Borehole	Ground Surface Elevation (m)	Date and Measured Groundwater Elevation (m)				
Northwest	609	183.54	June 27/14 dry to 177.75	July 9/14 dry to 177.75			
Northeast	602	180.40	June 9/14 178.27	June 11/14 179.70	June 16/14 179.58	July 9/14 179.76	
	604	180.71	June 16/14 175.44	June 20/14 173.64	July 9/14 175.53		
	606	180.49	June 6/14 172.96	June 9/14 173.02	June 16/14 173.30	July 9/14 173.91	
Southwest	616	182.85	May 16/14 172.66	May 21/14 174.99	June 4/14 177.21	July 9/14 178.58	
	619	182.25	May 22/14 dry to 173.08	June 4/14 dry to 173.08	July 9/14 173.69		
	621	183.08	May 13/14 dry to 176.98	May 15/14 dry to 176.98	May 21/14 178.66	June 4/14 179.06	July 9/14 179.21
Southeast	624	183.08	May 6/14 dry to 179.12	June 4/14 182.65	June 12/14 182.44	July 9/14 182.71	

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions.

Each of the boreholes advanced in the northwest quadrant of the interchange remained dry during and following drilling and the piezometer installed in borehole 609 was found to be dry immediately following installation on June 27, 2014 and twelve days later on July 9, 2014. Based on the soil colour change from brown to grey, the inferred groundwater level is at about elevation 181 metres for design purposes.

In the northeast quadrant, groundwater seepage was observed during drilling of each borehole from the fill materials between 1.4 and 4.4 metres below the ground surface, or between elevations 176.2 and 179.3 metres, and from the deeper granular layers interbedded within the tills. Groundwater levels encountered during and immediately following drilling ranged from 0.4 to 5.8 metres below the ground surface, or elevation 174.8 to 180.3 metres. The groundwater levels in the piezometers were measured at between elevations 173.0 and 179.8 metres. Based on the soil colour change from brown to grey and the measured groundwater elevations, the inferred groundwater level in this area is at about elevation 180 metres for design purposes.

Each of the boreholes advanced in the southeast quadrant remained dry during and immediately following drilling. The groundwater level in the piezometer installed in borehole 624 was measured at between elevations 182.4 and 182.7 metres. Based on the measured groundwater elevations, the inferred groundwater level in this area is at about elevation 182.5 metres for design purposes.

Eight of the nine boreholes advanced in the southwest quadrant remained dry during and immediately following drilling. Groundwater seepage was observed below elevation 178.3 metres in borehole 615, advanced adjacent to the existing Highway 40 embankment. The groundwater levels measured in the piezometers installed in boreholes 619 and 620 were between elevations 173.7 and 178.9 metres. Based on the soil colour change from brown to grey and the measured groundwater elevations, the inferred groundwater level in this area is at about elevation 181 metres for design purposes.

There appears to be a confined sand and gravel aquifer at depth which was encountered at various locations within the Highway 401/Highway 40 interchange, such as borehole 604, boreholes 101 and 104 to 107 advanced for the Highway 401/40 Underpass (Geocres Report No. 40J8-59) and boreholes 303 and 304 advanced for the McGregor Creek Bridge (Geocres Report No. 40J8-63). The inferred groundwater level of this aquifer is elevation 175.5 metres.

The inferred groundwater levels in the interchange quadrants are summarized in the following table.

Quadrant	Ramp(s)	Inferred Groundwater Elevation (m)
Northwest	N-W	181.0
Northeast	E-N/S & S-W	180.0
Southeast	S-E	182.5
Southwest	W-N/S & N-E	181.0

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 Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Simon Lutz, Mr. Michael Arthur and Mr. Brett Thorner, E.I.T. under the direction of the Field Investigation Manger, Mr. David J. Mitchell. The CPTs were supervised by Mr. Mrinmoy Kanungo, P.Eng.

Routine laboratory tests were carried out at Golder's 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. The consolidation testing was conducted in Golder's Mississauga laboratory under the supervision of Dr. J. Paul Dittrich, P.Eng. The Mississauga laboratory is a MTO registered laboratory in the specialty of soil and rock including testing for Foundation Engineering Low and High complexity.

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., a senior geotechnical engineer and Principal 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.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
D. U. PROUT
Jul 29/15
PROVINCE OF ONTARIO

Dirka U. Prout, P.Eng.
Geotechnical Engineer

Storer J. Boone, Ph.D., P.Eng.
Principal

REGISTERED PROFESSIONAL ENGINEER
F.J. HEFFERNAN
Jul 29/15
PROVINCE OF ONTARIO

Fintan J. Heffernan, P.Eng.
MTO Designated Contact

NG/SJB/FJH/cr

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PART B

FOUNDATION DESIGN REPORT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
CHATHAM-KENT, ONTARIO
GWP 3093-09-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION

6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design of the high fill ramp embankments associated with the interchange reconfiguration. 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.

This report addresses the design of high fill ramp embankments associated with the Highway 401/ Highway 40 interchange reconfiguration. For the purposes of this report, the project site is presumed to be divided into the four quadrants formed by the two highways. The following table summarizes the locations and heights of the proposed high fill embankments.

Quadrant	Ramp	Location (Stations)	Height (m)
Northwest	N-W	10+000 to 10+140	4.5 to 6.0
Northeast	S-W	10+075 to 10+210	4.5 to 8.4
	E-N/S	10+230 to 10+430	4.5 to 9.1
Southwest	W-N/S	10+280 to 10+500	4.5 to 9.5
	N-E	10+000 to 10+160	4.5 to 9.0
Southeast	S-E	10+070 to 10+140	4.5 to 5.0

The drawings provided by Dillon indicate that the ramp embankment side slopes will generally have inclinations no steeper than two horizontal to one vertical (2H:1V). Where two ramps are adjacent, such as in the northeast and southwest quadrants, they will form a single composite embankment. In the northeast quadrant, a retaining wall is to be constructed on the northern side of the E-N/S Ramp in order to reduce the construction footprint on the McGregor Creek floodplain. Design of the retaining wall has been addressed under separate cover (Geocres Report No. 40J8-62).

Embankment settlement considerations near the Highway 40/Lucas Drain overpass site are somewhat different than at other structure and embankment locations on this project site, as reported in the Foundation Investigation and Design Report for that structure (Geocres Report No. 40J8-60). Most of the settlement issues are, like at the other sites, controlled largely by the new loads imposed by changing the embankment widths and elevations. The Highway 40/Lucas Drain site, however, is different in that a softer layer of clayey silt to silty clay was found underlying the site below approximately elevation 167 metres, whereas at the other locations granular deposits dominated the stratigraphy below this elevation. The transitions between the conditions at the Highway 40/Lucas Drain overpass site and those at the Highway 401/Highway 40 Underpass and ramps south of Highway 401 sites is unknown. Recommendations and discussions related to these issues are provided in subsequent sections of this report.

Uncontrolled fill materials were encountered in the boreholes advanced in the northeast quadrant of the site. At the borehole locations, fill materials were between 2.4 and 4.0 metres thick and extended to elevations between 176.2 and 177.7 metres. Based on a review of the site conditions pre-dating the existing interchange and contouring of the fill and native soil interface, fill may extend to depths of about 4.5 metres in some local areas. The fill composition included sand and gravel, silty clay, clayey silt and silt along with pockets clayey silt and varying amounts of wood, organic materials and other debris. These materials could lead to adverse settlements and, therefore, recommendations are provided in this report to address settlement mitigation measures.

6.1 Embankment Design

The existing fill materials encountered at the site, particularly in the northeast quadrant, are not considered suitable for construction of the proposed embankment without incurring the risk of unpredictable settlement (magnitude and location). Further, any organic, soft or loose materials in the areas of new embankments that were associated with the former McGregor Creek or Lucas Drain channels are unsuitable for support of the new embankments without incurring adverse settlements.

In general, it is recommended that surficial topsoil, uncontrolled fill materials and former channel fill materials be sub-excavated and the embankment founded on the firm to very stiff clayey silt till. Excavations for fill removal and subgrade preparation will extend below the ground level outside of the existing embankments by between 2 and about 4 metres. Preliminary estimates indicate that approximately 3,000 to 4,000 cubic metres (m^3) of fill removal, with an average cut depth of 3 metres, may be required in the area of the planned retaining wall in the northeast quadrant (addressed in a separate report). In the area of the northeast quadrant and south of the proposed retaining wall beneath the E-N/S and S-W Ramps, approximately 16,000 to 18,000 m^3 of fill removal with an average cut depth of about 2 metres, may be required, based on plans available during preparation of this report.

Embankment fill alternatives include rock fill, granular fill and lightweight fills; however, based on the existing embankment fills being granular, local availability of granular fill materials, and the relatively shallow depth to competent founding soils, it is anticipated that the new embankments will generally be constructed using imported fill meeting the relevant Ontario Provincial Standard Specifications (OPSS) provincial requirements.

Typically, highway embankments in the area are planned with side slopes of about 2H:1V. If the existing fill materials are fully removed and the entire volume of the new embankments are constructed using new controlled fill materials, there will be an excess volume of the existing fill materials that must be managed by either transportation to an off-site management area or re-use on-site. To optimize on site reuse of the existing fill the new embankments could be constructed with a core and shell approach. In this case, the core of the embankments could be constructed with new engineered fill fully beneath the roadway to the edge of rounding and with side slopes of about 1.5H:1V. The remaining volume of the embankment (the “shell”) could be constructed to final side slopes of 2H:1V or flatter using some portion of the existing fill. In this case, the reused fill materials may not necessarily meet the material composition/gradation criteria for engineered fill; however, provided that these fill materials are compacted and benched into the core, they should be suitable for the slope flattening. This core and shell approach is commonly used in MTO’s northeast and northwest regions when the embankments are commonly constructed of engineered fill (granular or rock fill) and slope flattening is accomplished using excavated and excess swamp materials. Slope flattening should be carried out in accordance with Ontario Provincial Standard Drawing (OPSD) 202.010.

If the existing uncontrolled fill materials might be reused, it will be essential to monitor the composition of these materials during excavation. Some of the existing fill may include significant quantities of fully or partially decomposed organic matter and should be stockpiled separately for use in landscaped areas. If, or where debris is encountered within the fill, it should be removed from the site to an appropriate location for further management. At the time this report was prepared the relative volumes of materials suitable for reuse in slope flattening, landscaping or removal from the site could not be accurately determined from the available information.

If sufficient volume is not available within the MTO properties available for this project for management of excess materials, rammed aggregate piers could be used to assist with limiting settlement where the new embankments are constructed in areas presently occupied by the uncontrolled fill. Rammed aggregate piers are constructed by pre-boring a hole on the order of 1 metre diameter to a pre-selected depth and placing and compacting crushed rock aggregates in thin lifts within the hole. Compaction is carried out by using high-energy ram systems. During compaction, the aggregates are also typically displaced outward which assists in achieving a larger diameter aggregate pier and densifying the surrounding soils. Some systems can construct these piers without pre-boring a hole and rely on displacement of the surrounding ground with driven mandrills. Typically, rammed aggregate piers are used where the desired depth of ground improvement ranges between about 2 and 8 metres and are spaced approximately 1.5 to 2.5 metres centre-to-centre, depending on the degree of ground improvement necessary. Settlement estimates are commonly determined by the specialist subcontractor based on:

- weighted-average soil properties (existing ground and pier materials); and
- empirical relationships that compare the proprietary equipment and methods with spacing, depth, surrounding soil conditions and final performance.

In this case, the depth of existing fill to be improved is near the lower threshold of depth where use of rammed aggregate piers becomes a suitable alternative. The practicality and cost-effectiveness of using rammed aggregate piers will depend on the available space for management of the excess soils and relative costs and schedule implications for excavation and replacement versus rammed aggregate piers.

6.1.1 Stability

Due to the generally stiff to hard consistency of the near surface native cohesive soils, adverse stability performance issues are not anticipated provided any organics, uncontrolled fill materials, soft soils, and otherwise deleterious materials are removed from the embankment subgrades as recommended in this report. Critical sections from each high fill area were selected for slope stability analyses using SLOPE/W Version 7.19, a commercially available software package by Geo-Slope International for limit equilibrium stability analyses. In the following report sections, critical cross sections selected for slope stability and settlement analyses correspond to the greatest new embankment heights. The cross-section geometry was interpolated from the provided drawings and, where necessary, assumed side slope inclinations of 2H:1V. A factor of safety against instability of at least 1.3 was found to be available for the slopes as presented in the cross sections on

Figures C-1 to C-3 in Appendix C. For slope stability analyses it has been assumed that removal of organic materials and deleterious fills and subgrade preparation are carried out as described in this report.

6.1.2 Settlement

Data Interpretation

Estimates of engineering parameters used in the settlement analyses were based on SPT N values obtained in the boreholes advanced in each quadrant for the high fill embankments and oedometer tests carried out on samples from boreholes 604 and 616. In addition to direct estimation from the oedometer tests, the compression index, C_c, undrained shear strength, s_u, and preconsolidation pressure, σ'_p, were interpreted using correlations developed for Geocres Report No. 40J6-28 (Subsurface Conditions Interpretation Report for the Windsor Essex Parkway) since the soils at the Highway 401/Highway 40 interchange site are of similar geologic origin and composition.

Undrained shear strengths were also correlated to the SPT N field values using the following relationship:

$$s_{u(SPT)} = 9.4\, N_{field}$$

where: $s_{u(SPT)}$ = undrained shear strength as derived from the SPT (kPa)
N_{field} = field SPT N value using automatic hammer

This correlation between the SPT values is an approximation due to the inherent variability of the energy delivered during the SPT procedure; however, the correlation was based on comparisons of data from multiple boreholes and high-quality, strain-controlled field vane shear testing carried out for preparation of Geocres No. 40J6-28.

Stress-strain properties were estimated using a correlation as follows:

$$C_c = 0.0086\, w_n - 0.0086$$

$$C_r = 0.11\, C_c$$

w_n = natural water content expressed as a per cent

The recompression index, C_r, was calculated using the correlation relating C_r to C_c from Geocres No. 40J6-28 where C_r is approximately 11 per cent of C_c. The C_c values obtained from the oedometer test completed at this site were consistent with the correlation between water content and C_c identified in Geocres No. 40J6-28. Also, the preconsolidation pressure was established using the undrained shear strength values based on CPT data. The following well-known relationship was used to estimate the preconsolidation pressure based on undrained shear strength measurements.

$$\sigma'_p = s_u / 0.22 \quad \text{(after Mesri 1975 [6])}$$

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[6] Mesri, G. (1975). New Design Procedure for Stability of Soft Clays: Discussion. Journal of the Geotechnical Engineering Division, ASCE 101(4), 409 – 411.

Based on the reasonably consistent interpretations of undrained shear strength obtained by independent correlations with SPT, CPT and field vane shear data and low initial void ratios (based on water content data) it was considered that, for the purposes of applying consolidation theory to settlement estimates, all of the soils at the site would exhibit recompression behaviour and that the preconsolidation pressure would have little if any effect on the magnitude of settlement.

Settlement Analyses

Settlement of the founding native soils due to the proposed embankment loading was analysed using Settle^{3D} Version 2.0, a 3-dimensional program by Rocscience for the analysis of vertical consolidation and settlement under foundations, embankments and surface loads, the results of which are discussed below. Additional settlement of the embankment fill materials should be anticipated due to internal compression of the new engineered fill. The settlement of the embankment fill materials is expected to be less than 1 per cent of the embankment height and, assuming granular fill materials are used, is expected to occur during construction. Further, the settlement estimates described in this report are associated with settlement that occurs due to compression of the native soils, regardless of whether rammed aggregate piers are used or the existing uncontrolled fill materials are removed and replaced with new engineered fill.

In general, it was found that the total settlement of the founding soils would be less than the MTO’s allowable post-construction settlements for new embankments as describe below; however, where the N-E and N-W Ramp embankments form parts of bridge approaches, total settlements could exceed the allowable post-construction settlement within the bridge approach transition zone depending on construction staging. The majority of the settlement is expected to occur during construction in proportion to the rate and magnitude of applied loads. Estimated settlements for the proposed high fill embankments are summarized below specific to each of the high fill embankments.

It should be noted that the selected preconsolidation pressures used in the analyses are based on the correlations between the SPT, CPT, field shear vane data and oedometer testing as described above. This methodology for deriving the preconsolidation pressure was considered to be more representative than using the oedometer data only.

Estimated Settlement, Northwest Quadrant – N-W Ramp

Between Stations 10+000 and 10+140, the N-W Ramp will be between 4.5 and 6.0 metres in height and will form part of the McGregor Creek bridge approach embankment. The existing ground surface elevation in the area of the N-W Ramp high fill varies from about 179.0 metres near Station 10+000 to about 183.2 metres near Station 10+140.

The engineering parameters and simplified stratigraphy utilized in the settlement analysis are provided in the table below.

Material and Elevation (m)	γ (kN/m^3)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Clayey Silt Till 179.6 to 182.0	22	-	-	0.3	0.128	0.0140	818	0.51
Clayey Silt Till 174.0 to 179.6	22	-	-	0.3	0.146	0.0161	682	0.51
Clayey Silt Till 170.0 to 174.0	22	-	-	0.3	0.146	0.0191	455	0.51
Sandy Silt Till 164.0 to 170.0	22	38,000	38,000	0.3	-	-	-	-

It has been indicated that the N-W Ramp is to be constructed prior to the construction of the new ramp bridge over McGregor Creek in order to provide access for the bridge construction. The maximum total settlement due to the embankment construction is estimated to be 74 millimetres which will occur at about Station 10+100. This is within the MTO post-construction settlement criteria for new embankments of maximum 200 millimetres within 20 years.

The total settlement within 75 metres of the bridge abutment is expected to vary from 95 millimetres at the abutment to 50 millimetres. This exceeds the MTO's post-construction settlement criteria for longitudinal transition zones. In order to reduce the post-construction settlement, consideration may be given to using lightweight embankment fill within the transition zone.

Estimated Settlement, Northeast Quadrant – E-N/S and S-W Ramps

Approximately 200 metres of the E-N/S Ramp, from Station 10+230 to 10+430, and 135 metres of the S-W Ramp, from Station 10+075 to 10+210, will require high fill embankments with proposed heights between 4.5 and 9.1 metres. Where the new alignment coincides with the existing alignment, the S-W Ramp embankment will incorporate the existing ramp embankments at about Stations 10+090 and 10+215. The existing ground surface elevation along the new S-W Ramp alignment between Stations 10+085 and 10+205 varies from 180.5 to 182.0 metres.

The E-N/S Ramp will coincide with the existing Highway 40 embankment at Highway 40 Station 10+130. The existing E-N/S Ramp ground surface is between about elevations 180.5 and 181.0 metres in the proposed high fill area. In order to restrict the embankment width and prevent the embankment footprint from encroaching on McGregor Creek, a retaining wall is to be constructed along the north side of the E-N/S Ramp to retain the embankment fills, with the proposed top of wall elevation ranging from about 185.6 to 190.2 metres. Recommendations for design of the retaining wall have been reported under separate cover for this assignment (Geocres Report No.40J8-62).

The engineering parameters and simplified stratigraphy utilized in the settlement analysis are provided in the table below.

Material and Elevation (m)	γ (kN/m^3)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Clayey Silt Till 175.5 to 177.0	22	-	-	0.3	0.141	0.0155	900	0.51
Clayey Silt to Silty Clay Till 174.0 to 175.5	22	-	-	0.3	0.147	0.0162	680	0.51
Clayey Silt Till 172.0 to 174.0	22	-	-	0.3	0.138	0.0152	455	0.51
Sandy Silt Till 164.0 to 172.0	22	110,000	110,000	0.3	-	-	-	-

The anticipated maximum magnitude of settlement in the northeast quadrant due to new embankment construction is 75 millimetres which is expected to occur at the critical section described above. Approximately 20 millimetres of the total settlement may be attributed to primary settlement of the granular deposits which is expected to occur relatively quickly during or immediately following construction. Ninety per cent of the long-term settlement is expected to occur within about 2 years following embankment construction with 50 per cent settlement within the first year. This is within the MTO's post-construction settlement criteria for new embankments of maximum 200 millimetres within 20 years. If the use of lightweight fill is considered for construction of the proposed retaining wall, reduced settlement may be realized.

Estimated Settlement, Southeast Quadrant – S-E Ramp

The proposed high fill embankment associated with the S-E Ramp extends from Station 10+070 to 10+140 and will be 4.5 to 5.0 metres in height. The existing W-N/S and N/S-E Ramp embankments will be incorporated into the new S-E Ramp where the alignments coincide at about Stations 10+050, 10+150 and 10+215.

The engineering parameters and simplified stratigraphy utilized in the settlement analysis are summarized in the table below.

Material and Elevation (m)	γ (kN/m^3)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Clayey Silt to Silty Clay Till 177.5 to 182.6	21	-	-	0.3	0.139	0.0153	455	0.47
Clayey Silt Till 172.5 to 177.5	21	-	-	0.3	0.120	0.0132	682	0.47
Clayey Silt Till 167.0 to 172.5	21	-	-	0.3	0.155	0.0171	909	0.47
Sandy Silt Till 160.0 to 167.0	22	51,000	51,000	0.3	-	-	-	-

Assuming that the silty clay to clayey silt deposit is not present beneath the majority of this ramp, the anticipated maximum magnitude of settlement in the southeast quadrant due to new embankment construction is about 80 millimetres which is expected to occur at about Station 10+085. If the compressible layer of silty clay to clayey silt deposit extends sufficiently north and east to the area of the maximum S-E Ramp embankment height, the total settlement could be as much as 100 millimetres and develop over a period of as much as 10 years. Although these settlement magnitudes and rates would not be acceptable near structures, both values are within the MTO's post-construction settlement criteria for new embankments of maximum 200 millimetres.

Estimated Settlement, Southwest Quadrant – W-N/S and N-E Ramps

High fills will comprise the W-N/S and N-E Ramp embankments. Between Stations 10+280 and 10+500, the W-N/S Ramp will have heights of between 4.5 and 9.5 metres. Between Stations 10+000 and 10+160, the N-E Ramp will have heights of between 4.5 and 9.0 metres. The existing ground surface in the area of the proposed embankment alignments varies from elevation 182.2 to 183.5 metres. The N-E Ramp embankment will form part of the Highway 401/Highway 40 underpass approach embankment at Highway 40 Station 9+944. The engineering parameters and simplified stratigraphy utilized in the settlement analysis are summarized in the table below.

Material and Elevation (m)	γ (kN/m^3)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Clayey Silt to Silty Clay Till 176.0 to 181.0	21	-	-	0.3	0.134	0.0147	909	0.47
Clayey Silt to Silty Clay Till 172.6 to 176.0	21	-	-	0.3	0.124	0.0136	636	0.47
Sandy Silt Till 169.0 to 172.6	22	23,000	23,000	0.3	-	-	-	-

Assuming that the silty clay to clayey silt deposit is not present beneath the majority of this ramp, the maximum settlement of the founding soils due to the construction of the combined W-N/S and N-E Ramp embankments is expected to be about 90 millimetres. This settlement is expected to occur on the N-E Ramp in the area of the Highway 401/Highway 40 underpass and on the W-N/S Ramp near Highway 40. If the compressible layer of silty clay to clayey silt deposit extends sufficiently north and west to the area of the maximum ramp embankment height in the southwest quadrant, the total settlement could be as much as 100 millimetres and develop over a period of as much as 10 years. Although these settlement magnitudes and rates would not be acceptable near structures, both values are within the MTO's post-construction settlement criteria for new embankments of maximum 200 millimetres.

Settlement Performance Requirements

According to the MTO Embankment Settlement Criteria for Design (July 2010) the total post-construction settlement of the paved portion of non-freeways for new embankments on compressible soils shall not exceed 200 millimetres over a 20 year period. The N-E Ramp will coincide with the Highway 401/Highway 40 underpass approach embankment and the N-W Ramp will form part of the McGregor Creek bridge approach embankment. Within a transition zone, defined as an area up to 75 metres from a transition point, or abutment, the maximum allowable post-construction embankment settlement varies from 20 to 100 millimetres over a 20 year period.

Settlement Management and Mitigation Methods

If settlement mitigation is considered necessary, based on the estimated settlement ranges summarized in this report, options for such mitigation are described below:

- **Early Embankment Construction/Preloading:** Construction of modifications to existing embankments and new embankments well in advance of structure construction will assist in minimizing the effects of settlement on the riding surface of pavements. By providing the opportunity for consolidation settlement of the native cohesive deposits to occur prior to further construction preloading will minimize the effect of settlement of the embankments relative to the proposed pavements and bridges.

- **Vertical Drains:** Installation of prefabricated vertical drains (wick drains) at the site was considered as a method to accelerate settlement of the clayey silt to silty clay layer found below elevation 167 metres in the immediate vicinity of the Highway 40/Lucas Drain overpass. Installing vertical drains through the overlying glacial till soils may be problematic and also impractical given the relatively small site. Sand drains (precursor to wick drains), constructed using conventional borehole drilling techniques and equipment, could be used in lieu of wick drains. While these are typically more costly on a per metre basis the overall cost can be less as compared to wick drains for small sites.

- **Lightweight Fill:** An alternative for reducing the magnitude of long-term settlement is to use lightweight fill for embankment construction. Typically, lightweight fill is not economically practical for general use and is most suited for areas underlain by deep compressible subsurface deposits where long-term post-construction creep settlements affect the performance of the highway and where there is no available time in the construction schedule for a sufficient preload or surcharge period. Three lightweight fill materials are available to achieve this purpose, listed in order of increasing unit weight:

  - Expanded Polystyrene (EPS): EPS is formed in blocks typically measuring about 1.2 by 0.6 by 0.2 metres ranging up to 2.0 by 0.75 by 0.75 metres with unit weights ranging from about 0.1 to 0.4 kilonewtons per cubic metre (kN/m^3), though EPS meeting the minimum compressive strength criteria for roadway applications is typically about 0.2 kN/m^3. Because the unit weight of EPS is less than that of water and due to the proximity of the retaining wall to the watercourse, the buoyancy of the EPS blocks must be considered during design. The effects of fluctuating groundwater and flood water levels due to the 100-year flood event, or other extreme event as recommended by a hydraulic or river engineer should be evaluated. These effects would include flotation, long-term water absorption and horizontal sliding due to unbalanced water levels. The EPS blocks must be protected against buoyancy effects both during and after construction;

- Tire Derived Aggregate (TDA): TDA consists of scrap tires shredded to strips 50 to 300 millimetres in length. This material has a unit weight in the range of 8 kN/m^3. The use of TDA may create the potential for leeching of environmentally adverse chemicals and, therefore, require permitting, monitoring and/or capping. Also, in order to prevent self-heating and potential combustion, the thickness of TDA used as embankment fill is restricted to no more than 3 metres. Regulatory approvals and related processes to control environmental concerns are not yet resolved in Ontario.
- Cellular Concrete: Cellular concrete is a product of cement, water, a foaming agent and air placed by injecting air and foaming agent into a cement-water slurry to produce a cured concrete-like material with unit weights typically on the order of 4 to 8 kN/m^3 and unconfined compressive strengths of 0.5 MPa or greater; and
- Blast Furnace Slag: Granular, water-cooled blast furnace slag can be used as a lightweight fill and, for MTO applications, typically exhibits unit weight values ranging from less than 12.5 kN/m^3 ("ultralightweight blast furnace slag") to about 14.5 kN/m^3 or less. Blast furnace slag is susceptible to crushing if over compacted. A non-standard special provision (NSSP) for lightweight fill material has been included in Appendix D which discusses construction methods and means of preventing overcrushing.

In order to limit differential settlement along the new and/or modified embankments, new embankment filling should be completed to as near the full final extent and as early as practical before paving and structure construction. Outside of the immediate vicinity of the Highway 40/Lucas Drain structure, the vertical and horizontal extent of the clayey silt to silty clay layer below elevation 167 metres is not known and, therefore, the need for and installation of vertical drains would be subject to significant uncertainty and are likely not warranted. Further, given the anticipated magnitude of total and differential settlements along the length of the proposed high fill embankments as compared to the MTO settlement criteria, use of lightweight fill materials should not be necessary in general for the high fill embankments.

Settlement Monitoring

A detailed settlement monitoring program should be carried out to assess the actual settlement performance of the new fill sections. It is recommended that the settlement monitoring program consist of conventional MTO settlement platforms installed on the properly prepared subgrade prior to placement of new embankment fill. These settlement platforms should be placed at approximately 50 metre intervals along the length of each new embankment or embankment widening. For new embankments, at least one row of settlement platforms should be installed as close to the embankment centreline as possible and additional platforms installed with the tops of the survey risers located near the crest of the embankments on alternating sides of the centreline settlement platforms. Each settlement monitoring point (platform and riser) should be surveyed using instruments, methods and personnel capable of achieving a repeatable accuracy of plus or minus 2 millimetres or less, and the survey referenced to two independent benchmarks. The settlement platforms should be paired with vibrating wire piezometers to monitor the pore water pressure. A NSSP should be included in the Contract Documents to indicate the need for settlement monitoring.

6.2 Excavations, Groundwater Control and Subgrade Preparation

The existing fill materials encountered at the site are not considered suitable for support of the proposed embankment. It is recommended that the surficial topsoil and fill materials be sub-excavated and the embankments be founded on the firm to very stiff clayey silt till. It is expected that excavations for fill removal and subgrade preparation will extend below the inferred groundwater level by between about 2 and 4 metres. In order to facilitate construction, the use of sumps and pumps may be required provided that the site is isolated from creek and drain flows.

Excavation operations for the proposed embankments should be conducted in accordance with OPSS 902. 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 uncontrolled fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey soils and properly dewatered granular materials may be classified as Type 2 soils. Temporary open cut slopes within the fill materials should be maintained no steeper than 1H:1V.

Excavations for removal of the uncontrolled fill will extend through the surficial topsoil and fill and terminate in the native firm to hard clayey silt till. In some areas, these excavations are expected to extend below the inferred groundwater level within the fill of 180 metres. Seepage should be expected from the saturated sand layers within the existing fill. Considering that the bulk of the native material expected within the excavation depths is cohesive in nature, it may be possible to control seepage by pumping from well filtered sumps. In order to limit dewatering requirements and improve material handling and trafficability in this area, it is preferred that work on the ramps and retaining wall is carried out during dry periods with low water levels within McGregor Creek. Surface water runoff should be directed away from the excavations at all times. The appropriate NSSP for the control of surface and groundwater flows should be included in the Contract Documents. Driven sheet piles used for excavation support during removal of the existing fill in the northeast quadrant, where the associated retaining wall will be constructed, will assist in limiting ground water inflow and its effects, particularly since the sheet piles will be driven into the underlying silty clay soils.

Based on the available information, it is expected that the interface elevation between the existing uncontrolled fill and native soils will be variable, particularly in the areas of the former McGregor Creek and Lucas Drain channels. Care should be taken during construction to minimize disturbance of the subgrade soils during excavation since the silty clay soils will be sensitive in the presence of moisture/water and construction traffic. Because of the variability of the uncontrolled fill and native soil interface elevation, and the sensitivity of the native soils to disturbance, all excavations and mass removal of existing fill should be carried out such that the final 0.5 metres of excavation is completed with qualified geotechnical personnel on site. If soft/loose, wet or other deleterious materials are found at the foundation level, these materials should be sub-excavated and replaced with engineered fill consisting of compacted OPSS Granular A or Granular B Type II materials. Where possible, it will be advantageous to build a working subgrade of at least 1 metre thick with compacted granular materials as soon as practicable following exposure of the native soils. This working subgrade should limit the influence of water on the native subgrade and could provide a means to control water that enters the excavations through use of properly filtered sumps and pumps.

6.3 Embankment Construction

Except for the top approximately 0.5 metres, where Granular A and B Type III material will be placed for the pavement structure, the embankment fill should consist of an approved granular borrow such as SSM, Granular B Type II or Type III. Granular embankment fill materials should be placed in maximum 300 millimetre thick loose lifts, properly benched into the existing embankments in accordance with OPSD 208.010, and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of 2H:1V or flatter. Embankments with total height greater than 8 metres should be constructed with minimum 2 metre wide mid-height benches. All grading and embankment construction should be conducted in accordance with OPSS 206 (November 2013) and MTO Special Provision 105S10 (amendment to OPSS 501).

6.4 Erosion Protection

Temporary erosion and sediment control measures should be implemented during construction in accordance with OPSS 805. If slopes are constructed with mid-height benches, they should be sloped away from the slope face and drain to a positive outlet. The completed slopes should be topsoiled and seeded or sodded, as applicable, immediately after construction. If permanent erosion protection measures will be delayed significantly after final grading, the surface should be roughened or provided with a temporary erosion protection blanket.

7.0 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., a senior geotechnical engineer and Principal 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.

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
D. U. PROUT
Jul 29/15
PROVINCE OF ONTARIO

DUProut

Dirka U. Prout, P.Eng.
Geotechnical Engineer

Storer J. Boone, Ph.D., P.Eng.
Principal

REGISTERED PROFESSIONAL ENGINEER
F.J. HEFFERNAN
Jul 29/15
PROVINCE OF ONTARIO

FJ Heffernan

Fintan J. Heffernan, P.Eng.
MTO Designated Contact

NG/SJB/FJH/cr

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TABLE I

COMPARISON OF SETTLEMENT MITIGATION ALTERNATIVES

High Fill Ramp Embankments
Highway 401 and Highway 40 Interchange Reconfiguration
GWP 3093-09-00

MITIGATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR[1]	RISKS/ CONSEQUENCES
Preload	• Best suited for infill areas where impacts on traffic and drainage are low.	• Relatively inexpensive. • Simple to implement. • Effective in reducing post-construction settlement.	• Time required to achieve desired results may be incompatible with project schedule. • Highly dependent on project staging.	• Low	• Disruption to construction schedule if rate of settlement is slower than predicted. • Post-construction settlement (creep) may still occur.
Vertical Drains	• Not feasible for this site.	• Significant reduction in time to achieve 100 per cent of primary consolidation.	• Expensive pre-drilling will be required through the heavily consolidated near-surface soils which are present at the site.	• High	• Obstructions likely present in till strata. • Reports of difficulty installing vertical drains in cohesive deposits with N values of 10 to 15 blows per 0.3 metres.

COMPARISON OF SETTLEMENT MITIGATION ALTERNATIVES

MITIGATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR[1]	RISKS/ CONSEQUENCES
Lightweight Fills	• Suitable for all high fill locations.	• Reduction in total settlement by 25 to 30 per cent or more. • Can be used to reduce differential settlement near bridge abutments and other structures. • Offers reduced lateral loads on retaining/bridge structures.	• More expensive than conventional earth fills. • Requires erosion protection.	• High to Very High	• Preloading may still be required in some high fill areas.

NOTES:
1. The estimated relative cost factor represents an approximately simplified cost estimate for each option divided by the estimated cost for the least expensive option (e.g., a relative cost factor of 2 indicates that the foundation option is twice as costly as the least expensive option).
2. Table to be read in conjunction with accompanying report.

Prepared By:	NG
Checked By:	SJB

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
FoS	factor of safety

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	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_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
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

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* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

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
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
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.) drive open 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, 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 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) Non-Cohesive (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	**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.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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London, Ontario

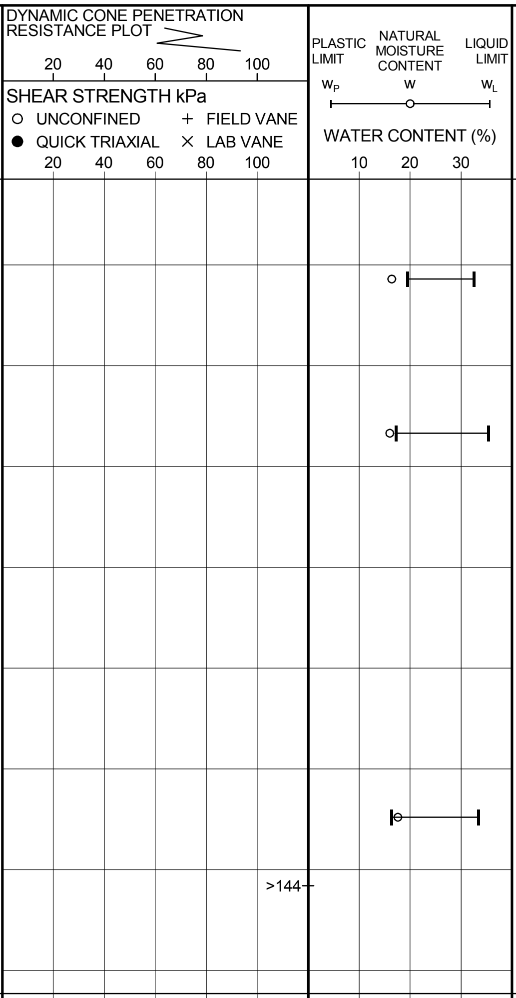
RECORD OF BOREHOLE No 403

1 OF 1

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693778.7 , E 338163.1
ORIGINATED BY MA
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 5, 2014
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
180.85	GROUND SURFACE											
0.00	TOPSOIL, silt, roots Brown											
0.12	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Brown					180						
		1	SS	14					3	20	47	30
		2	SS	16		179						
178.72 2.13	SILTY CLAY TILL, some sand, trace gravel Stiff to very stiff Brown to grey below about elev. 178.3m											
		3	SS	17		178			3	15	43	39
		4	SS	12		177						
		5	SS	13								
		6	SS	11		176						
175.67 5.18	CLAYEY SILT TILL, some sand, trace gravel Firm to very stiff Grey											
		7	SS	5		175						
		8	SS	6		174			1	18	44	37
							>144					
		9	SS	7		173						
172.77 8.08	END OF BOREHOLE Groundwater not established during drilling on June 5, 2014.											



+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 06/11/14

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 601

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693861.1 , E 338355.6
ORIGINATED BY SL
DIST HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 6, 2014 - June 9, 2014
CHECKED BY

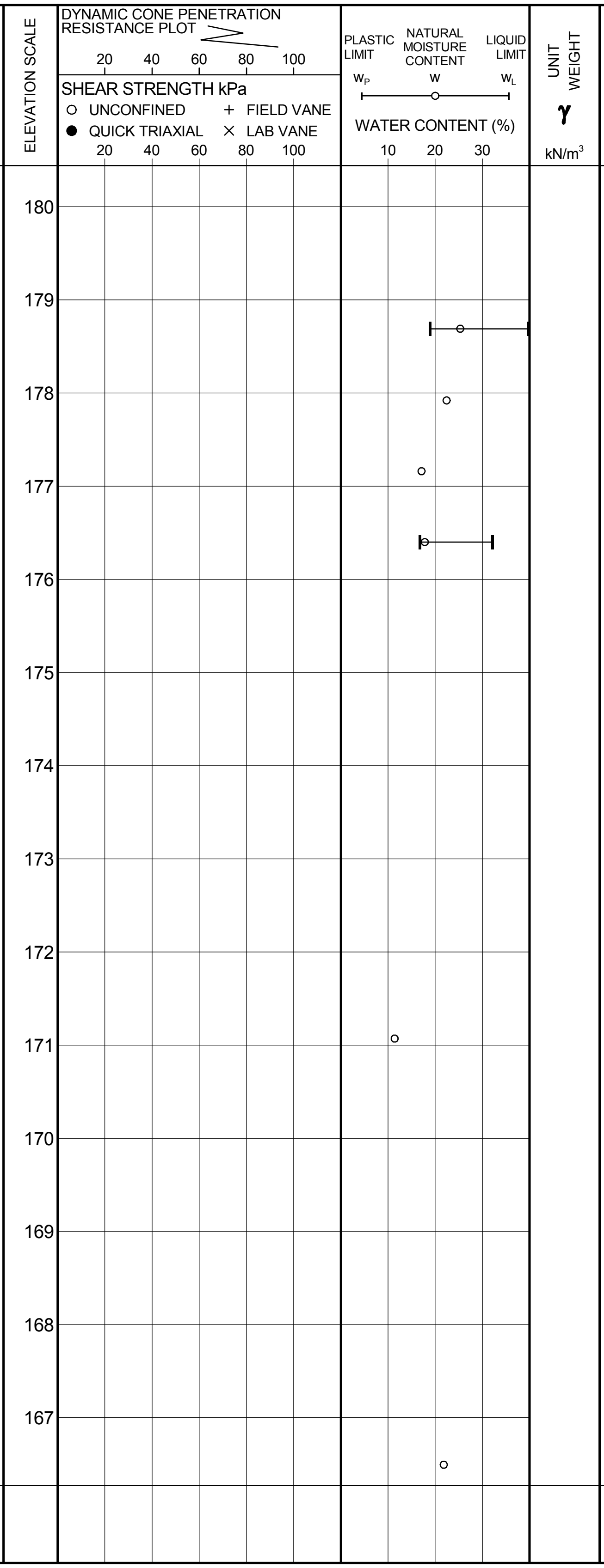
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
180.44	GROUND SURFACE
0.00	TOPSOIL, clayey silt, some sand, roots
0.30	Brown
0.52	FILL, sandy silt, some clay, trace gravel Grey
	FILL, silty clay, trace to some sand, trace gravel, roots Firm Brown and grey
178.31 2.13	FILL, sand and gravel, some silt Dark brown Loose
177.70 2.74	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey
173.12 7.32	SAND, some gravel Compact Grey
172.67 7.77	CLAYEY SILT TILL, sandy, trace gravel Stiff Grey
171.60 8.84	SANDY SILT TILL, some gravel, trace to some clay Compact Grey
167.33 13.11	SILT, some sand, trace to some clay, trace gravel Very dense Grey
166.27 14.17	END OF BOREHOLE Groundwater encountered at about elev. 178.3m during drilling on June 6, 2014.

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	8				
2	SS	7	0	6	56	38
3	SS	7				
4	SS	20				
5	SS	16	1	17	45	37
6	SS	13				
7	SS	18				
8	SS	14				
9	SS	14				
10	SS	18	16	56	21	7
11	SS	26				
12	SS	19				
13	SS	84	1	20	71	8

GROUND WATER CONDITIONS ∇ (elev. approx. 178.3)



REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

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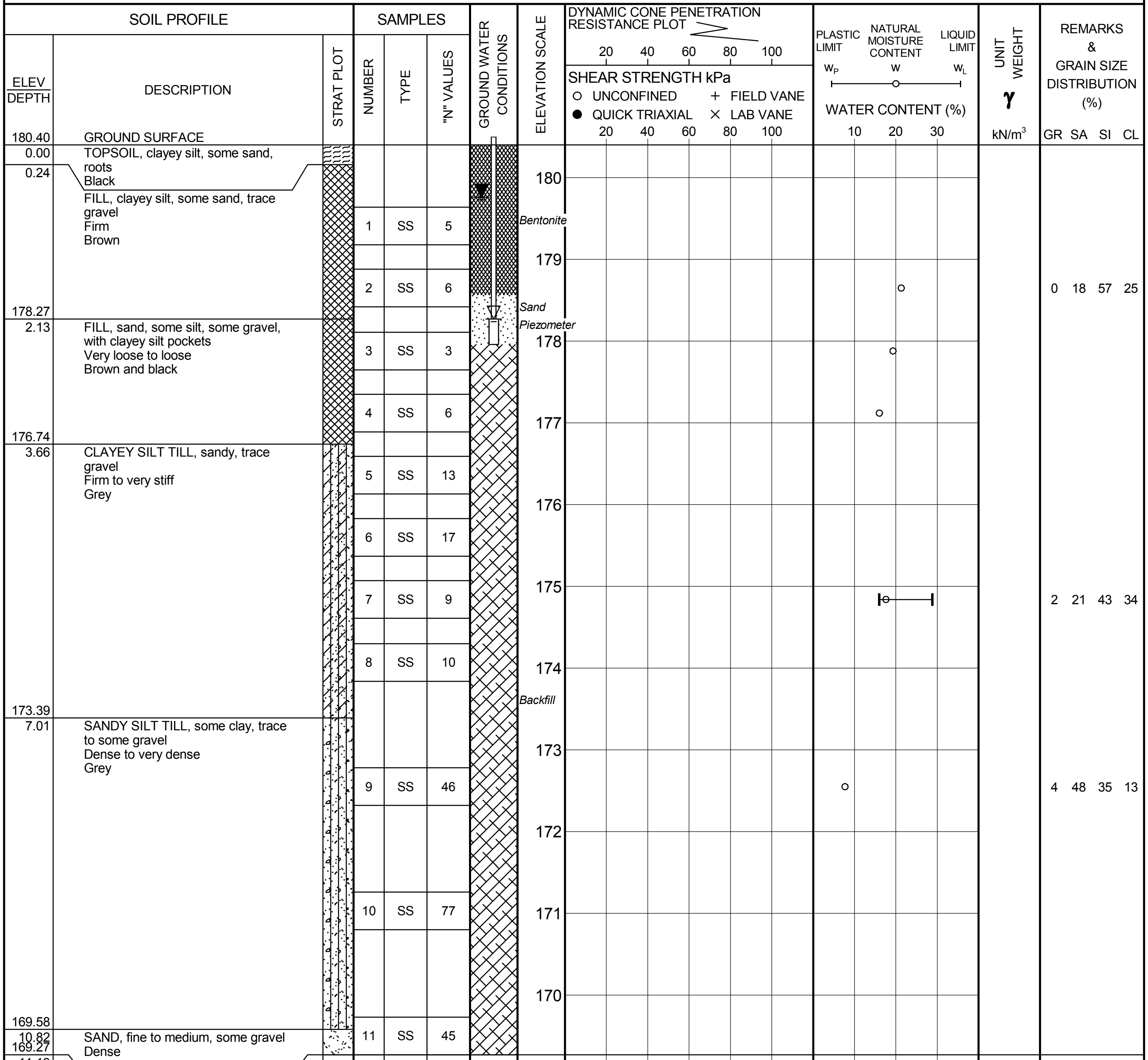
RECORD OF BOREHOLE No 602

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
DIST ______ HWY 401
DATUM GEODETIC
LOCATION N 4693845.8 , E 338407.1
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
DATE June 9, 2014
ORIGINATED BY SL
COMPILED BY WDF
CHECKED BY ______



ELEV DEPTH	DESCRIPTION
180.40	GROUND SURFACE
0.00	TOPSOIL, clayey silt, some sand, roots Black
0.24	FILL, clayey silt, some sand, trace gravel Firm Brown
178.27 2.13	FILL, sand, some silt, some gravel, with clayey silt pockets Very loose to loose Brown and black
176.74 3.66	CLAYEY SILT TILL, sandy, trace gravel Firm to very stiff Grey
173.39 7.01	SANDY SILT TILL, some clay, trace to some gravel Dense to very dense Grey
169.58 10.82	SAND, fine to medium, some gravel Dense Grey
169.27 11.13	END OF BOREHOLE

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	5				
2	SS	6	0	18	57	25
3	SS	3				
4	SS	6				
5	SS	13				
6	SS	17				
7	SS	9	2	21	43	34
8	SS	10				
9	SS	46	4	48	35	13
10	SS	77				
11	SS	45				

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Groundwater encountered at about elev. 178.3m during drilling on June 9, 2014.

Water level measured in Piezometer at elev. 178.27m following installation on June 9, 2014.

Water level measured in Piezometer at elev. 179.70m on June 11, 2014.

Water level measured in Piezometer at elev. 179.58m on June 16, 2014.

Water level measured in Piezometer at elev. 179.76m on July 9, 2014.

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

$○^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 603

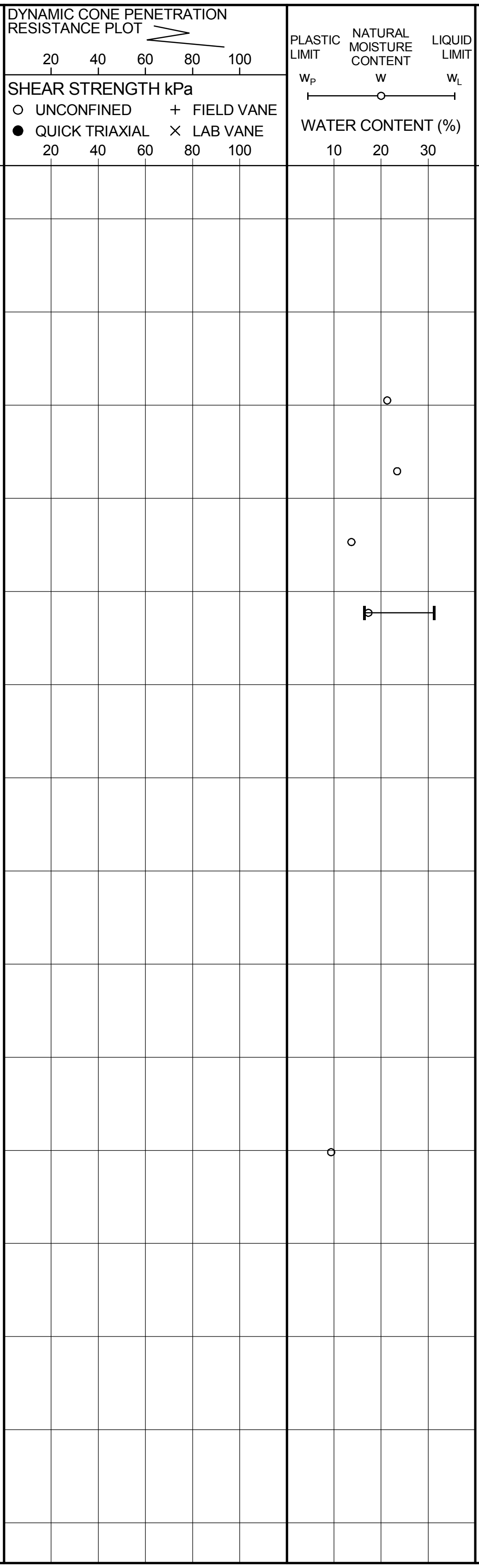
1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693846.9 , E 338293.6
ORIGINATED BY SL
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE
COMPILED BY WDF
DATUM GEODETIC
DATE June 12, 2014
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
180.57	GROUND SURFACE										
0.00	TOPSOIL, clayey silt, some sand, roots Dark brown										
0.24	FILL, clayey silt, some sand, trace gravel, roots Firm Brown and grey					180					
		1	SS	8							
						179					
		2	SS	4							
178.44 2.13	FILL, sandy silt, some clay Loose Brown and grey										
		3	SS	4		178		0	30	51	19
177.67 2.90	FILL, silty sand, some gravel, with clayey silt pockets Very loose Grey and black										
		4	SS	2							
176.91 3.66	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Grey					177					
		5	SS	19							
						176					
		6	SS	15				1	20	44	35
		7	TO	PH	▽	175					
		8	SS	14							
						174					
173.71 6.86	CLAYEY SILT, trace sand, silt pockets Stiff to very stiff Grey							Possible methane gas pocket near elev. 173.9m			
		9	SS	12		173					
						172					
171.58 8.99	SANDY SILT TILL, some clay, trace gravel Compact Grey	10	SS	24							
						171					
170.51 10.06	SAND AND GRAVEL, trace silt Compact Grey										
		11	SS	23		170		61	37		(2)
169.60 10.97	SANDY SILT TILL, some clay, trace gravel Compact to very dense Grey										
						169					
		12	SS	28							
						168					
166.91 13.66	SILTY FINE SAND Very dense Grey	13	SS	92		167					
166.55 14.02	SANDY SILT TILL, some clay, trace gravel Very dense Grey										
						166					

DYNAMIC CONE PENETRATION RESISTANCE PLOT
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%)



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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 603

2 OF 2 **METRIC**

PROJECT 13-1132-0111			
W.P. 3093-09-00	LOCATION N 4693846.9 , E 338293.6		ORIGINATED BY SL
DIST	HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE	COMPILED BY WDF
DATUM GEODETIC		DATE June 12, 2014	CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
	SANDY SILT TILL, some clay, trace gravel Very dense Grey		14	SS	111		165		○		4	39	41	16
163.78 16.79	SILT, trace sand, trace clay Very dense Grey		15	SS	101		164				Possible methane gas pocket near elev. 164.1m			
16.92	END OF BOREHOLE Groundwater encountered at about elev. 174.8m during drilling on June 12, 2014.													

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

$\circ^{3\%}$ STRAIN AT FAILURE

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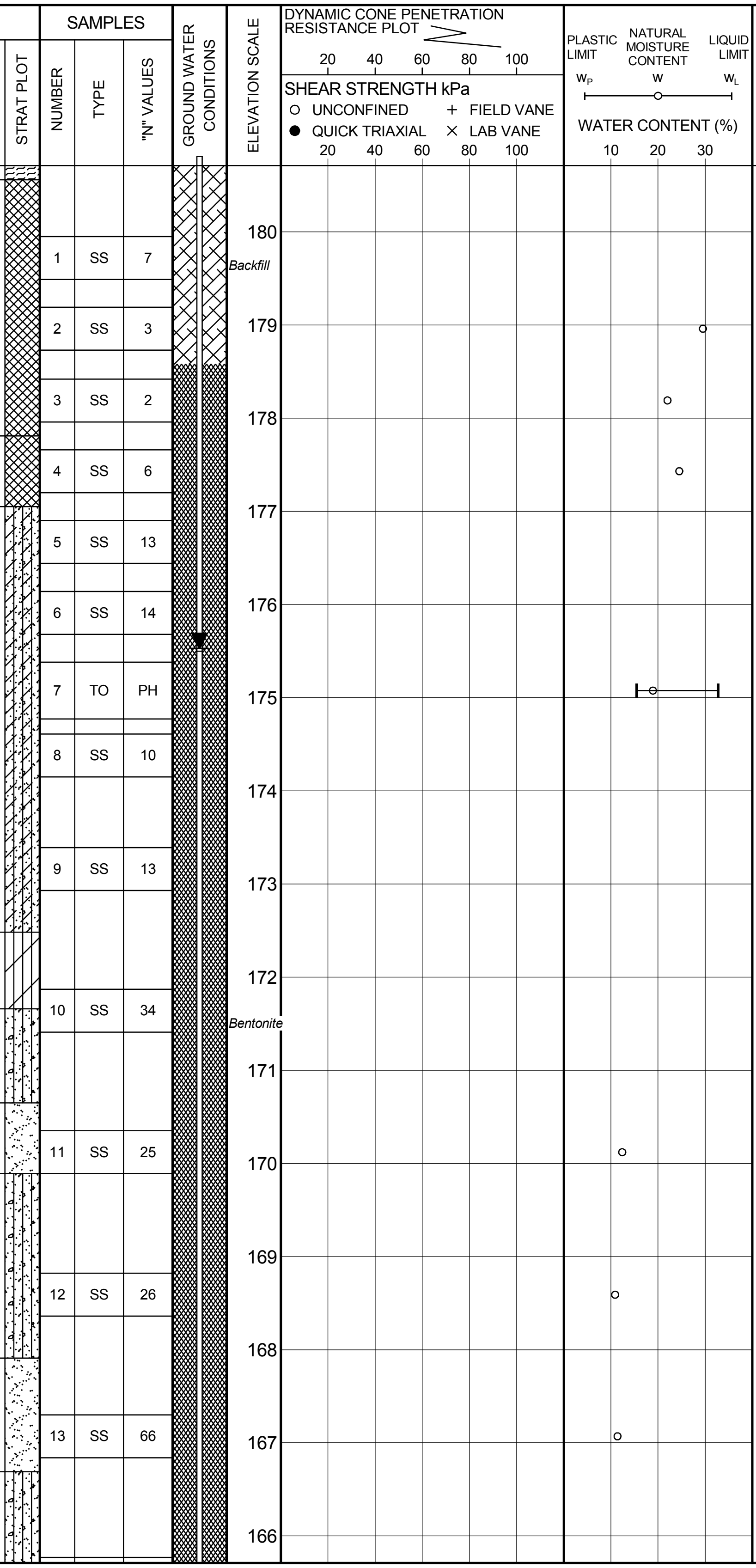
London, Ontario

RECORD OF BOREHOLE No 604

1 OF 2 **METRIC**

PROJECT	13-1132-0111		
W.P.	3093-09-00	LOCATION	N 4693799.9 , E 338254.4
DIST		HWY	401
BOREHOLE TYPE	POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE		
DATUM	GEODETIC	DATE	June 12, 2014 - June 13, 2014
ORIGINATED BY	SL	COMPILED BY	WDF
CHECKED BY			

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
180.71	GROUND SURFACE								
0.00	TOPSOIL, clayey silt, some sand, roots Black								
0.15	FILL, clayey silt, some sand, trace gravel, roots Soft to firm Brown								
		1	SS	7					
		2	SS	3		0	16	62	22
		3	SS	2					
177.81 2.90	FILL, silty sand, some clay, trace gravel Loose Grey								
		4	SS	6					
177.05 3.66	CLAYEY SILT TILL, some sand, trace gravel Stiff Grey								
		5	SS	13					
		6	SS	14					
		7	TO	PH	20.8	C			
		8	SS	10					
		9	SS	13					
172.48 8.23	CLAYEY SILT, some sand, silt pockets Hard Grey								
171.66 9.05	SANDY SILT TILL, some clay, trace gravel Dense Grey	10	SS	34					
170.65 10.06	SAND, some silt, some gravel, trace to some clay Compact Grey	11	SS	25		16	56	18	10
169.89 10.82	SANDY SILT TILL, some clay, trace gravel Compact Grey								
		12	SS	26		1	43	40	16
167.91 12.80	SAND, medium to coarse, trace to some gravel, trace to some silt Very dense Grey								
		13	SS	66		8	82	(10)	
166.69 14.02	SANDY SILT TILL, some clay, trace gravel Very dense Grey								



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+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 07/01/15

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 604

2 OF 2 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693799.9 , E 338254.4
ORIGINATED BY SL
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE
COMPILED BY WDF
DATUM GEODETIC
DATE June 12, 2014 - June 13, 2014
CHECKED BY ____

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES						
	SANDY SILT TILL, some clay, trace gravel Very dense Grey	14	SS	120		165				
164.25 16.46	SAND, fine, some silt, sandy silt seams Very dense Grey	15	SS	78		164				
163.34 17.37	SAND AND GRAVEL, trace to some silt Very dense Grey	16	SS	50/ 75mm	Sand	163 162				
		17	SS	100	Piezometer	161		○		27 61 (12)
160.75 19.96	END OF BOREHOLE Groundwater not established during drilling on June 12, 2014. Water level measured in Piezometer at elev. 175.44m on June 16, 2014. Water level measured in Piezometer at elev. 173.64m on June 20, 2014. Water level measured in Piezometer at elev. 175.53m on July 9, 2014.									

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

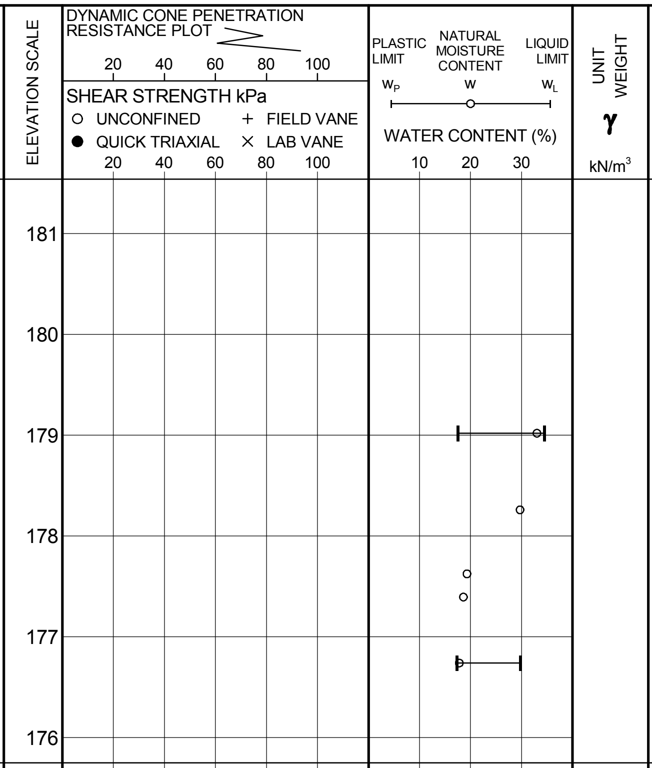
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 605

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693782.8 , E 338259.8
ORIGINATED BY SL
DIST HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 4, 2014
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
181.54	GROUND SURFACE								
0.03	TOPSOIL, silty sand, some gravel, roots Black								
0.24	FILL, silty sand and gravel Brown								
	FILL, clayey silt, some sand, some gravel, black sand pockets Firm Brown	1	SS	5					
180.17 1.37	FILL, silt, some sand Loose Grey	2	SS	7					
179.41 2.13	FILL, clayey silt, sandy, trace gravel Firm Grey	3	SS	5		3	30	44	23
178.64 2.90	FILL, silty sand and gravel, with silty clay pockets, plastic and wood Compact Grey	4	SS	16					
177.52 4.02	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey	5	SS	11	▼				
		6	SS	15		1	19	46	34
		7	SS	18					
175.75 5.79	END OF BOREHOLE Groundwater not established during drilling on June 4, 2014. Water level in open borehole at elev. 177.46m following drilling on June 4, 2014.								



REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

$\circ^{3\%}$ STRAIN AT FAILURE

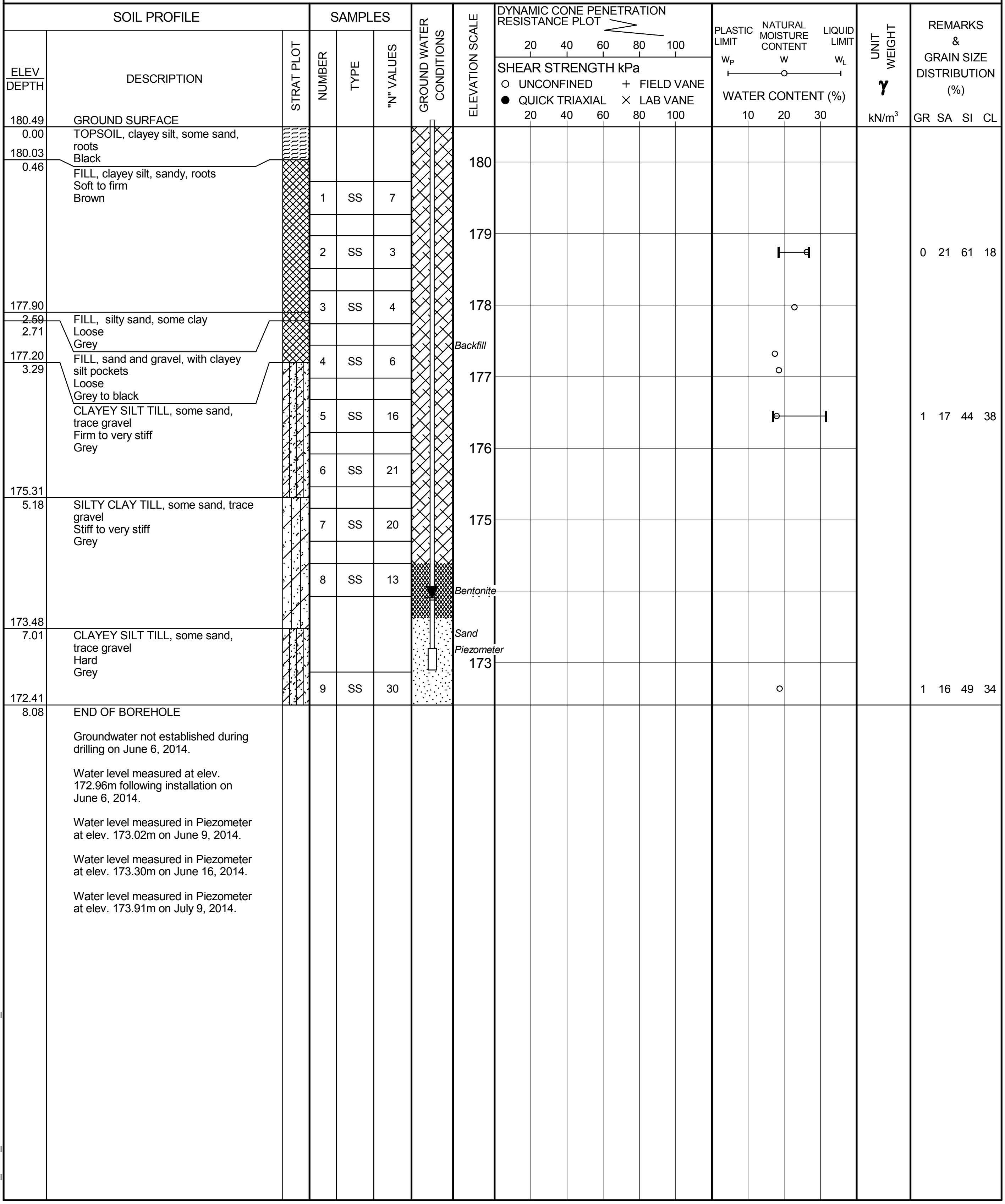
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 606

1 OF 1

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693847.9 , E 338325.5
ORIGINATED BY SL
DIST HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 6, 2014
CHECKED BY



ELEV DEPTH	SOIL PROFILE DESCRIPTION
180.49	GROUND SURFACE
0.00	TOPSOIL, clayey silt, some sand, roots Black
180.03 0.46	FILL, clayey silt, sandy, roots Soft to firm Brown
177.90 ~~2.59~~ 2.71	FILL, silty sand, some clay Loose Grey
177.20 3.29	FILL, sand and gravel, with clayey silt pockets Loose Grey to black
	CLAYEY SILT TILL, some sand, trace gravel Firm to very stiff Grey
175.31 5.18	SILTY CLAY TILL, some sand, trace gravel Stiff to very stiff Grey
173.48 7.01	CLAYEY SILT TILL, some sand, trace gravel Hard Grey
172.41 8.08	END OF BOREHOLE

SAMPLES NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	7				
2	SS	3	0	21	61	18
3	SS	4				
4	SS	6				
5	SS	16	1	17	44	38
6	SS	21				
7	SS	20				
8	SS	13				
9	SS	30	1	16	49	34

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Groundwater not established during drilling on June 6, 2014.

Water level measured at elev. 172.96m following installation on June 6, 2014.

Water level measured in Piezometer at elev. 173.02m on June 9, 2014.

Water level measured in Piezometer at elev. 173.30m on June 16, 2014.

Water level measured in Piezometer at elev. 173.91m on July 9, 2014.

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 06/11/14

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

RECORD OF BOREHOLE No 607

1 OF 1 **METRIC**

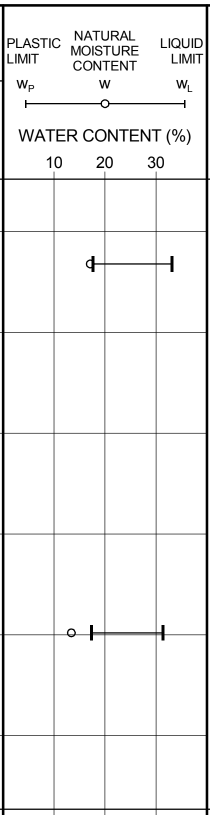
PROJECT 13-1132-0111
W.P. 3093-09-00
DIST
HWY 401
DATUM GEODETIC
LOCATION N 4693662.4 , E 338158.4
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
DATE June 6, 2014
ORIGINATED BY MA
COMPILED BY WDF
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
183.52	GROUND SURFACE									
0.00	TOPSOIL, silt, some sand Brown									
0.27	CLAYEY SILT, sandy Stiff Brown					183				
		1	SS	11			0	21	43	36
182.30 1.22	CLAYEY SILT TILL, some sand, trace gravel Firm to very stiff Grey	2	SS	16		182				
		3	SS	27		181				
		4	SS	20		180				
		5	SS	20						
		6	SS	14		179	1	15	44	40
		7	SS	12		178				
177.27		8	SS	7						
6.25	END OF BOREHOLE Groundwater not established during drilling on June 6, 2014.									

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

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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

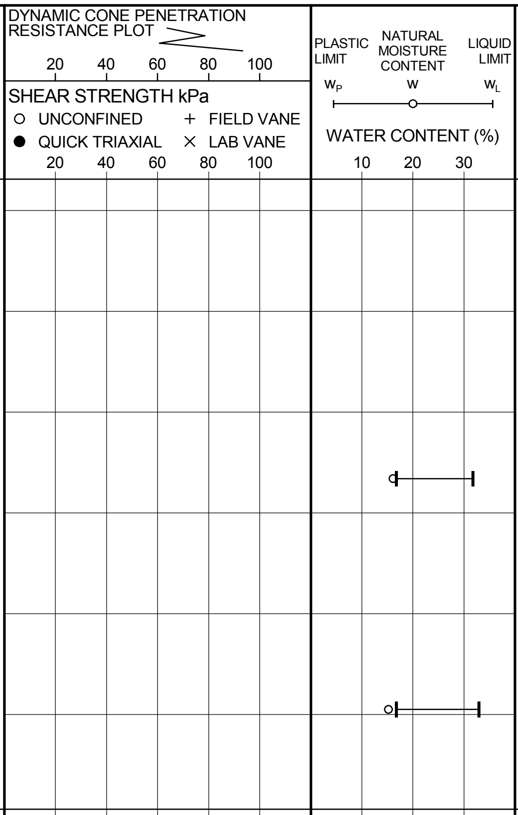
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 608

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693725.6 , E 338144.9
ORIGINATED BY MA
DIST HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 6, 2014
CHECKED BY

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
183.31	GROUND SURFACE							
0.09	TOPSOIL, sand, some gravel, roots							
182.85 0.46	Brown SILTY SAND, some topsoil, roots							
0.70	Brown SILT, some sand Loose	1	SS	7				
182.09 1.22	Brown CLAYEY SILT, trace sand, sandy silt seams Firm	2	SS	17				
	Brown CLAYEY SILT TILL, some sand, trace gravel Very stiff Brown to grey below about elev. 180.9m	3	SS	22				
		4	SS	23	2	18	45	35
		5	SS	19				
		6	SS	17				
		7	SS	16	4	14	44	38
177.06		8	SS	18				
6.25	END OF BOREHOLE Groundwater not established during drilling on June 6, 2014.							



UNIT WEIGHT γ kN/m³

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 609

1 OF 1 **METRIC**

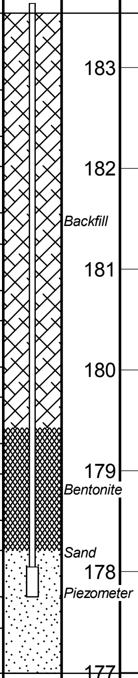
PROJECT 13-1132-0111			
W.P. 3093-09-00	LOCATION N 4693640.2 , E 338104.0	ORIGINATED BY BT
DIST HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY WDF
DATUM GEODETIC	DATE June 27, 2014	CHECKED BY

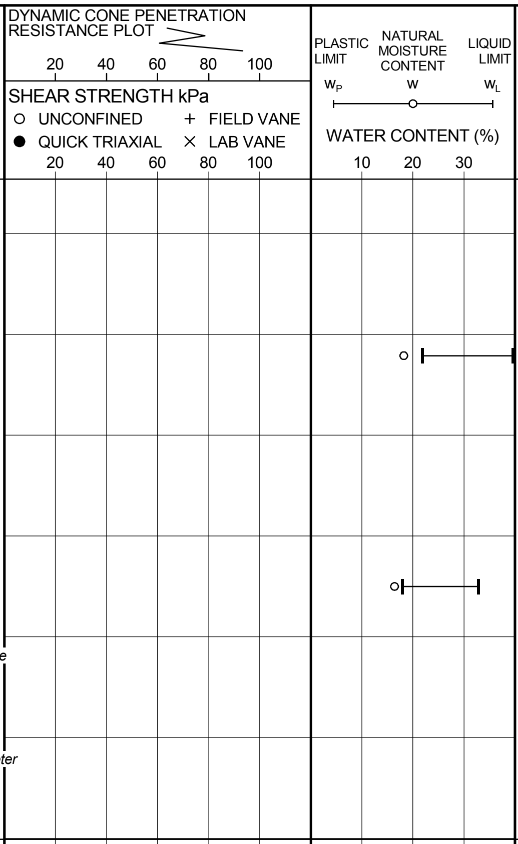
ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
183.54	GROUND SURFACE								
0.00	TOPSOIL, silt Brown								
0.12	SILTY CLAY, trace to some sand, trace gravel Stiff Brown				183				
		1	SS	9					
					182				
		2	SS	8		1	9	58	32
181.41 2.13	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Brown and grey below about elev. 180.6m								
		3	SS	21	181				
		4	SS	14					
					180				
		5	SS	13		4	16	45	35
					179				
		6	SS	12					
		7	SS	12	178				
		8	SS	9					
176.99 6.55	END OF BOREHOLE				177				

Groundwater not established during drilling on June 27, 2014.

Piezometer dry to elev. 177.75m on June 27, 2014.

Piezometer dry to elev. 177.75m on July 9, 2014.





UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 610

1 OF 1 **METRIC**

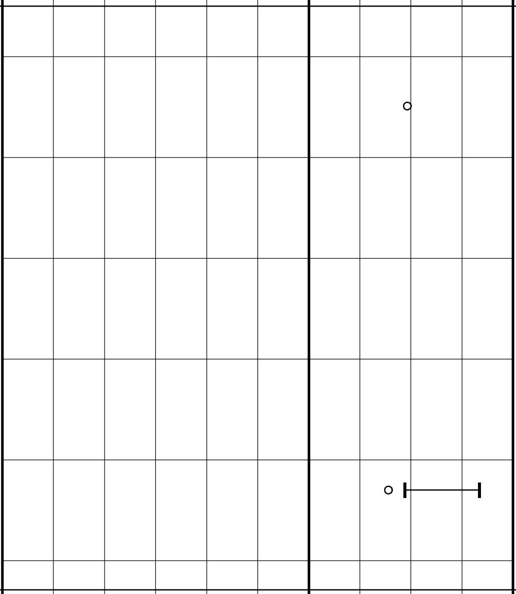
PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693630.3 , E 338064.0
ORIGINATED BY BT
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 27, 2014
CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
183.50	GROUND SURFACE									
0.00	TOPSOIL, silt									
0.12	Brown CLAYEY SILT, trace to some sand Stiff Brown					183				
		1	SS	8			0	3	73	24
182.13 1.37	SILTY CLAY, trace sand Stiff Brown					182				
		2	SS	14						
181.37 2.13	CLAYEY SILT TILL, some sand, trace to some gravel Stiff to very stiff Brown to grey below about elev. 180.6m	3	SS	16		181				
		4	SS	13		180				
		5	SS	15						
		6	SS	19		179	6	14	45	35
177.71		7	SS	19		178				
5.79	END OF BOREHOLE Groundwater not established during drilling on June 27, 2014.									

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



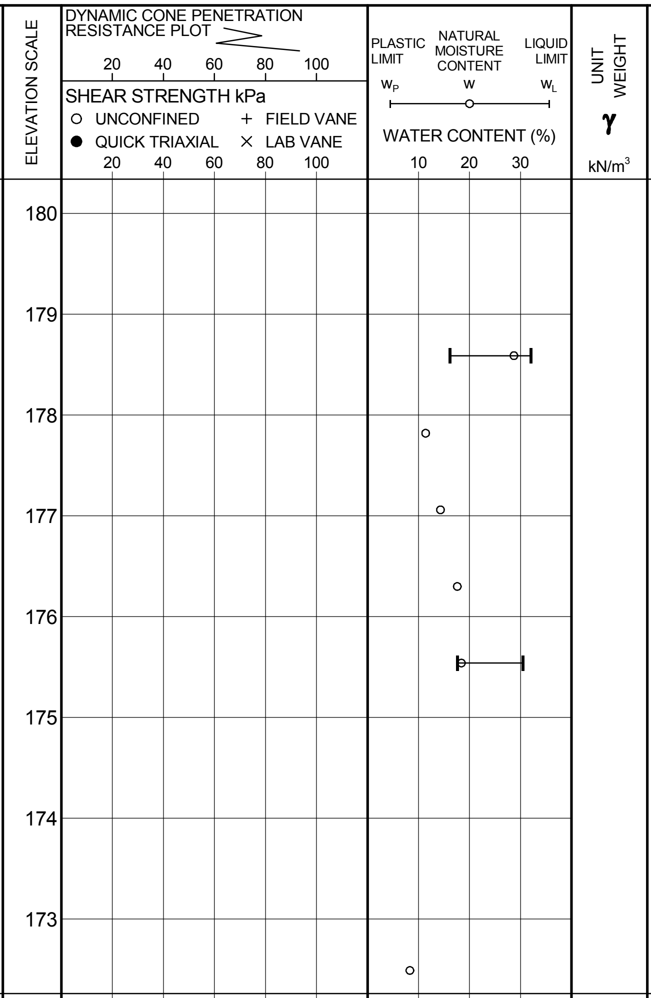
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 611

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693837.8 , E 338376.2
ORIGINATED BY SL
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 9, 2014
CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
180.34	GROUND SURFACE								
0.00	FILL, silty clay, some sand, some gravel Firm Brown	1	SS	6	▼				
179.18 1.16	FILL, silty fine sand Loose Brown								
178.76 1.58	FILL, silty sand and gravel, with clayey silt pockets Loose Dark brown to black	2	SS	7		0	19	51	30
178.21 2.13	FILL, silty sand, some gravel, roots, wood, with clayey silt pockets Very loose to loose Dark grey to black	3	SS	9					
		4	SS	3					
176.68 3.66	CLAYEY SILT TILL, some sand, trace gravel Very stiff Grey	5	SS	15					
		6	SS	20		4	18	43	35
		7	SS	16					
		8	SS	15					
173.85 6.49	SANDY SILT TILL, trace to some gravel, trace to some clay Compact to very dense Grey	9	SS	86		7	55	30	8
172.26 8.08	END OF BOREHOLE Groundwater not established during drilling on June 9, 2014. Water level in open borehole at elev. 179.88m following drilling on June 9, 2014.								



$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

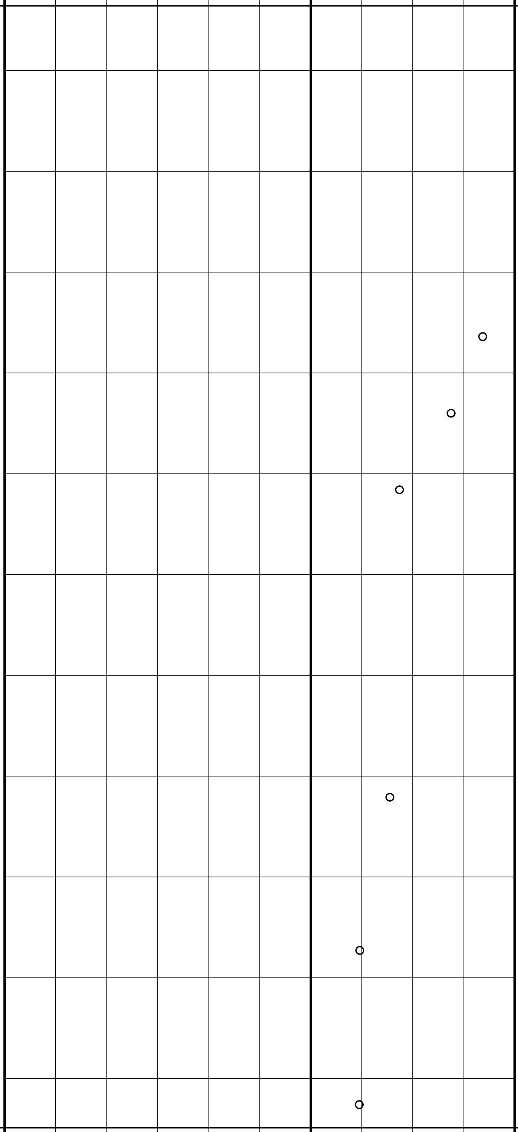
RECORD OF BOREHOLE No 612

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693802.9 , E 338289.5
ORIGINATED BY SL
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE June 5, 2014
CHECKED BY

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
180.64	GROUND SURFACE								
0.00	TOPSOIL, clayey silt, some sand, trace gravel, roots Dark brown								
180.24 0.40	FILL, clayey silt, some sand, some gravel, roots Firm Brown and grey	1	SS	7					
179.27 1.37	FILL, clayey silt, some sand, wood, black sand seams Firm to stiff Brown to black	2	SS	9	▼				
		3	SS	5	▽				
177.44 3.20	FILL, fine to coarse sand, trace gravel, large wood pieces, with clayey silt and silty clay pockets Loose Black	4	SS	4					
		5	SS	6					
176.22 4.42	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey	6	SS	18					
		7	SS	17					
		8	SS	14					
		9	SS	22		0	17	46	37
172.11 8.53	SILTY SAND, trace to some gravel, trace clay Compact Grey	10	SS	10		6	70	20	4
170.58 10.06	SANDY SILT TILL, some clay, trace gravel Very dense Grey	11	SS	50		3	43	42	12
169.51 11.13	END OF BOREHOLE Groundwater encountered at about elev. 177.6m during drilling on June 5, 2014. Water level in open borehole at elev. 178.81m following drilling on June 5, 2014.								



$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 613

1 OF 1

METRIC

PROJECT 13-1132-0111

W.P. 3093-09-00

LOCATION N 4693809.4 , E 338368.6

ORIGINATED BY SL

DIST ____ HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

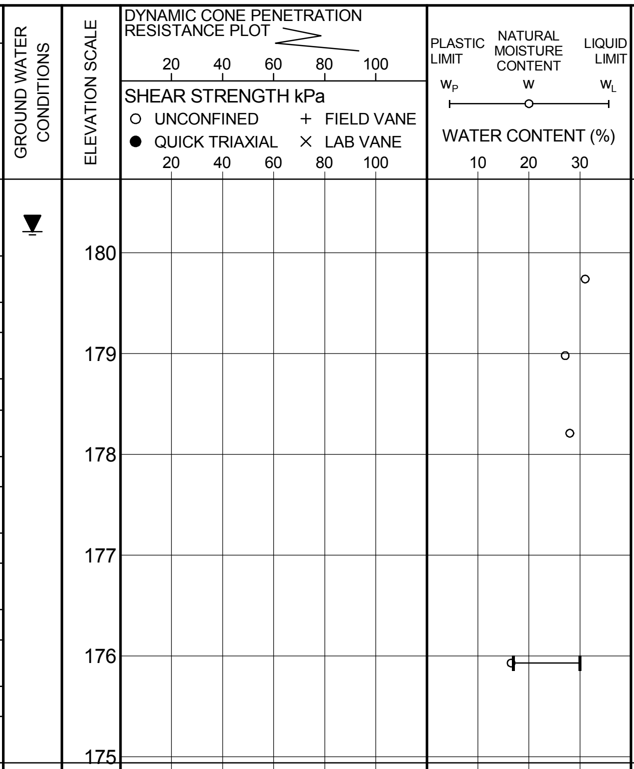
COMPILED BY WDF

DATUM GEODETIC

DATE June 5, 2014

CHECKED BY ____

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
180.73	GROUND SURFACE							
0.09	TOPSOIL, clayey silt, some sand, trace gravel, roots Dark brown							
	FILL, clayey silt, some sand, black silt pockets, trace to some gravel, roots, large wood pieces Soft to firm Brown	1	SS	4	0	14	59	27
		2	SS	5				
		3	SS	3				
177.83 2.90	FILL, silty sand, some gravel, wood, with silty clay pockets Very loose Grey and black	4	SS	2				
176.77 3.96	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Grey	5	SS	9				
		6	SS	16	3	20	43	34
		7	SS	13				
174.94 5.79	END OF BOREHOLE Groundwater not established during drilling on June 5, 2014. Water level in open borehole at elev. 180.21m following drilling on June 5, 2014.							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

$\circ^{3\%}$ STRAIN AT FAILURE

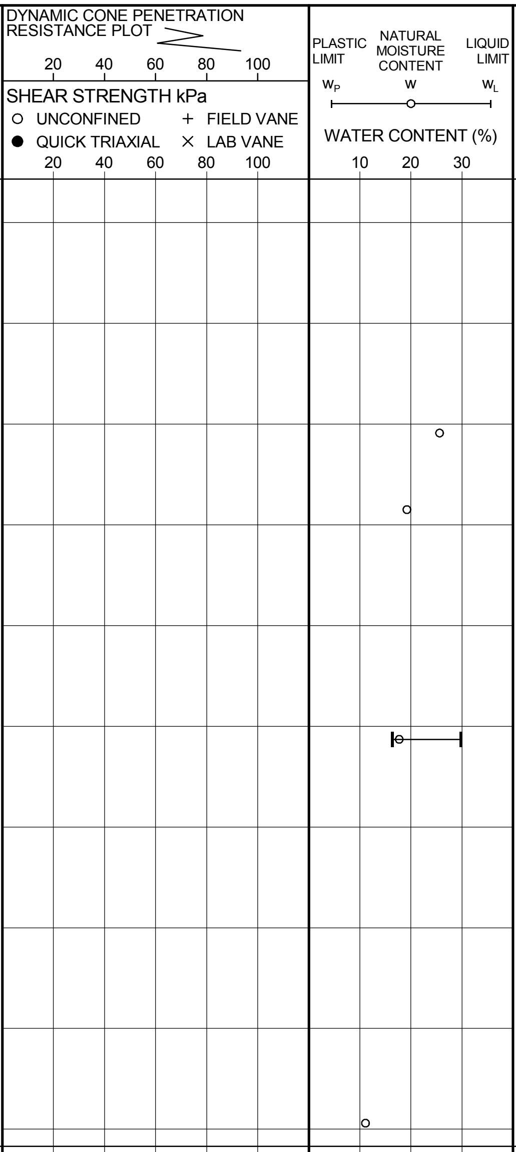
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 614

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
DIST ______ HWY 401
DATUM GEODETIC
LOCATION N 4693823.8 , E 338327.3
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
DATE June 5, 2014
ORIGINATED BY SL
COMPILED BY WDF
CHECKED BY ______

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
180.43	GROUND SURFACE										
0.09	TOPSOIL, silt, some sand, roots Black				▼	180					
	FILL, sandy silty clay Firm Brown	1	SS	4							
179.06 1.37	FILL, silty clay, silty sand seams, roots Firm Grey	2	SS	5		179					
178.30 2.13	FILL, silt and sand, some clay Very loose Dark grey	3	SS	1		178		0	51	35	14
177.53 2.90	FILL, silty sand and gravel, roots Compact Dark grey	4	SS	11		177					
176.62 3.81	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Grey	5	SS	12		176					
		6	SS	13		175					
		7	SS	17				1	21	46	32
		8	SS	12		174					
						173					
172.81 7.62	SANDY SILT TILL, some gravel, some clay Dense Grey	9	SS	31							
7.92	SAND AND GRAVEL, trace to some silt Compact to dense Grey					172					
		10	SS	26		171		45	51		(4)
170.83 9.60	END OF BOREHOLE Groundwater not established during drilling on June 5, 2014. Water level in open borehole at elev. 179.82m following drilling on June 5, 2014.										



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

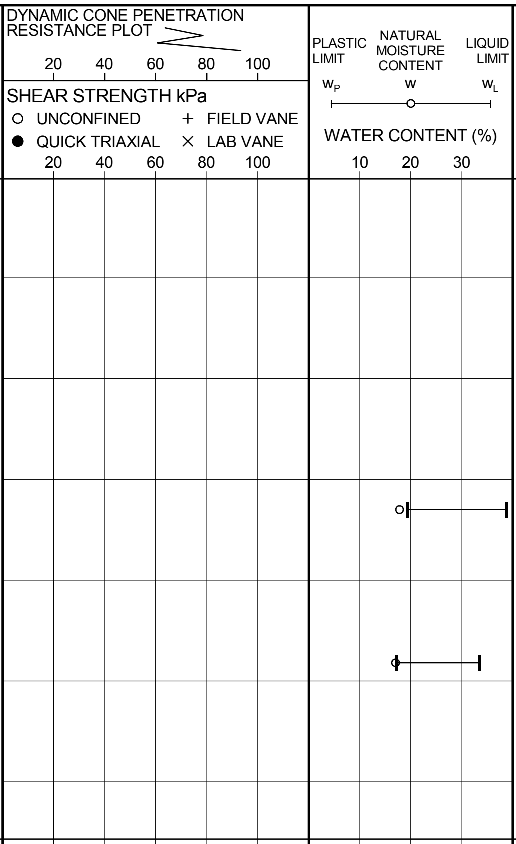
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 615

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693586.7 , E 338473.9
ORIGINATED BY SL
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE May 13, 2014
CHECKED BY ____

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				GR	SA	SI	CL
181.98	GROUND SURFACE											
0.03	VEGETATION											
	FILL, silty clay, trace to some sand, trace gravel, trace organics, roots to about elev. 180.8m Firm to stiff Brown to grey		1	SS	7		181					
			2	SS	8		180					
179.54 2.44	SILTY CLAY TILL, some sand, trace gravel Stiff to very stiff Brown		3	SS	11							
			4	SS	18	▼	179		1	15	50	34
178.32 3.66	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey		5	SS	14		178					
			6	SS	14		177		2	15	50	33
			7	SS	15		176					
175.43 6.55			8	SS	16							
	END OF BOREHOLE Groundwater not established during drilling on May 13, 2014. Water level in open borehole at elev. 178.81m following drilling on May 13, 2014.											



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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 616

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693567.5 , E 338434.9
ORIGINATED BY SL
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE May 16, 2014
CHECKED BY ____

ELEV / DEPTH	DESCRIPTION
182.85	GROUND SURFACE
0.00	FILL, sandy silt, trace clay, trace gravel, roots Dark brown
0.24	CLAYEY SILT TILL, some sand to sandy, trace gravel, with fine sand seams below about elev. 174.6m Stiff to very stiff Brown to grey below about elev. 180.6m
171.72 / 11.13	END OF BOREHOLE

Groundwater not established during drilling on May 16, 2014.

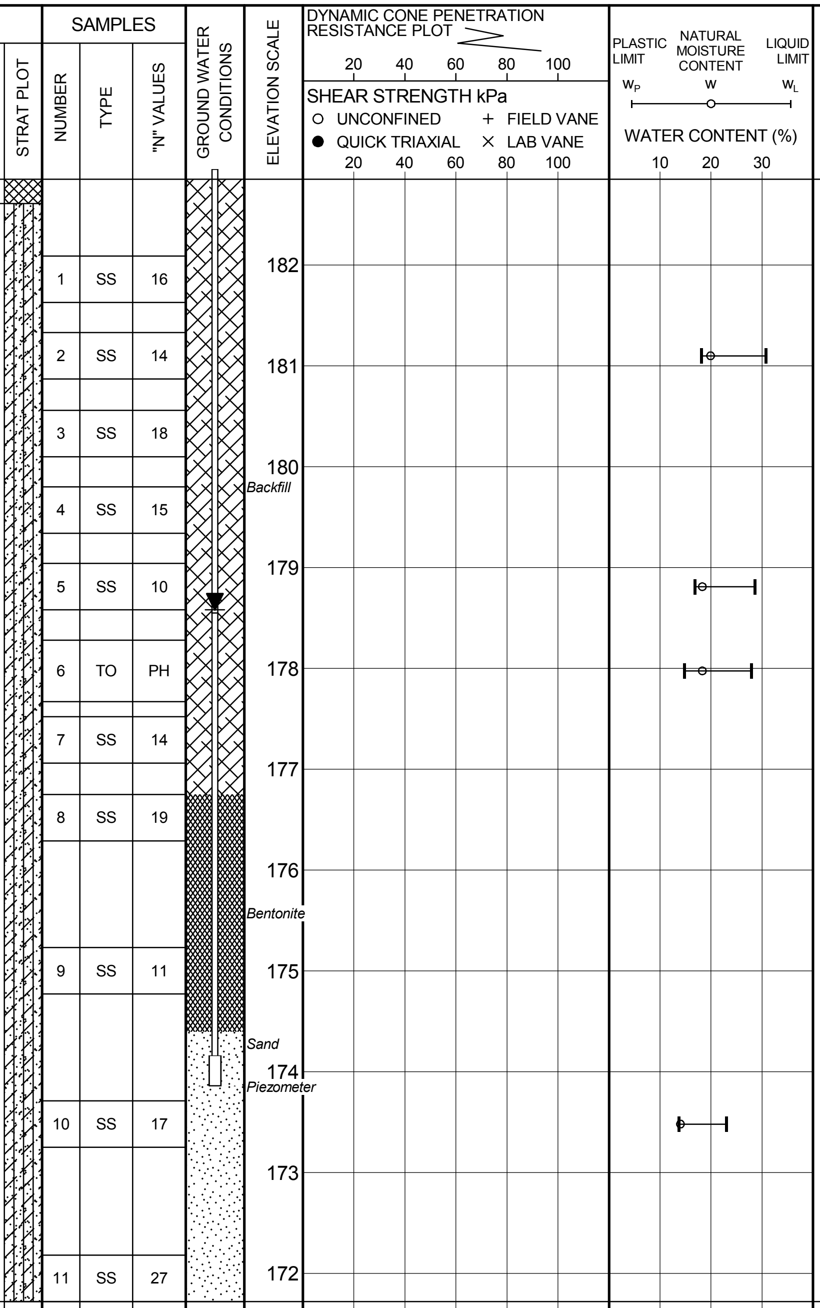
Water level measured in Piezometer at elev. 172.66m on May 16, 2014.

Water level measured in Piezometer at elev. 174.99m on May 21, 2014.

Water level measured in Piezometer at elev. 177.21m on June 4, 2014.

Water level measured in Piezometer at elev. 178.58m on July 9, 2014.

SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	16				
2	SS	14	1	23	49	27
3	SS	18				
4	SS	15				
5	SS	10	1	18	46	35
6	TO	PH	C			
7	SS	14				
8	SS	19				
9	SS	11				
10	SS	17	2	24	42	32
11	SS	27				



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

$\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 617

1 OF 1

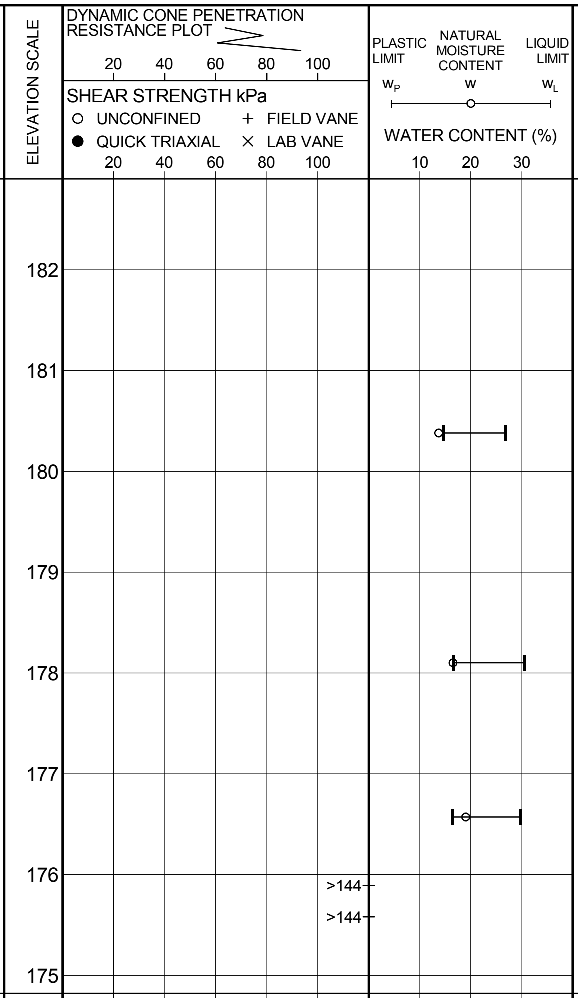
METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693511.6 , E 338357.8
ORIGINATED BY SL
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE May 22, 2014
CHECKED BY ____

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
182.90	GROUND SURFACE							
0.00	FILL, sandy silt, trace gravel, roots Dark brown							
182.59 0.31	FILL, silty fine sand Brown							
0.43	FILL, clayey silt, some sand, trace gravel Stiff Brown	1	SS	10				
		2	SS	12				
180.77 2.13	CLAYEY SILT TILL, some sand to sandy, trace gravel Stiff to very stiff Grey	3	SS	19	1	23	46	30
		4	SS	14				
		5	SS	11				
		6	SS	14	2	17	46	35
		7	SS	10				
		8	SS	10	2	17	45	36
		9	SS	13				
174.82 8.08	END OF BOREHOLE Groundwater not established during drilling on May 22, 2014.							

GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³



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

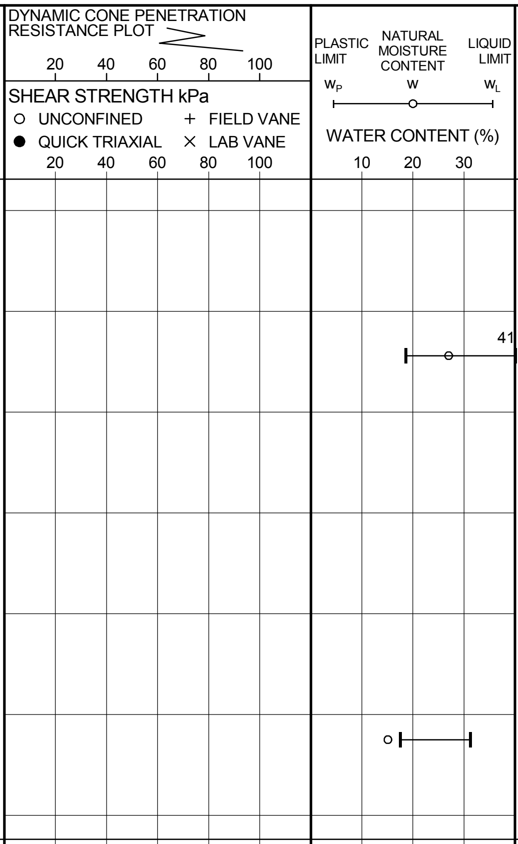
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 618

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693491.2 , E 338296.0
ORIGINATED BY SL
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY LMK
DATUM GEODETIC
DATE May 22, 2014
CHECKED BY ____

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.31	GROUND SURFACE				
0.00	FILL, sandy silt, trace gravel, roots Dark brown				
183.00 0.31	FILL, sandy silt, some clay Compact Brown	1	SS	11	
182.33 0.98	FILL, silty clay, trace sand Stiff Brown	2	SS	12	0 2 36 62
181.18 2.13	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey	3	SS	22	
		4	SS	16	
		5	SS	14	
		6	SS	17	
		7	SS	18	1 18 46 35
		8	SS	15	
176.76 6.55	END OF BOREHOLE Groundwater not established during drilling on May 22, 2014.				



UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

STRAT PLOT

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

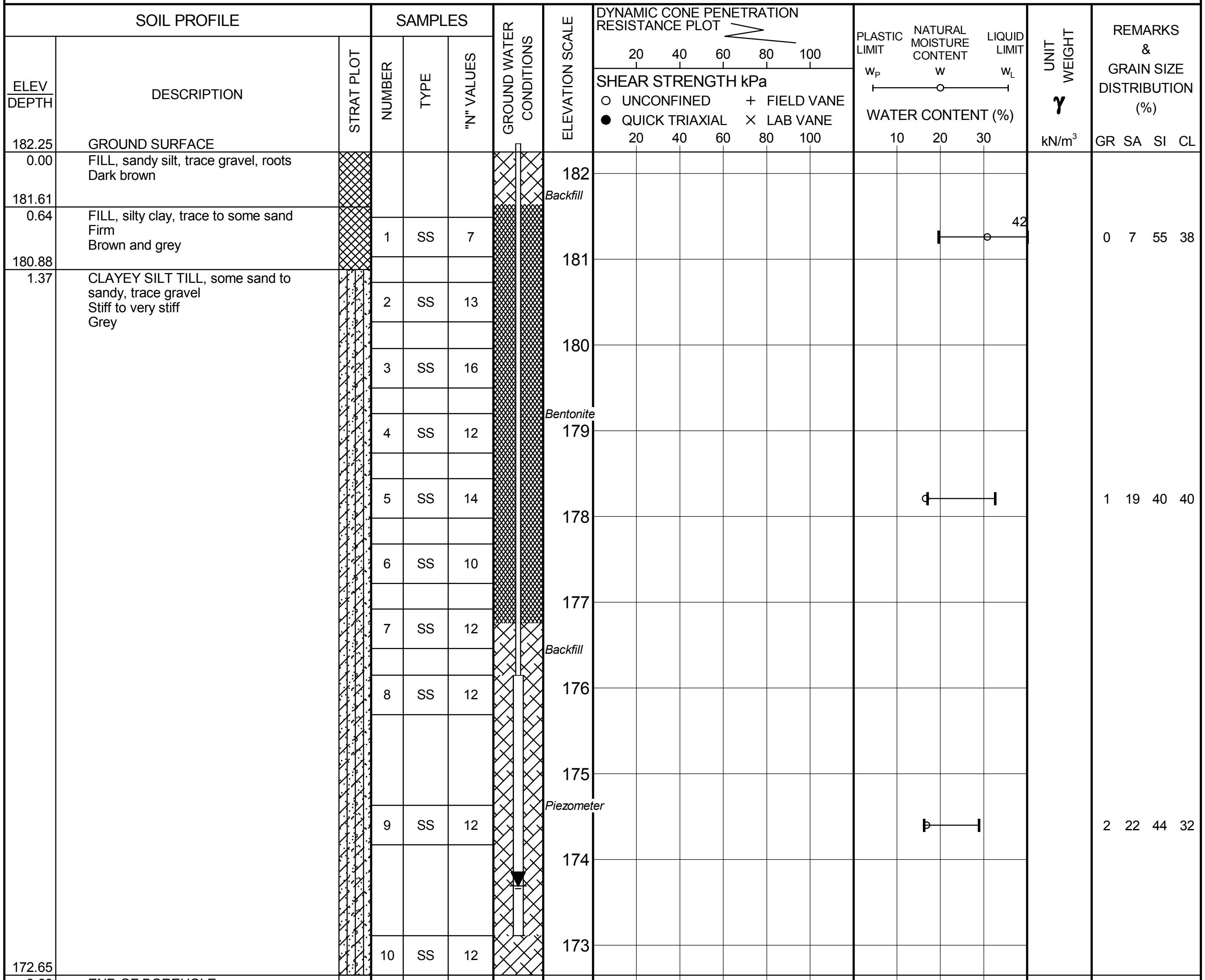
LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 06/11/14

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 619

1 OF 1 **METRIC**

PROJECT 13-1132-0111			
W.P. 3093-09-00	LOCATION N 4693492.4 , E 338398.7	ORIGINATED BY SL	
DIST	HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY LMK
DATUM GEODETIC	DATE May 22, 2014	CHECKED BY	



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
182.25	GROUND SURFACE								
0.00	FILL, sandy silt, trace gravel, roots Dark brown								
181.61 0.64	FILL, silty clay, trace to some sand Firm Brown and grey	1	SS	7		0	7	55	38
180.88 1.37	CLAYEY SILT TILL, some sand to sandy, trace gravel Stiff to very stiff Grey	2	SS	13					
		3	SS	16					
		4	SS	12					
		5	SS	14		1	19	40	40
		6	SS	10					
		7	SS	12					
		8	SS	12					
		9	SS	12		2	22	44	32
172.65		10	SS	12					
9.60	END OF BOREHOLE								

Groundwater not established during drilling on May 22, 2014.

Piezometer dry to elev. 173.08m on May 22 and June 4, 2014.

Water level measured in Piezometer at elev. 173.69m on July 9, 2014.

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

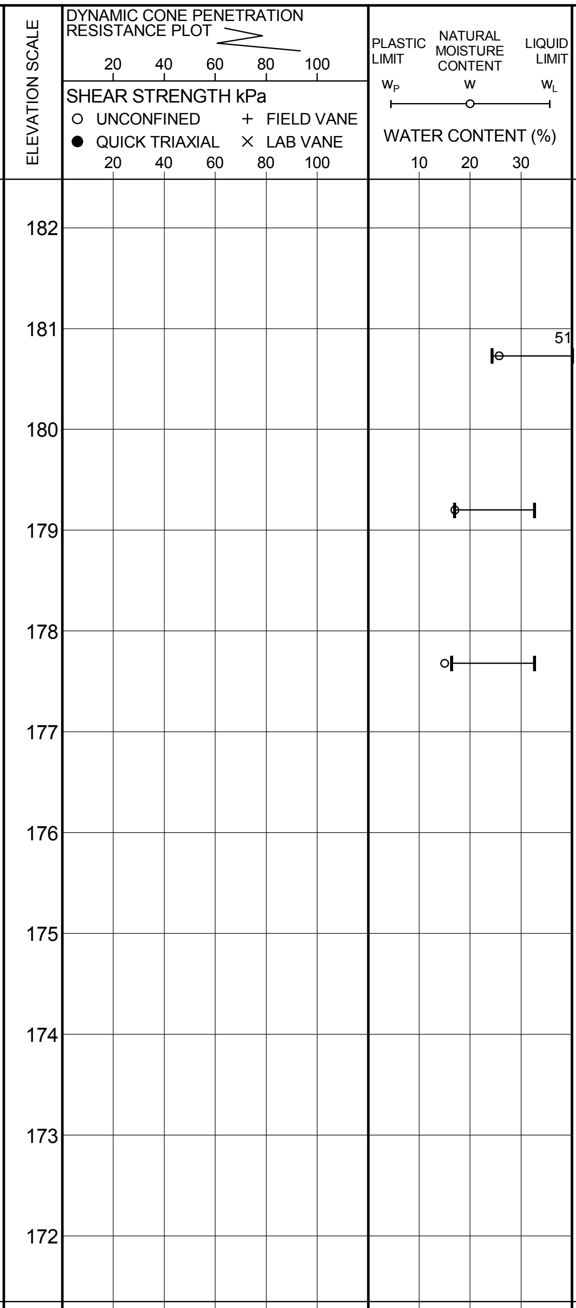
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 620

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
DIST HWY 401
DATUM GEODETIC
LOCATION N 4693508.2 , E 338452.5
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
DATE May 23, 2014
ORIGINATED BY SL
COMPILED BY LMK
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
182.48	GROUND SURFACE							
0.00	FILL, sandy silt, trace gravel, roots Dark brown							
0.27	FILL, sandy silt, trace clay Brown							
0.55	SILTY CLAY, trace sand Stiff Brown and grey	1	SS	14				
		2	SS	14	0	2	33	65
180.35 2.13	CLAYEY SILT TILL, some sand to sandy, trace to some gravel Stiff to very stiff Brown to grey below about elev. 178.8m	3	SS	17				
		4	SS	18	1	29	42	28
		5	SS	13				
		6	SS	12	4	18	44	34
		7	SS	16				
		8	SS	13				
		9	SS	14				
		10	SS	11				
		11	SS	13				
171.35 11.13	END OF BOREHOLE Groundwater not established during drilling on May 23, 2014.							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

Golder Associates
London, Ontario

PROJECT 13-1132-0111

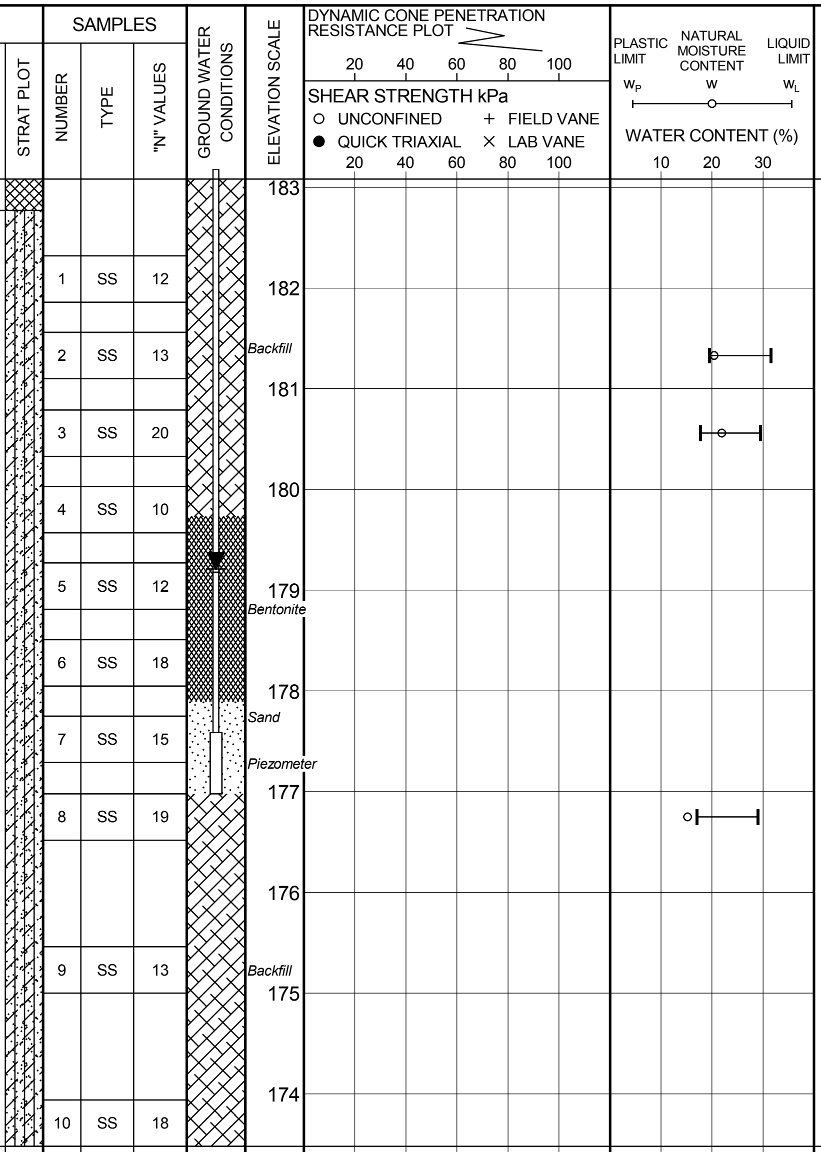
RECORD OF BOREHOLE No 621

1 OF 1

METRIC

W.P. 3093-09-00
LOCATION N 4693610.2 , E 338417.8
ORIGINATED BY SL
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE May 13, 2014
CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
183.08	GROUND SURFACE							
0.00	FILL, sandy silt, some gravel, roots Brown							
182.77 0.31	CLAYEY SILT TILL, some sand to sandy, trace to some gravel Stiff to very stiff Brown and grey below about elev. 179.4m	1	SS	12				
		2	SS	13	1	14	52	33
		3	SS	20	1	24	43	32
		4	SS	10				
		5	SS	12				
		6	SS	18				
		7	SS	15				
		8	SS	19	2	20	42	36
		9	SS	13				
		10	SS	18				
173.48 9.60	END OF BOREHOLE							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Groundwater not established during drilling on May 13, 2014.

Piezometer dry to elev. 176.98m on May 13 and May 15, 2014.

Water level measured in Piezometer at elev. 178.66m on May 21, 2014.

Water level measured in Piezometer at elev. 179.06m on June 4, 2014.

Water level measured in Piezometer at elev. 179.21m on July 9, 2014.

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 622

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693629.3 , E 338375.4
ORIGINATED BY SL
DIST
HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE May 14, 2014
CHECKED BY

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
183.06	GROUND SURFACE
0.00	FILL, sandy silt, some gravel, trace clay, roots Brown
182.24 0.82	FILL, silty clay, trace to some sand Stiff Brown
181.69 1.37	SILTY CLAY TILL, some sand to sandy, trace gravel Stiff to very stiff Brown to grey below about elev. 180.6m
176.51 6.55	END OF BOREHOLE Groundwater not established during drilling on May 14, 2014.

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	10	0	5	31	64
2	SS	12	1	21	44	34
3	SS	15				
4	SS	11				
5	SS	10				
6	SS	17				
7	SS	15				

GROUND WATER CONDITIONS

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

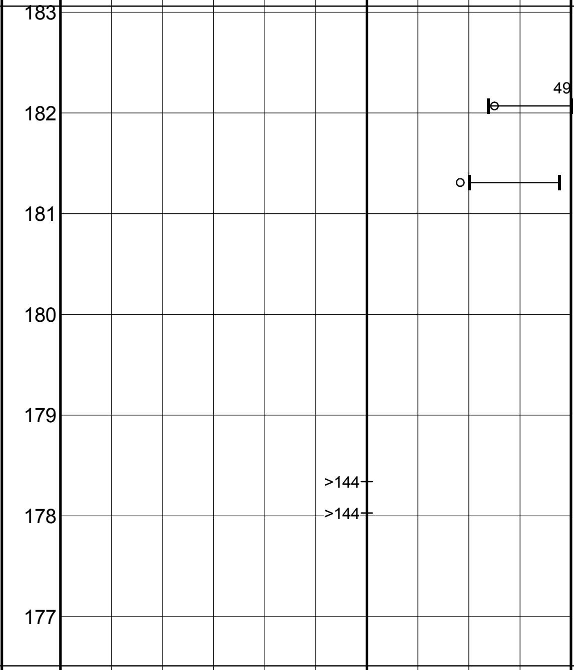
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%)
10 20 30

UNIT WEIGHT γ kN/m³

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



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

$\circ^{3\%}$ STRAIN AT FAILURE

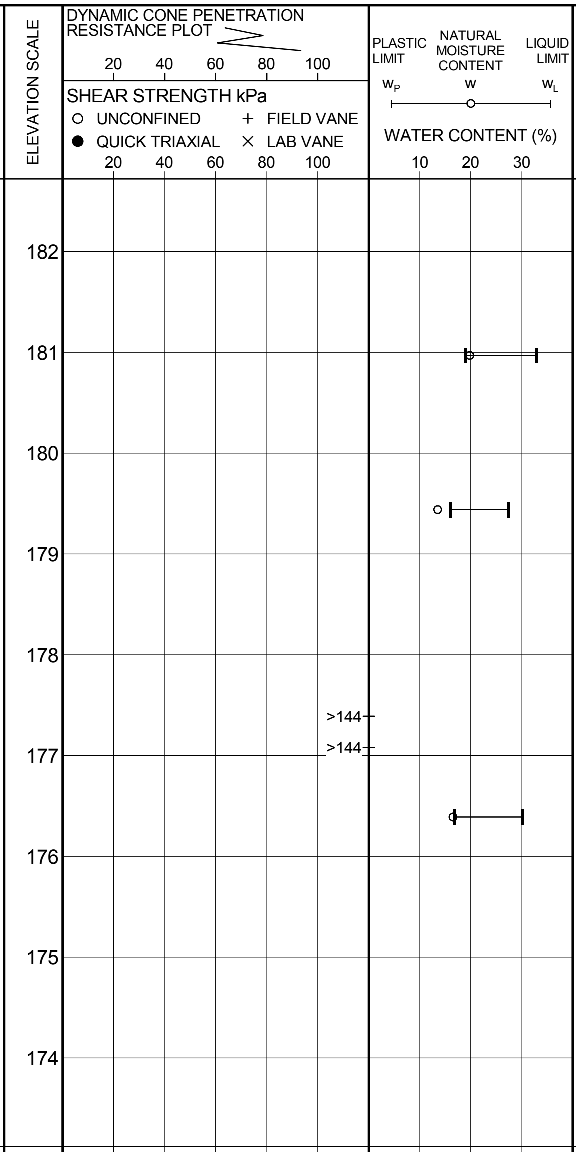
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 623

1 OF 1 **METRIC**

PROJECT 13-1132-0111			
W.P. 3093-09-00	LOCATION N 4693549.6 , E 338380.4		ORIGINATED BY SL
DIST	HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM	COMPILED BY WDF
DATUM GEODETIC		DATE May 15, 2014	CHECKED BY

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
182.72	GROUND SURFACE									
~~0.00~~ 0.12	FILL, silt, some sand, trace clay, roots Dark brown									
182.20 0.52	FILL, sandy silt, roots Brown									
	SILTY CLAY, some sand, trace gravel, roots Stiff Brown	1	SS	10						
181.35 1.37	CLAYEY SILT, some sand, roots Stiff Brown and grey	2	SS	14			0	16	56	28
180.59 2.13	CLAYEY SILT TILL, some sand to sandy, trace gravel Stiff to very stiff Grey	3	SS	14						
		4	SS	14			1	22	44	33
		5	SS	11						
		6	SS	10						
		7	SS	13			1	19	44	36
		8	SS	12						
		9	SS	12						
173.12 9.60	END OF BOREHOLE Groundwater not established during drilling on May 15, 2014.									



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

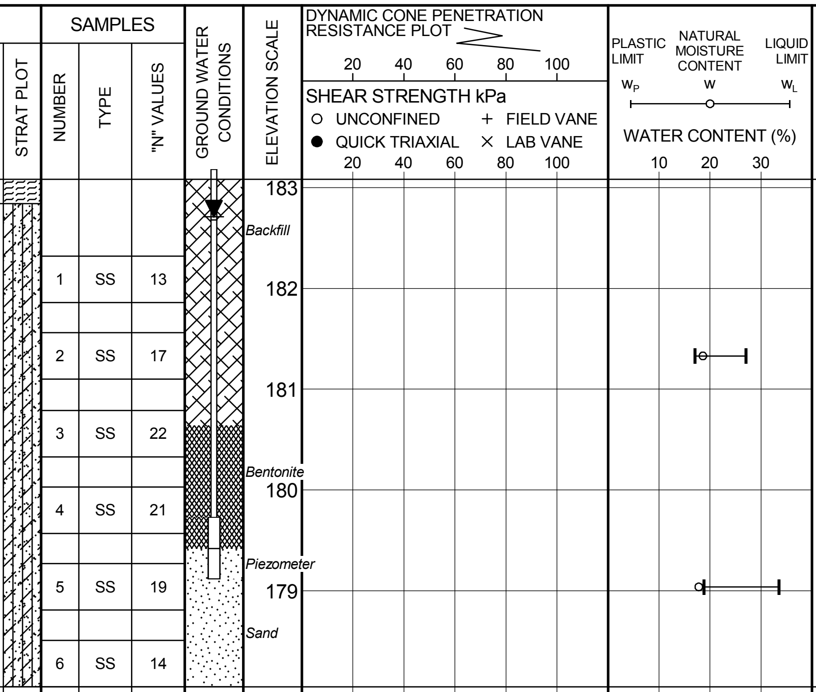
$\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 624

1 OF 1 **METRIC**

PROJECT	13-1132-0111		
W.P.	3093-09-00	LOCATION	N 4693593.6 , E 338543.6
DIST		HWY	401
BOREHOLE TYPE	POWER AUGER, HOLLOW STEM		
DATUM	GEODETIC	DATE	May 6, 2014
ORIGINATED BY	BT	COMPILED BY	WDF
CHECKED BY			



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
183.08	GROUND SURFACE								
0.00	TOPSOIL, silty clay, roots								
0.24	Brown CLAYEY SILT TILL, some sand to sandy, trace gravel Stiff to very stiff Brown to grey below about elev. 179.7m								
		1	SS	13					
		2	SS	17		4	22	48	26
		3	SS	22					
		4	SS	21					
		5	SS	19		2	15	46	37
178.05		6	SS	14					
5.03	END OF BOREHOLE								

Groundwater not established during drilling on May 6, 2014.

Piezometer dry to elev. 179.12m following installation on May 6, 2014.

Water level measured in Piezometer at elev. 182.65m on June 4, 2014.

Water level measured in Piezometer at elev. 182.44m on June 12, 2014.

Water level measured in Piezometer at elev. 182.71m on July 9, 2014.

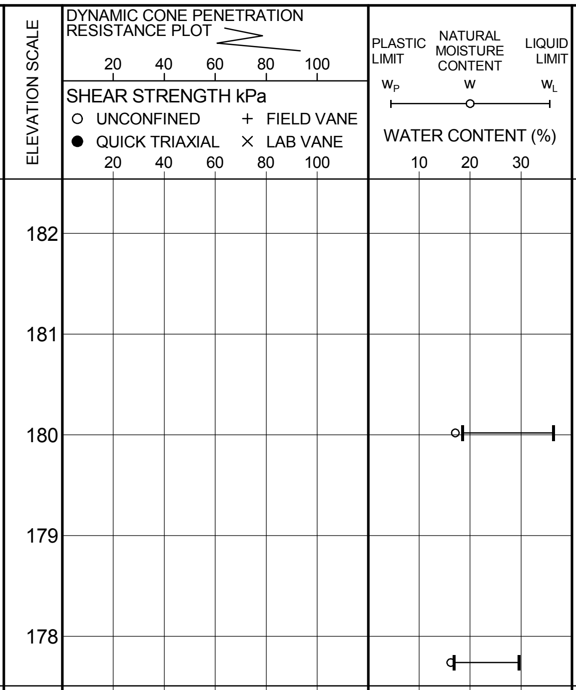
$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 625

1 OF 1 **METRIC**

PROJECT 13-1132-0111
W.P. 3093-09-00
LOCATION N 4693658.2 , E 338526.8
ORIGINATED BY BT
DIST ____ HWY 401
BOREHOLE TYPE POWER AUGER, HOLLOW STEM
COMPILED BY WDF
DATUM GEODETIC
DATE May 6, 2014
CHECKED BY ____

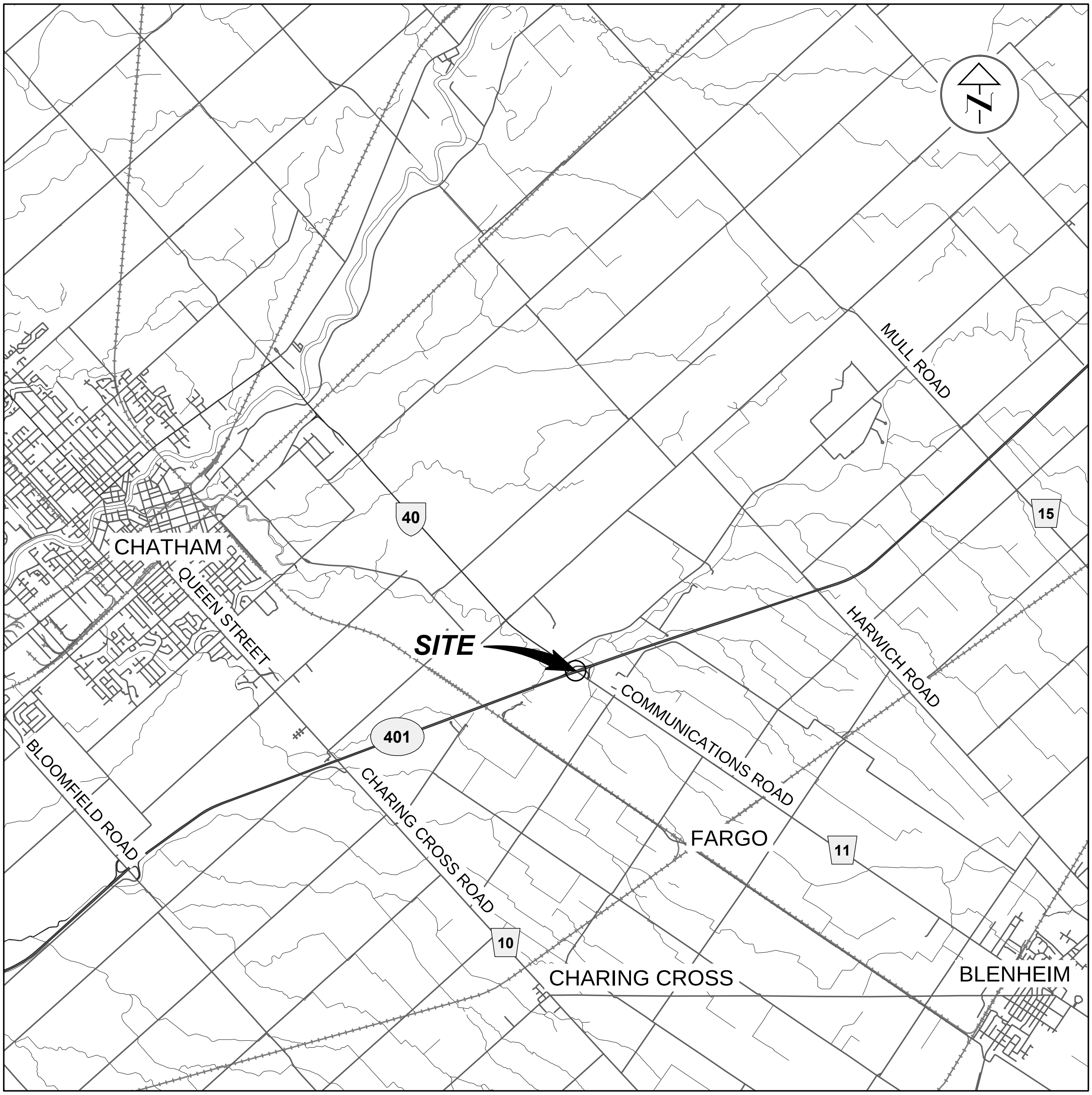
ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
182.54	GROUND SURFACE							
0.00	TOPSOIL, clayey silt Brown							
0.21	SILTY CLAY TILL, some sand, trace gravel Firm to very stiff Brown	1	SS	7				
		2	SS	12				
		3	SS	18	1	16	46	37
179.49 3.05	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey	4	SS	13				
		5	SS	18				
177.51		6	SS	11	2	17	45	36
5.03	END OF BOREHOLE Groundwater not established during drilling on May 6, 2014.							



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE



SCALE
0 2000 4000m
1:100,000

REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

NOTE

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

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

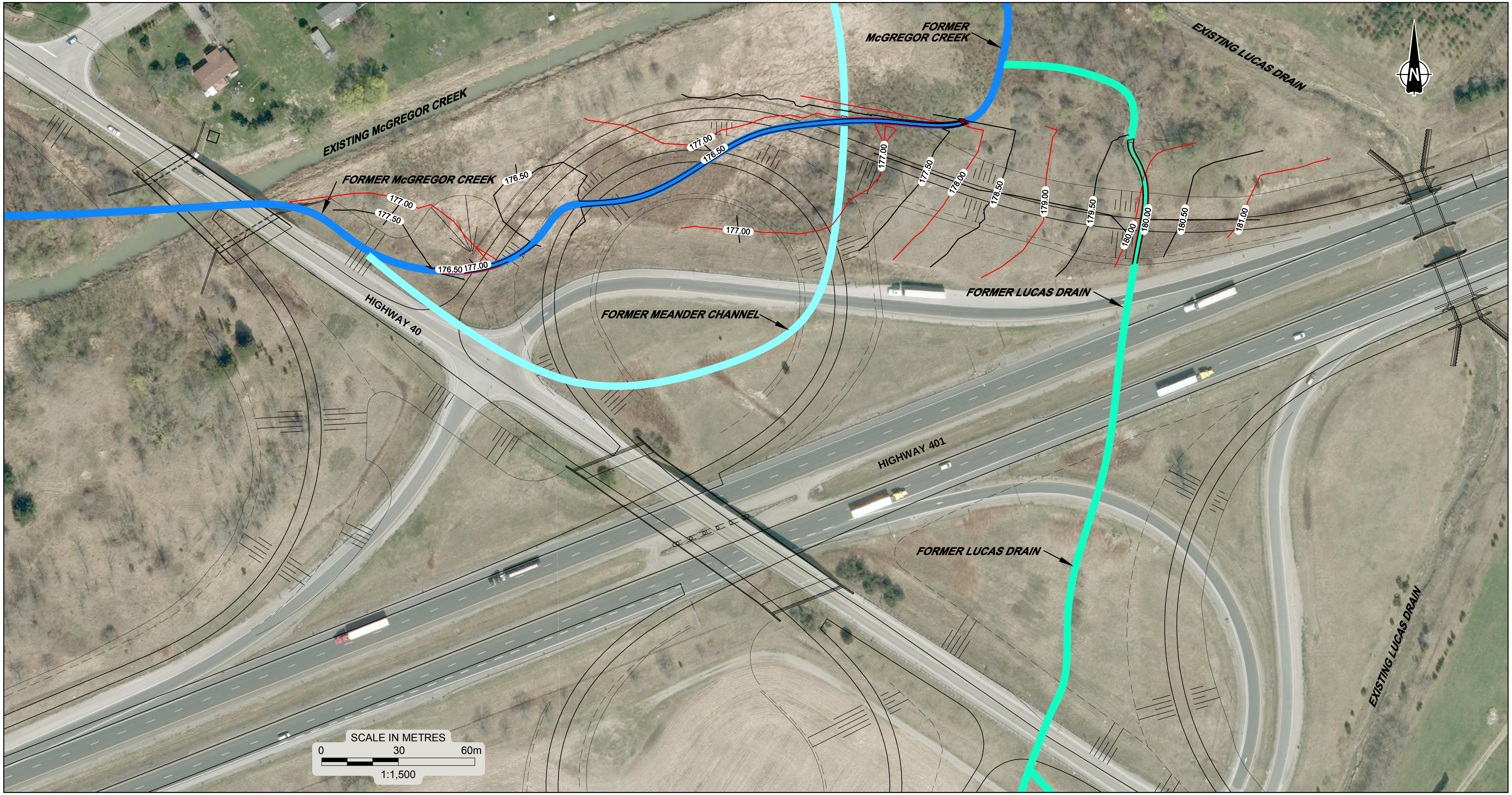
TITLE

KEY PLAN

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No. 1311320111-1000-F06001
			SCALE AS SHOWN REV. 0
CADD	LMK	July 17/14	**FIGURE 1**
CHECK			

Drawing file: 1311320111-1000-F06001.DWG Oct 14, 2014 - 1:56pm



REFERENCE

DRAWING BASED ON PLANS PROVIDED IN DIGITAL FORMAT BY DILLON CONSULTING LIMITED; AND 1955 AERIAL IMAGE No. 55-4217/55-202.

NOTES

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

ALL LOCATIONS ARE APPROXIMATE.

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

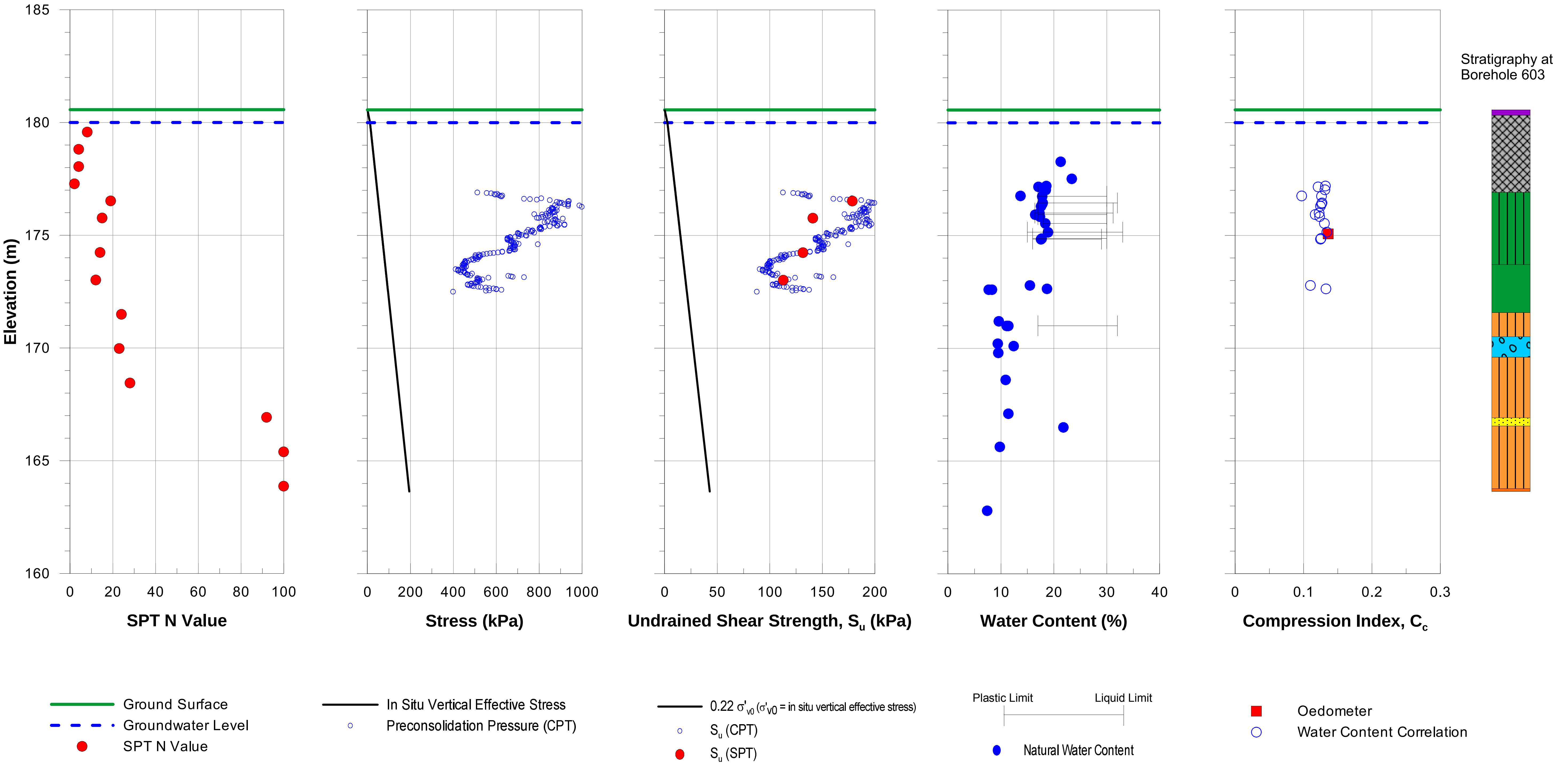
TITLE

FORMER LOCATION OF McGREGOR CREEK, LUCAS DRAIN, AND MEANDER CHANNEL

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.	1311320111-1000-F06002
			SCALE AS SHOWN	REV. 0
CADD	DCH\WDF	Feb. 13/15	FIGURE 2	
CHECK				

Drawing file: 1311320111-1000-F06002.DWG Feb 18, 2015 - 8:57am



🟪 TOPSOIL
▨ FILL
🟩 CLAYEY SILT TILL
🟩 CLAYEY SILT
🟧 SANDY SILT TILL
🟦 SAND AND GRAVEL
🟨 SILTY FINE SAND
🟧 SILT

NOTES
1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING TEXT.
2. OEDOMETER TEST DATA IS FROM BOREHOLE 604, SAMPLE 7.
3. SPT N VALUES OF 100 SHOWN ON THE ABOVE PLOT GENERALLY REPRESENT ACTUAL BLOW COUNTS OF GREATER THAN 100 OR PARTIAL BLOW COUNTS EXTRAPOLATED TO THE FULL PENETRATION DEPTH OF 0.3 m.

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

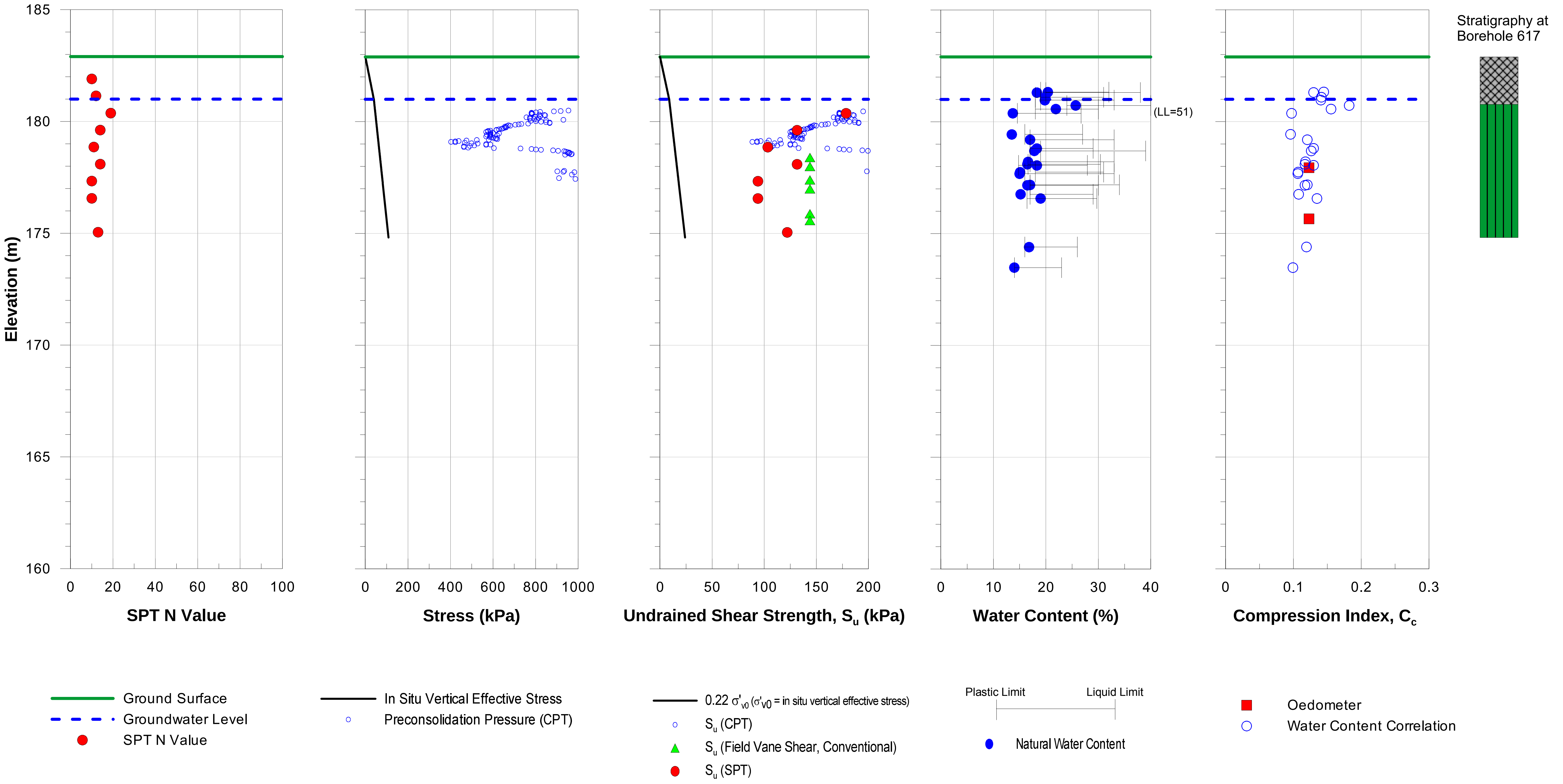
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE 603/CPT 603

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	**FILE No.**	1311320111-1000-F06003
			SCALE	AS SHOWN **REV.** 0
DRAWN	MK	Nov. 6/14		
CHECK				

FIGURE 3

N:\active\2013\1132-Geo\1132-0100\13-1132-0111 DILLON-GWP 3093-09-00-HWY 401-40\Drafting\GRAPHER FILES\1311320111-1000-F06003.grf



NOTES
1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING TEXT.
2. OEDOMETER TEST DATA IS FROM BOREHOLE 616, SAMPLE 6 (ELEVATION 177.95 m) AND BOREHOLE 104, SAMPLE 9 (ELEVATION 175.66 m).
3. SPT N VALUES OF 100 SHOWN ON THE ABOVE PLOT GENERALLY REPRESENT ACTUAL BLOW COUNTS OF GREATER THAN 100 OR PARTIAL BLOWCOUNTS EXTRAPOLATED TO THE FULL PENETRATION DEPTH OF 0.3 m.
4. ALL OF THE THE UNDRAINED SHEAR STRENGTHS MEASURED BY THE FIELD VANE SHEAR TESTS WERE IN EXCESS OF 144 kPa WHICH IS THE LIMIT OF THE MEASURING DEVICE USED.

▨ FILL
▥ CLAYEY SILT TILL

PROJECT
HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE
SUMMARY OF SUBSURFACE TEST DATA
BOREHOLE 617/CPT 617

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.	1311320111-1000-F06004
			SCALE AS SHOWN	REV. 0
DRAWN	MK	Nov. 7/14	FIGURE 4	
CHECK				

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MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

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

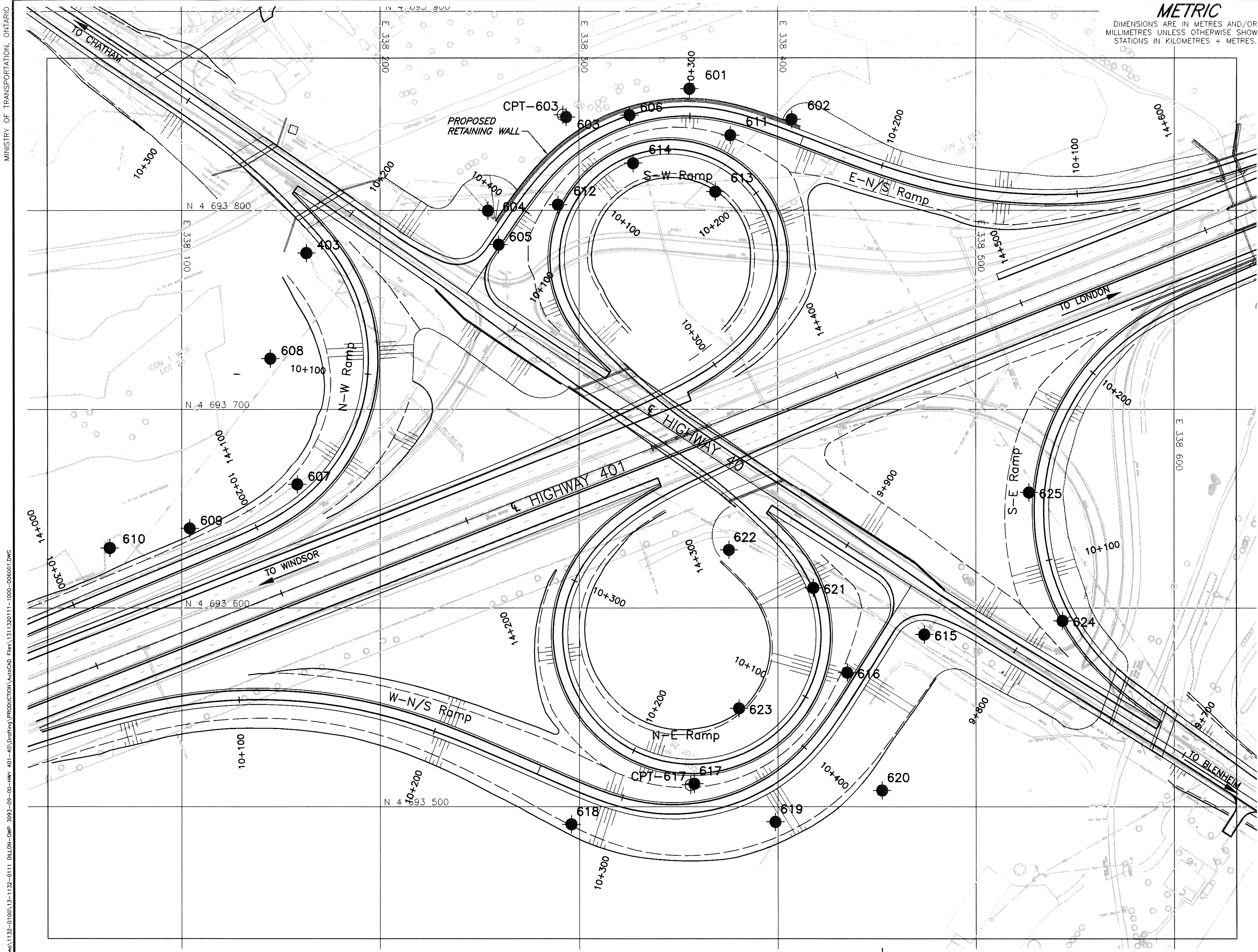
CONT No.
WP No. 3093-09-00

SHEET

HIGH FILL RAMP EMBANKMENTS

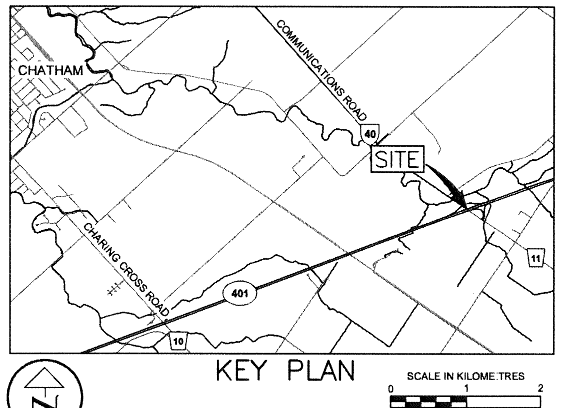
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
BOREHOLE LOCATIONS

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



PLAN

SCALE
20 0 20 m



KEY PLAN

SCALE IN KILOMETRES
0 1 2

LEGEND

● Borehole – Current Investigation

⊕ Piezo-Cone Penetration Test – Current Investigation

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
605	181.54	4 693 782.8	338 259.8
606	180.49	4 693 847.9	338 325.5
607	183.52	4 693 662.4	338 158.4
608	183.31	4 693 725.6	338 144.9
609	183.54	4 693 640.2	338 104.0
610	183.50	4 693 630.3	338 064.0
611	180.34	4 693 837.8	338 376.2
612	180.64	4 693 802.9	338 289.5
613	180.73	4 693 809.4	338 368.6
614	180.43	4 693 823.8	338 327.3
615	181.98	4 693 586.7	338 473.9
616	182.85	4 693 567.5	338 434.9
617	182.90	4 693 511.6	338 357.8
CPT-617	182.90	4 693 511.0	338 356.0
618	183.31	4 693 491.2	338 296.0
619	182.25	4 693 492.4	338 398.7
620	182.48	4 693 508.2	338 452.5
621	183.08	4 693 610.2	338 417.8
622	183.06	4 693 629.3	338 375.4
623	182.72	4 693 549.6	338 380.4
624	183.08	4 693 593.6	338 543.6
625	182.54	4 693 658.2	338 526.8
(Geocres No. 40J8-62)			
601	180.44	4 693 861.1	338 355.6
602	180.40	4 693 845.8	338 407.1
603	180.57	4 693 846.9	338 293.6
CPT-603	180.57	4 693 848.0	338 292.0
604	180.71	4 693 799.9	338 254.4
(Geocres No. 40J8-63)			
403	180.85	4 693 778.7	338 163.1

REFERENCE

Base plans provided in digital format by Dillon Consulting Limited.

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.

D. U. PROUT
Jul 29/15

F.J. HEFFERNAN
Jul 29/15

NO.	DATE	BY	REVISION
1	Feb.13/15	WDF	RAMP LOCATIONS REVISED

Geocres No. 40J8-61

HWY. 401		PROJECT NO. 13-1132-0111	DIST.
SUBM'D. NG	CHKD. DUP	DATE: Oct. 10/14	SITE:
DRAWN: LMK	CHKD. SJB	APPD. FJH	DWG. 1

PLOT DATE: July 29, 2015
FILENAME: N:\active\2013\1132-Geo\1132-0100\13-1132-0111 DILLON-GWP 3093-09-00-HWY 401-40\Drafting\PRODUCTION\AutoCAD Files\1311320111-1000-D06001.DWG

CONT No.
WP No. 3093-09-00

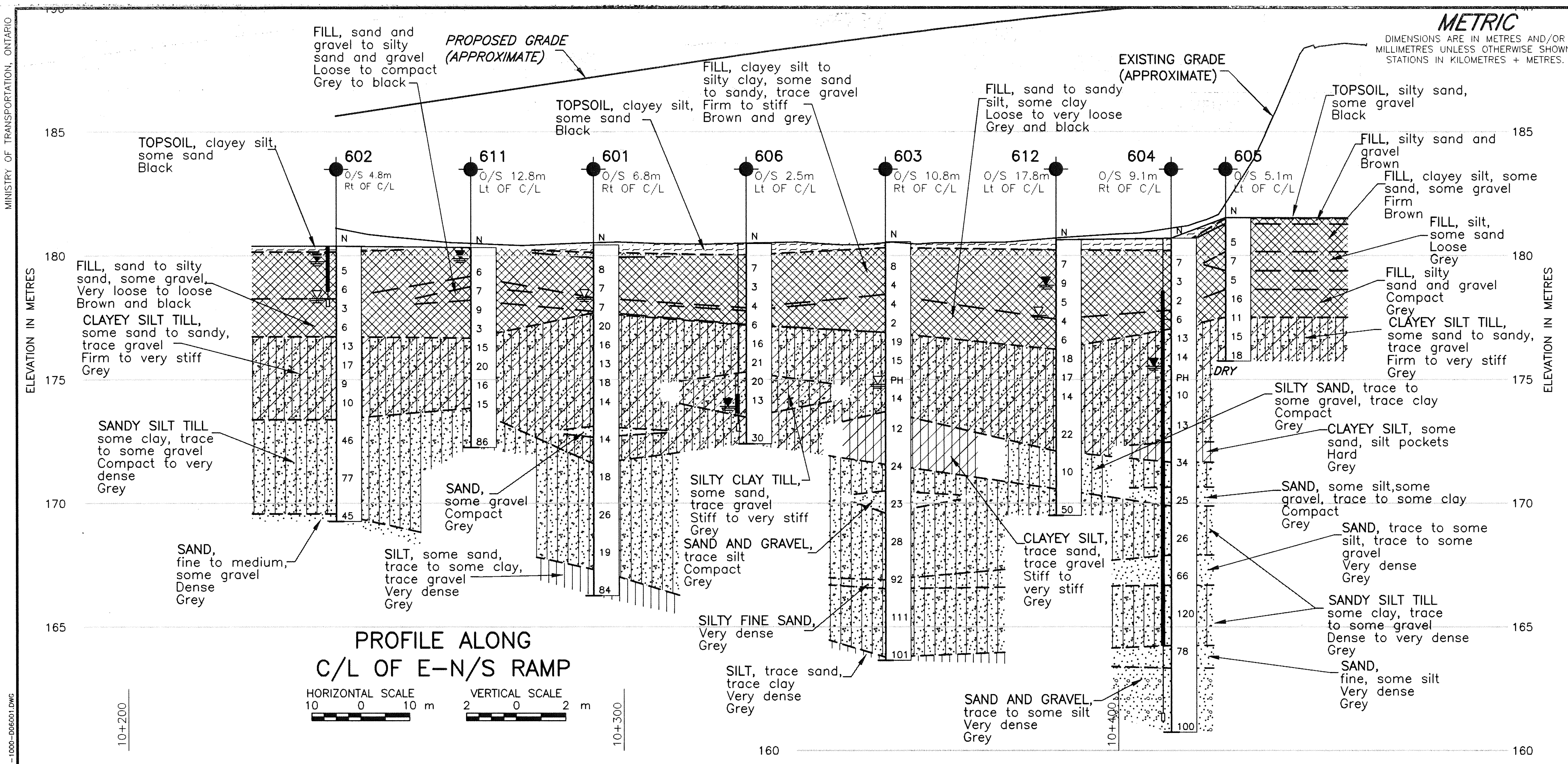
SHEET

HIGH FILL RAMP EMBANKMENTS

HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
BOREHOLE LOCATIONS

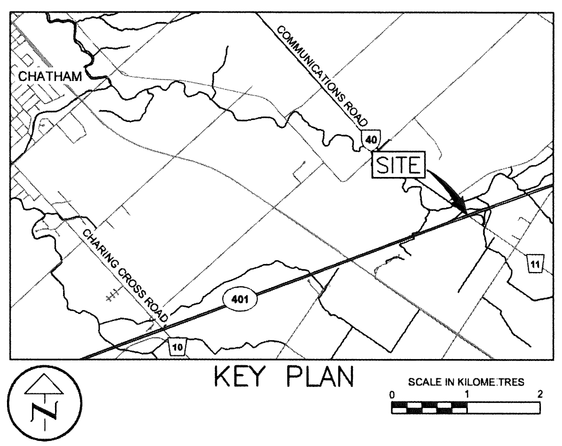
Golder Associates Ltd.
LONDON, ONTARIO, CANADA

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



PROFILE ALONG C/L OF E-N/S RAMP

HORIZONTAL SCALE 10 0 10 m
VERTICAL SCALE 2 0 2 m



KEY PLAN

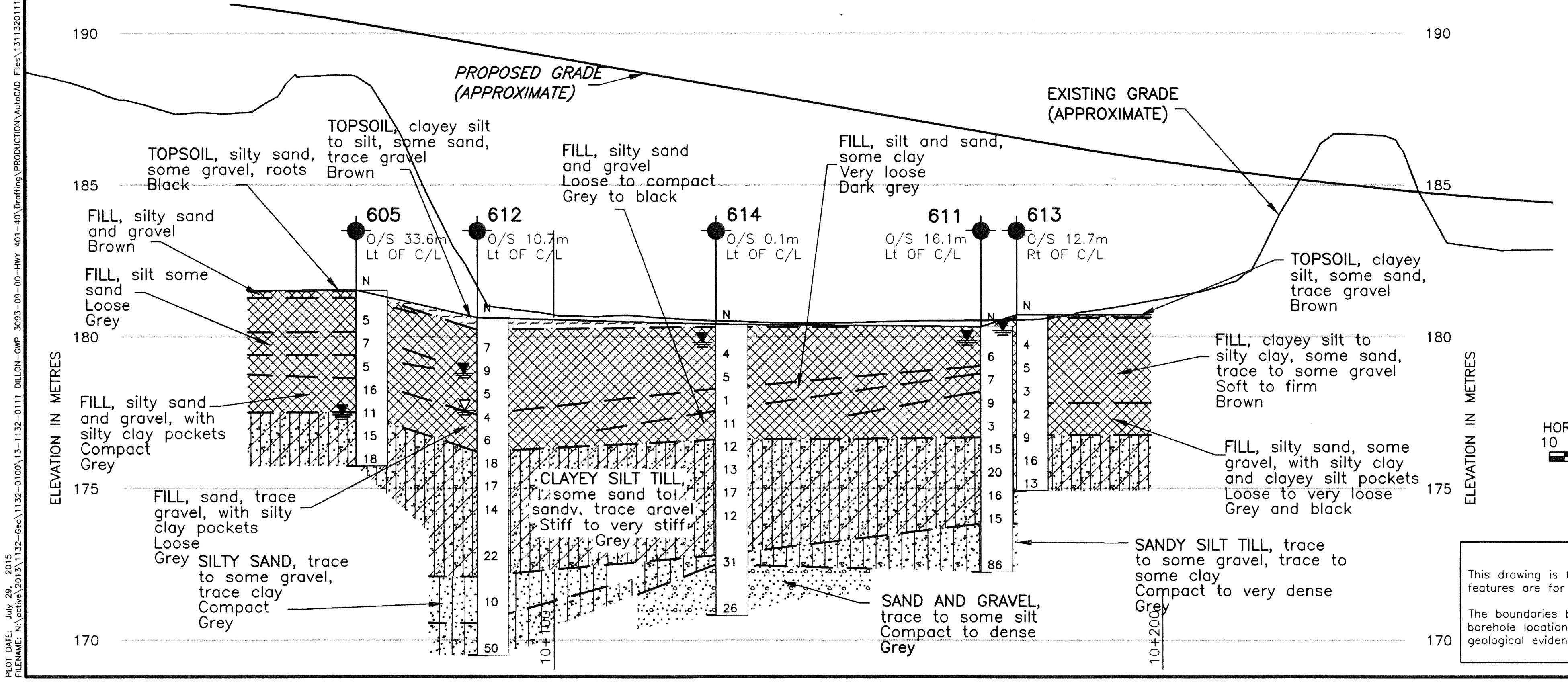
LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Standpipe
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL encountered during drilling
▼	WL in piezometer or open borehole, June & July, 2014.
DRY	WL not established during drilling.

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10) NORTHING	EASTING
605	181.54	4 693 782.8	338 259.8
606	180.49	4 693 847.9	338 325.5
611	180.34	4 693 837.8	338 376.2
612	180.64	4 693 802.9	338 289.5
613	180.73	4 693 809.4	338 368.6
614	180.43	4 693 823.8	338 327.3
(Geocres No. 40J8-62)			
601	180.44	4 693 861.1	338 355.6
602	180.40	4 693 845.8	338 407.1
603	180.57	4 693 846.9	338 293.6
604	180.71	4 693 799.9	338 254.4

REFERENCE

Base plans provided in digital format by Dillon Consulting Limited.



D.J. PROUT Jul 29/15

F.J. HEFFERNAN Jul 29/15

PROFILE ALONG C/L OF S-W RAMP

HORIZONTAL SCALE 10 0 10 m
VERTICAL SCALE 2 0 2 m

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.

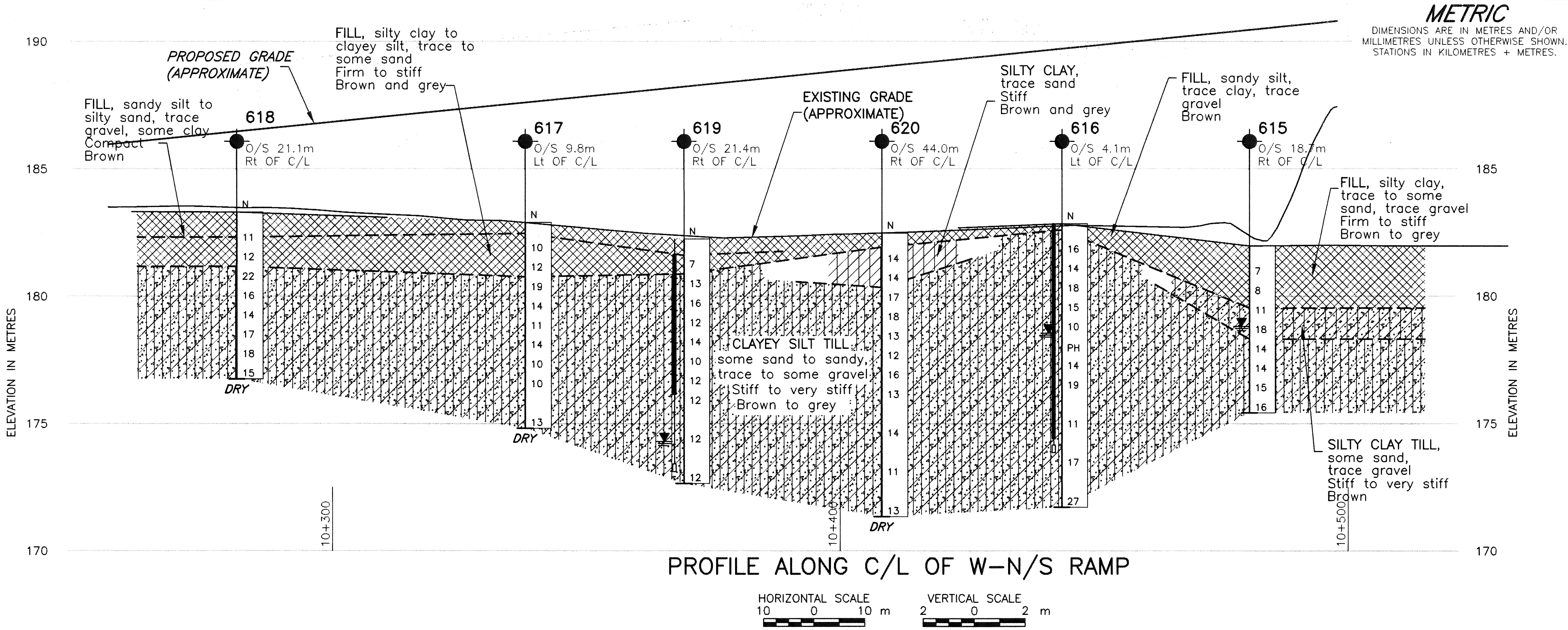
NO.	DATE	BY	REVISION
1	Feb.13/15	WDF	RAMP LOCATIONS REVISED

Geocres No. 40J8-61			
HWY. 401	PROJECT NO. 13-1132-0111		DIST.
SUBM'D. NG	CHKD. DUP	DATE: Oct. 10/14	SITE:
DRAWN: LMK	CHKD. SJB	APPD. FJH	DWG. 2

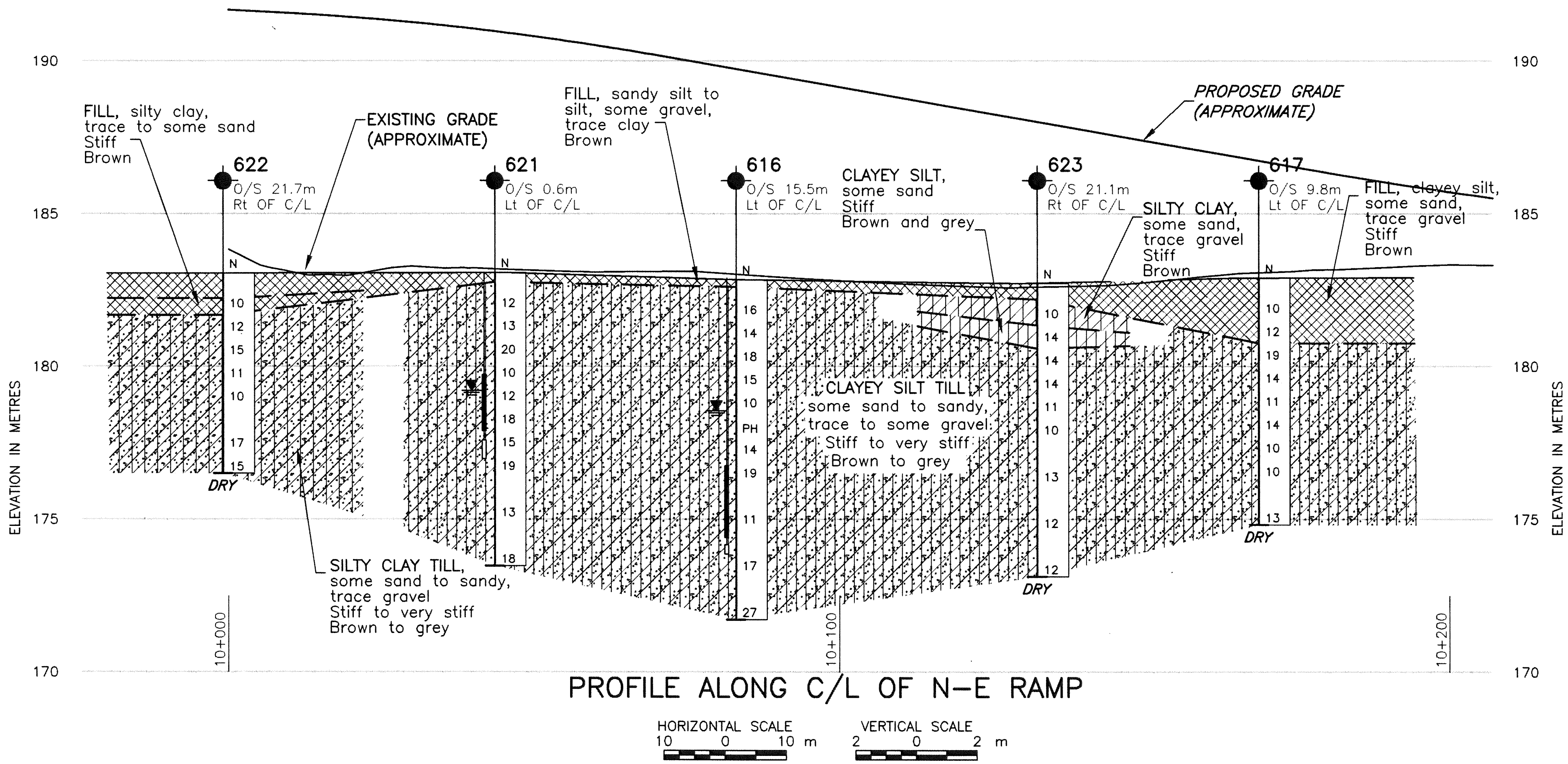
PLOT DATE: July 29, 2015
FILENAME: N:\Active\2013\1132-Geo\1132-0100\13-1132-0111 DILLON-OWP 3093-09-00-HWY 401-40\Drafting\PRODUCTION\AutoCAD Files\1311320111-1000-D6001.DWG

MINISTRY OF TRANSPORTATION, ONTARIO

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



PROFILE ALONG C/L OF W–N/S RAMP



PROFILE ALONG C/L OF N–E RAMP

D.J. PROUT Jul 29/15

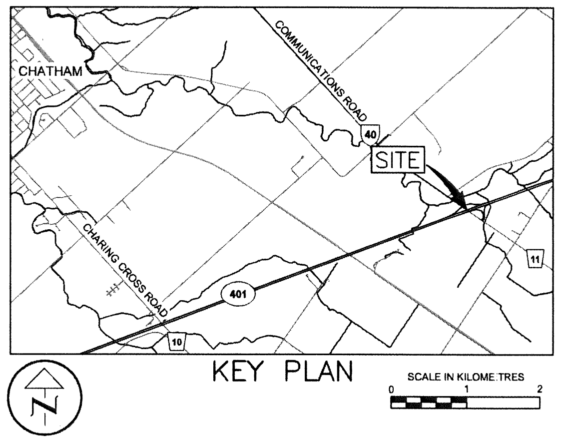
F.J. HEFFERNAN Jul 29/15

CONT No.
WP No. 3093–09–00

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
BOREHOLE LOCATIONS

SHEET

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Standpipe
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer, on May 13 & July 9, 2014.
DRY	WL not established during drilling.

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10) NORTHING	EASTING
615	181.98	4 693 586.7	338 473.9
616	182.85	4 693 567.5	338 434.9
617	182.90	4 693 511.6	338 357.8
618	183.31	4 693 491.2	338 296.0
619	182.25	4 693 492.4	338 398.7
620	182.48	4 693 508.2	338 452.5
621	183.08	4 693 610.2	338 417.8
622	183.06	4 693 629.3	338 375.4
623	182.72	4 693 549.6	338 380.4

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 provided in digital format by Dillon Consulting Limited.

NO.	DATE	BY	REVISION
1	Feb.13/15	WDF	RAMP LOCATIONS REVISED

Geocres No. 40J8–61

HWY. 401		PROJECT NO. 13–1132–0111	DIST.
SUBM'D. NG	CHKD. DUP	DATE: Oct. 10/14	SITE:
DRAWN: LMK	CHKD. SJB	APPD. FJH	DWG. 3

PLOT DATE: July 29, 2015
FILENAME: N:\active\2013\1132-Geo\1132-0100\13-1132-0111 DILLON-GWP 3093-09-00-HWY 401-40\Drafting\PRODUCTION\AutoCAD Files\1311320111-1000-D06001.DWG

MINISTRY OF TRANSPORTATION, ONTARIO

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

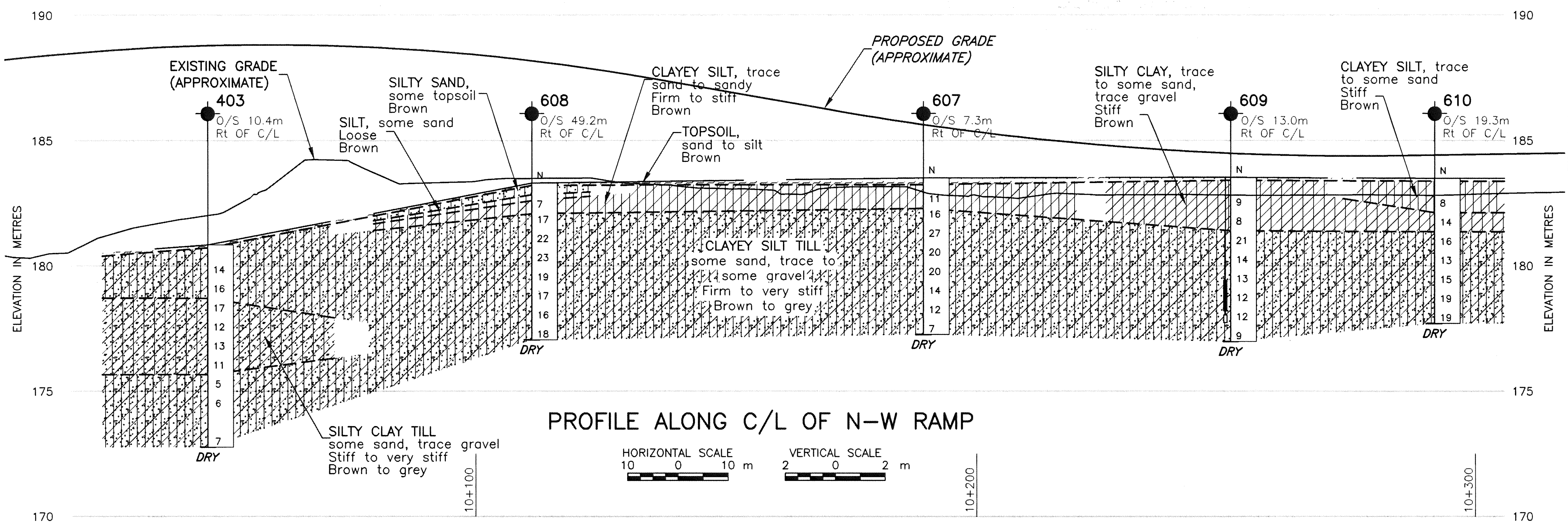
CONT No.
WP No. 3093-09-00

SHEET

HIGH FILL RAMP EMBANKMENTS

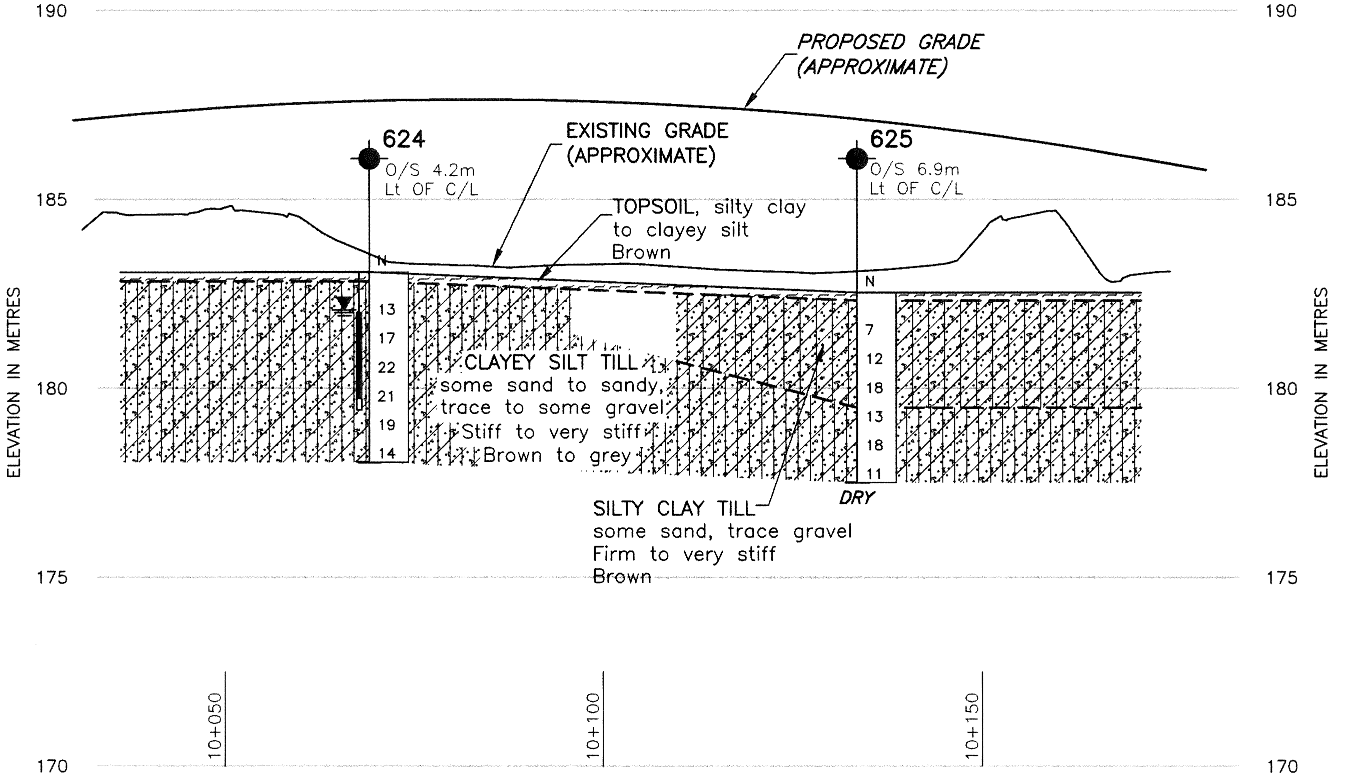
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA



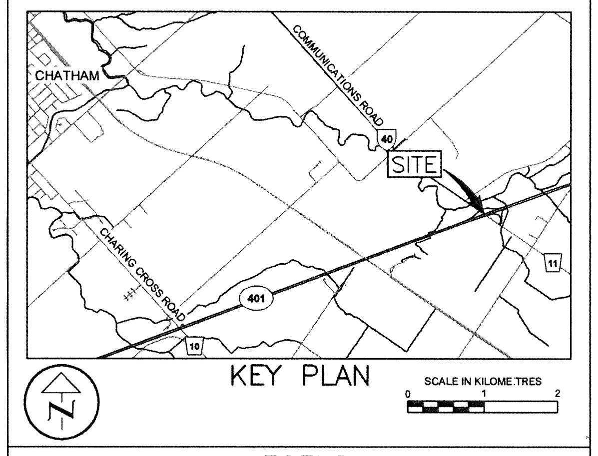
PROFILE ALONG C/L OF N-W RAMP

HORIZONTAL SCALE 10 0 10 m
VERTICAL SCALE 2 0 2 m



PROFILE ALONG C/L OF S-E RAMP

HORIZONTAL SCALE 10 0 10 m
VERTICAL SCALE 2 0 2 m



KEY PLAN

LEGEND

●	Borehole – Current Investigation
▮	Seal
▯	Standpipe
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL in piezometer, on July 9, 2014.
DRY	WL not established during drilling.

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
607	183.52	4 693 662.4	338 158.4
608	183.31	4 693 725.6	338 144.9
609	183.54	4 693 640.2	338 104.0
610	183.50	4 693 630.3	338 064.0
624	183.08	4 693 593.6	338 543.6
625	182.54	4 693 658.2	338 526.8
(Geocres No. 40J8-63)			
403	180.85	4 693 778.7	338 163.1

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 provided in digital format by Dillon Consulting Limited.

D.U. PROUT
Jul 29/15

F.J. HEFFERNAN
Jul 29/15

NO.	DATE	BY	REVISION
1	Feb.13/15	WDF	RAMP LOCATIONS REVISED

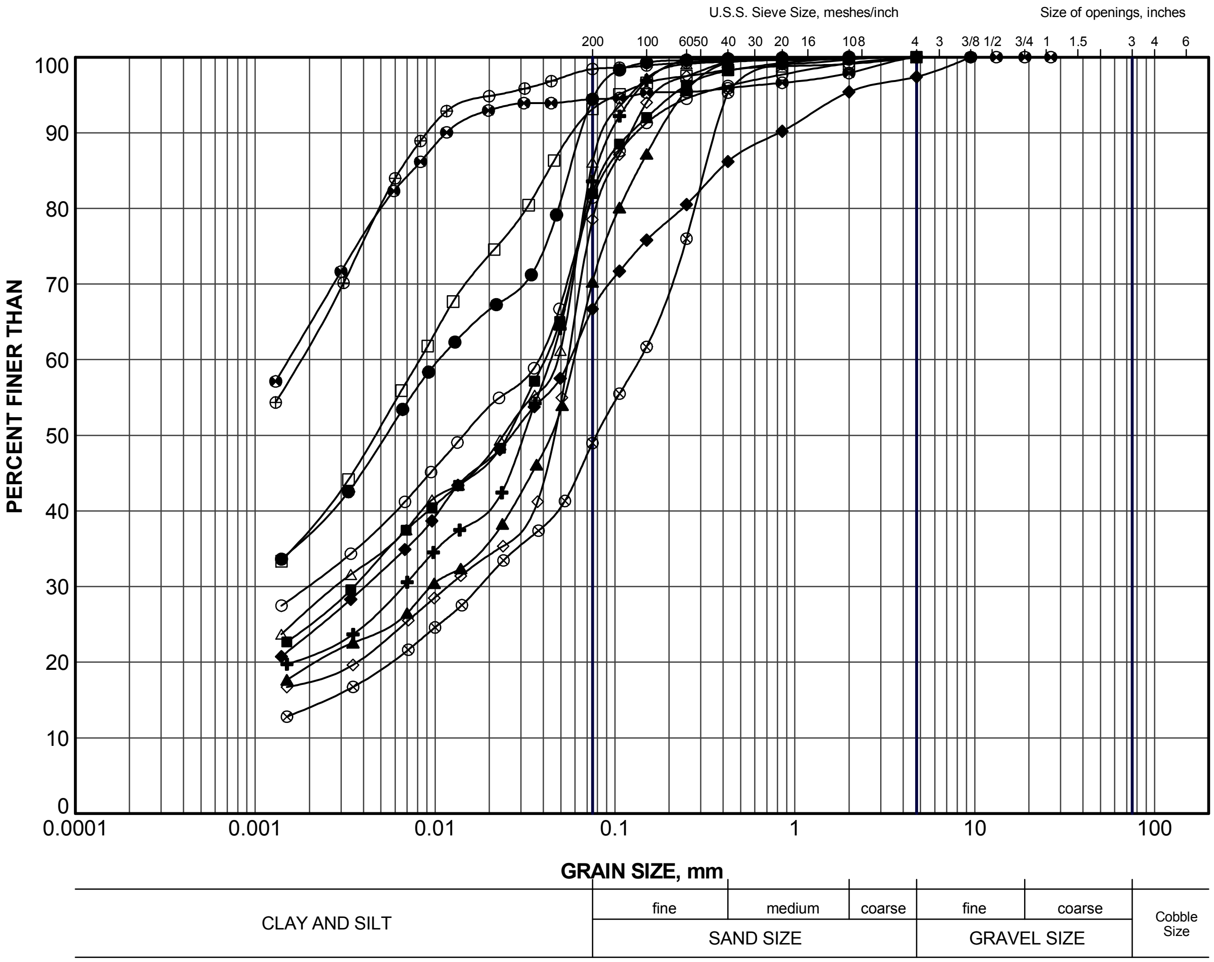
Geocres No. 40J8-61

HWY. 401		PROJECT NO. 13-1132-0111	DIST.
SUBM'D. NG	CHKD. DUP	DATE: Oct. 10/14	SITE:
DRAWN: LMK	CHKD. SJB	APPD. FJH	DWG. 4

PLOT DATE: July 29, 2015
FILENAME: N:\active\2013\1132-Geo\1132-0100\13-1132-0111 DILLON-GWP 3093-09-00-HWY 401-40\Drafting\PRODUCTION\AutoCAD Files\1311320111-1000-D06001.DWG

APPENDIX A

Laboratory Test Data – Routine Soils



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	2	178.7
■	602	2	178.7
▲	603	3	178.1
✚	604	2	179.0
◆	605	3	179.0
◇	606	2	178.7
○	611	2	178.6
△	613	1	179.7
⊗	614	3	177.9
⊕	618	2	181.6
□	619	1	181.3
⨂	622	1	182.1

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

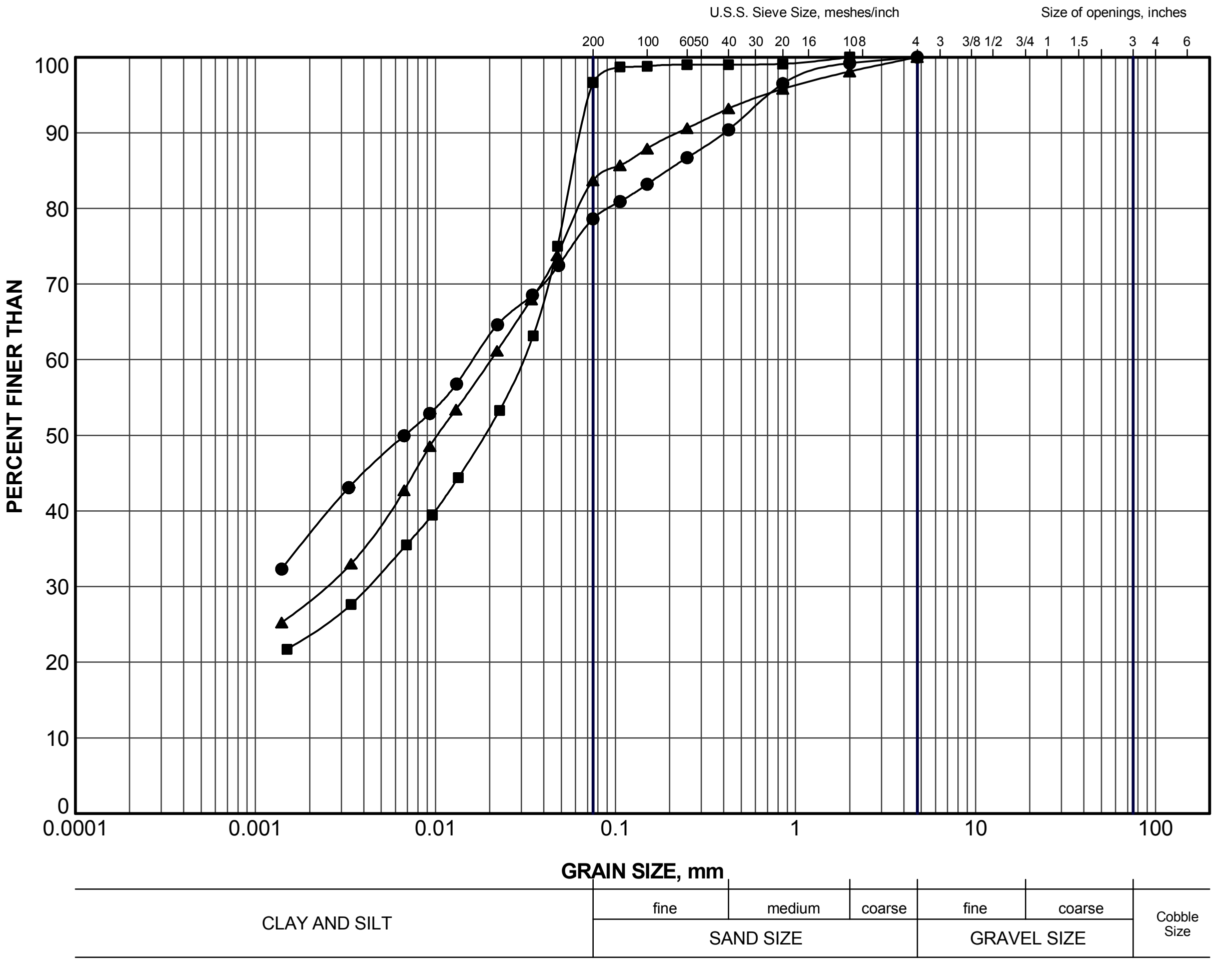
GRAIN SIZE DISTRIBUTION

FILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	13-1132-0111	FILE No.1311320111-1000-R060A1	
		SCALE N/A	REV.
DRAWN WDF	Oct 14/14	**FIGURE A-1**	
CHECK			

LDN_MTO_GSD-15 GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	607	1	182.7
■	610	1	182.5
▲	623	2	181.0

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

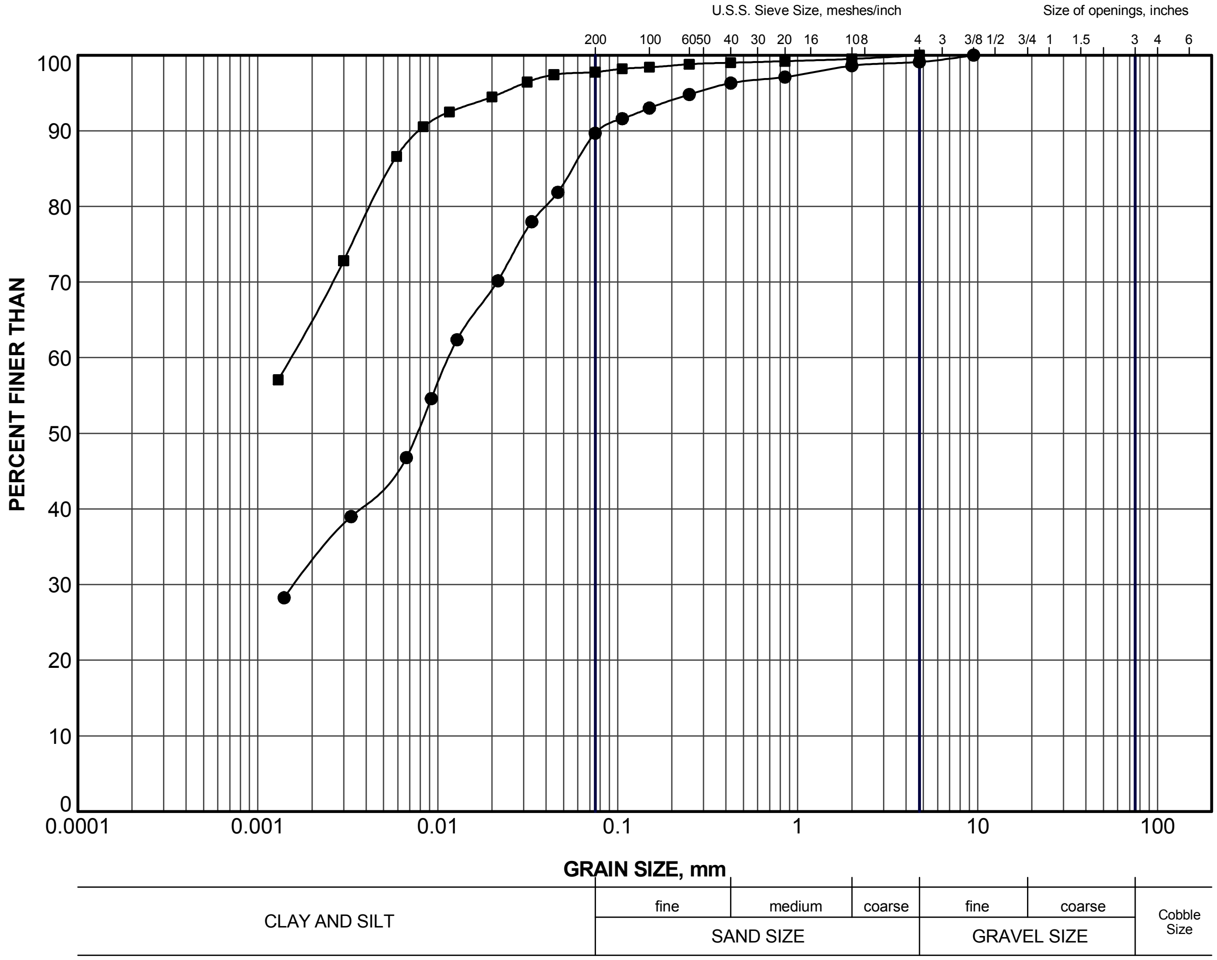
GRAIN SIZE DISTRIBUTION

CLAYEY SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R060A2
			SCALE N/A / REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-2**
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	609	2	181.8
■	620	2	180.7

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

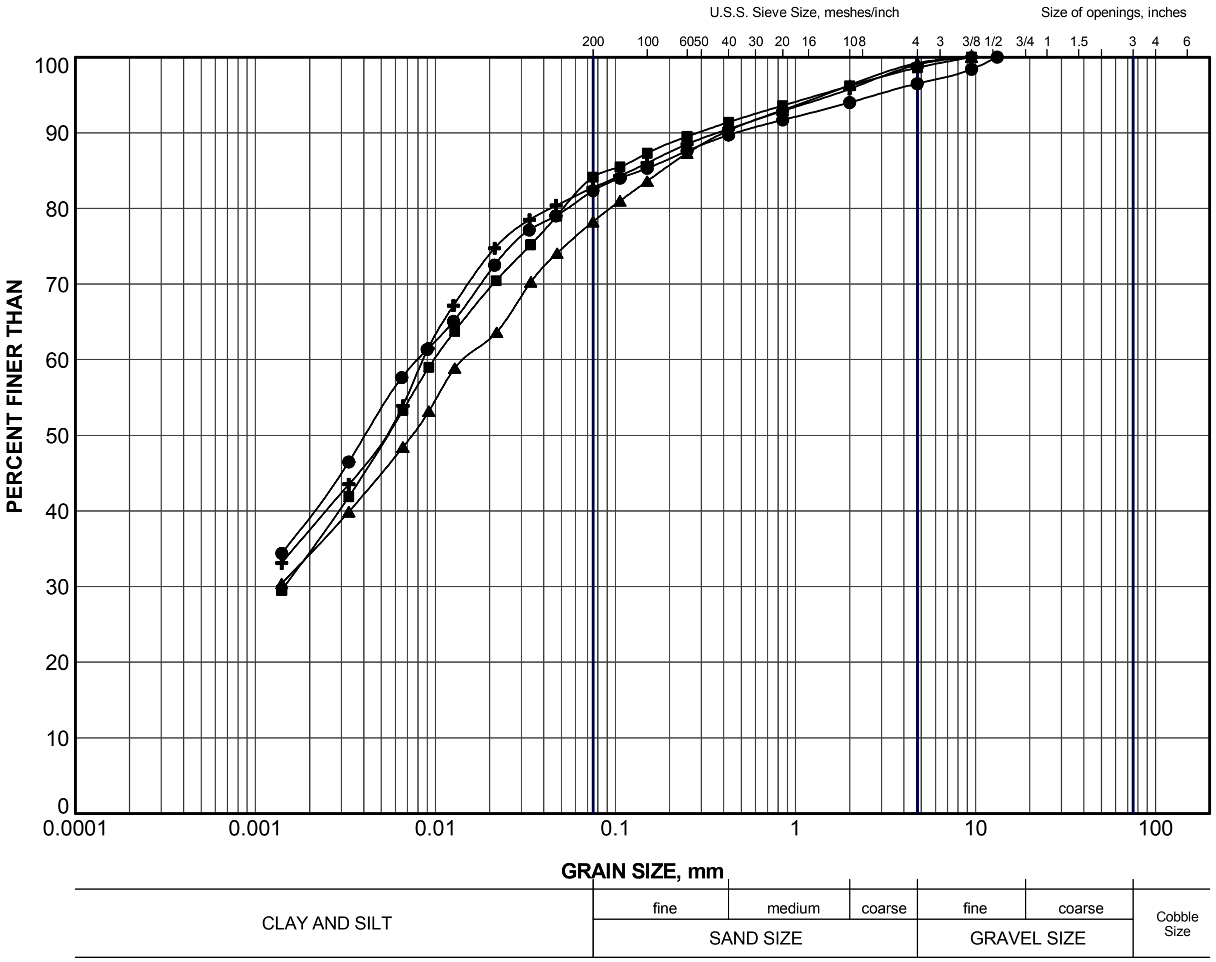
GRAIN SIZE DISTRIBUTION

SILTY CLAY

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R060A3
			SCALE N/A REV.
DRAWN	WDF	Oct 14/14	FIGURE A-3
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	403	3	178.3
■	615	4	178.7
▲	622	2	181.3
✚	625	3	180.0

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

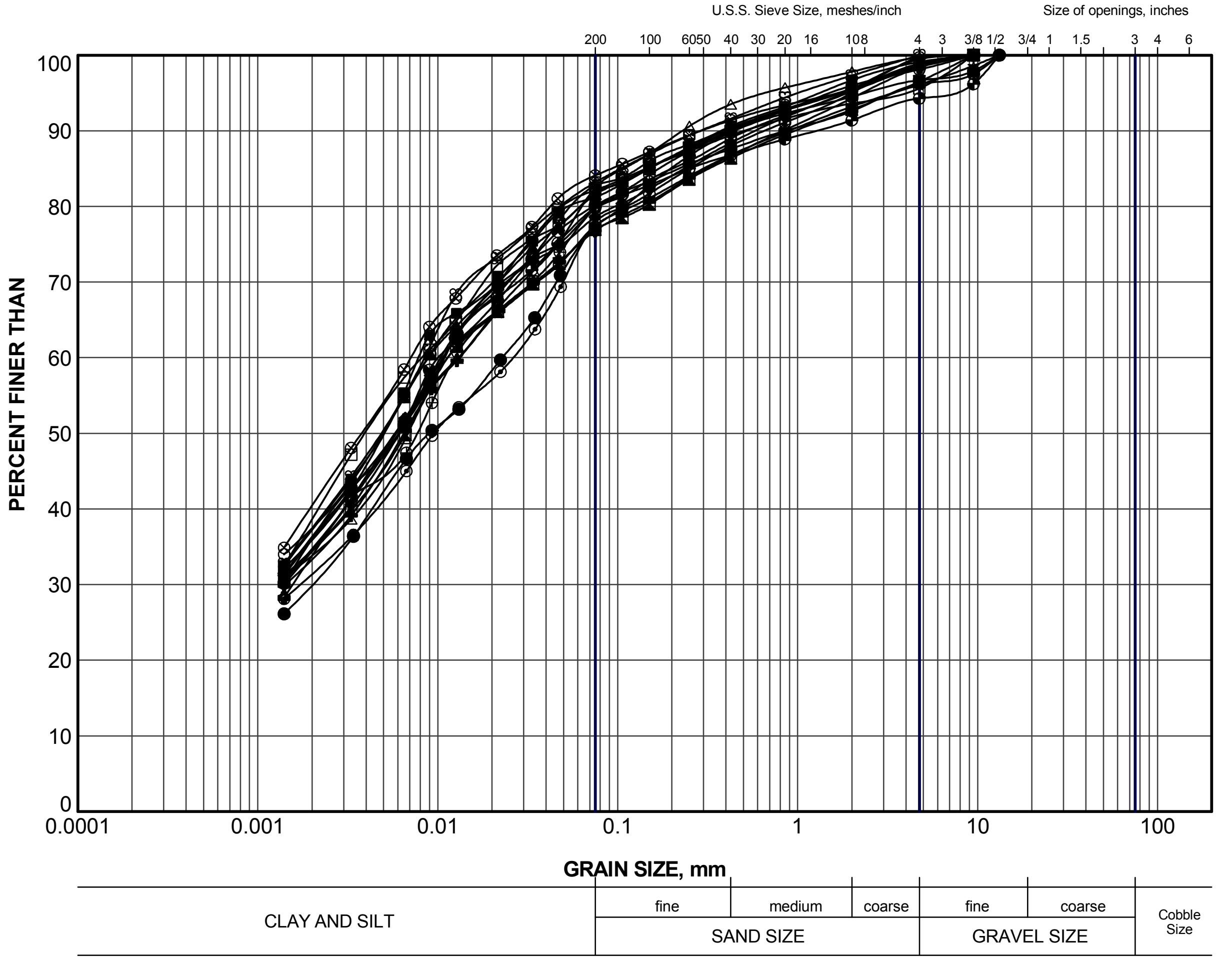
GRAIN SIZE DISTRIBUTION

SILTY CLAY TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.	13-1132-0111	FILE No. 1311320111-1000-R060A4	
		SCALE N/A	REV.
DRAWN WDF	Oct 14/14	**FIGURE A-4**	
CHECK			

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	403	1	179.9
■	403	8	174.5
▲	601	5	176.4
✚	602	7	174.8
◆	603	6	175.8
◇	605	6	176.7
○	606	5	176.5
△	606	9	172.6
⊗	607	6	179.0
⊕	608	4	180.3
□	608	7	178.1
⨂	609	5	179.5
◕	610	6	178.7
☆	611	6	175.5
⌘	612	9	172.8
⧗	613	6	175.9
⊙	614	7	174.9
✜	615	6	177.2

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

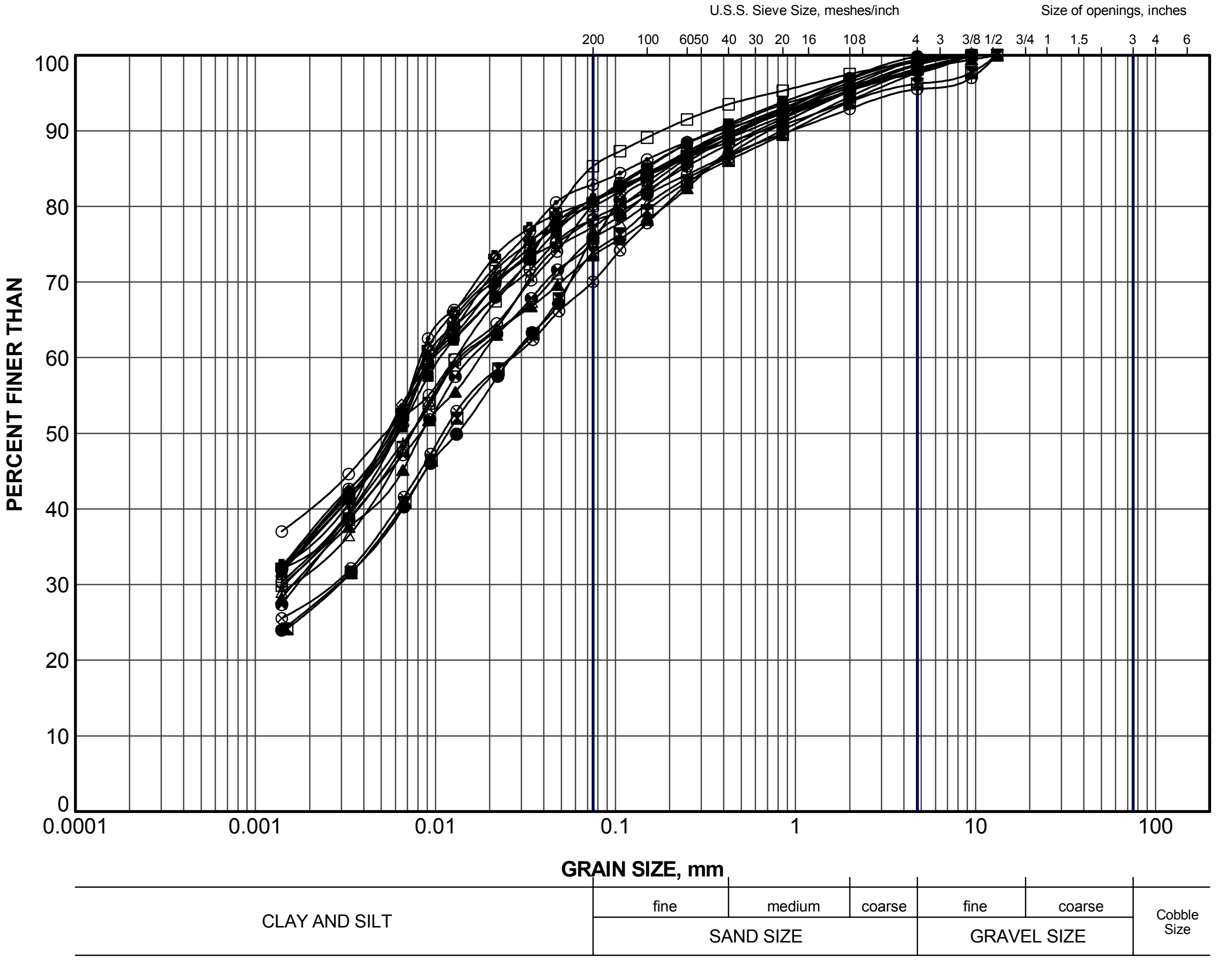
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

PROJECT No.	13-1132-0111	FILE No.1311320111-1000-R060A5	
		SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	FIGURE A-5
CHECK			

Golder Associates
LONDON, ONTARIO

LDN_MTO_GSD-20 GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	616	2	181.1
■	616	5	178.8
▲	616	10	173.5
✚	617	6	178.1
◆	617	8	176.6
◇	618	7	177.8
○	619	5	178.2
△	619	9	174.4
⊗	620	4	179.2
⊕	620	6	177.7
□	621	2	181.3
◍	621	3	180.6
◕	621	8	176.8
☆	623	4	179.4
⌘	623	7	176.4
⧗	624	2	181.3
⊙	624	5	179.0
✤	625	6	177.7

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

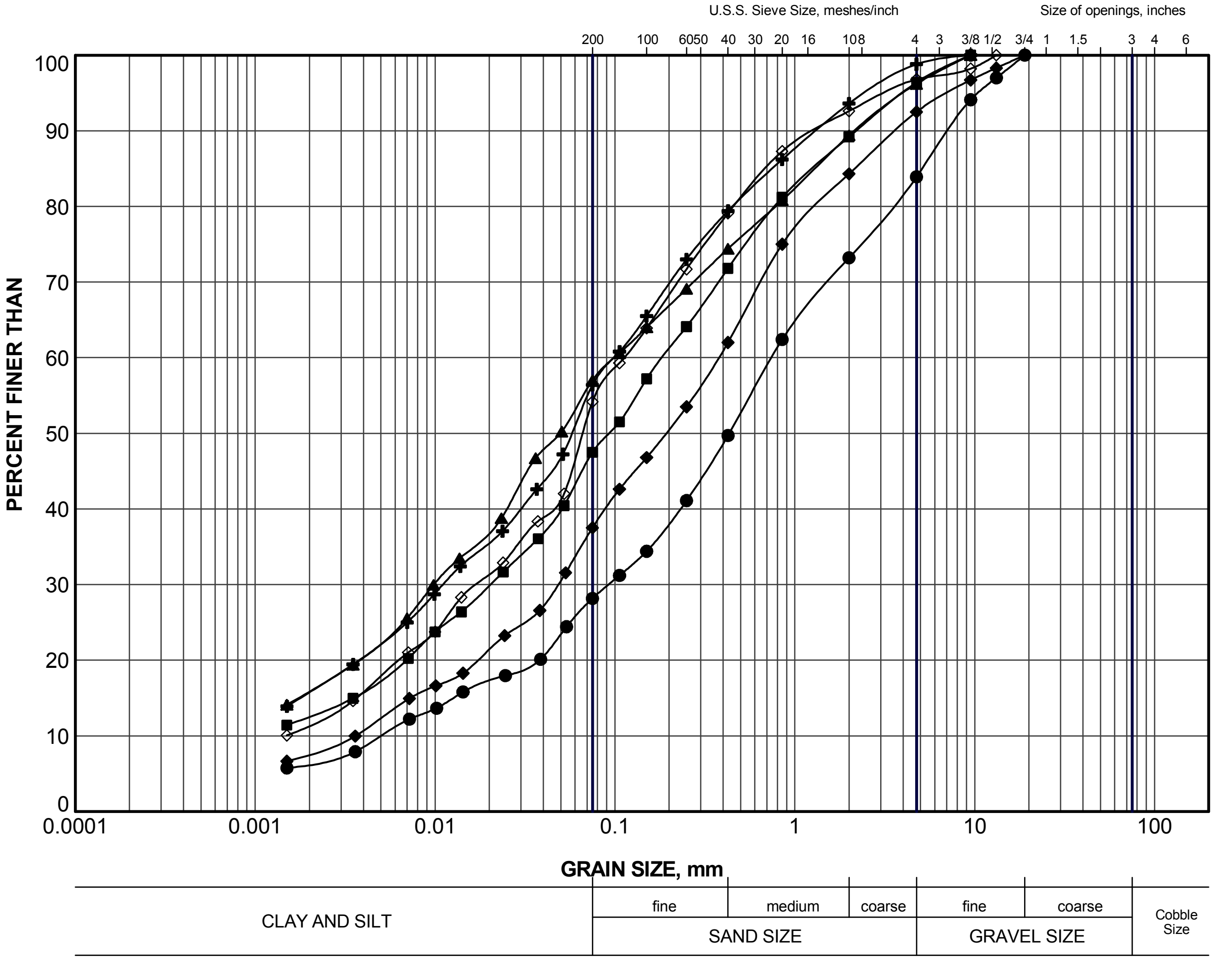
GRAIN SIZE DISTRIBUTION

CLAYEY SILT TILL

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R060A6	
			SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-6**	
CHECK				

LDN_MTO_GSD-20 GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	10	171.1
■	602	9	172.6
▲	603	14	165.4
✚	604	12	168.6
◆	611	9	172.5
◇	612	11	169.7

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

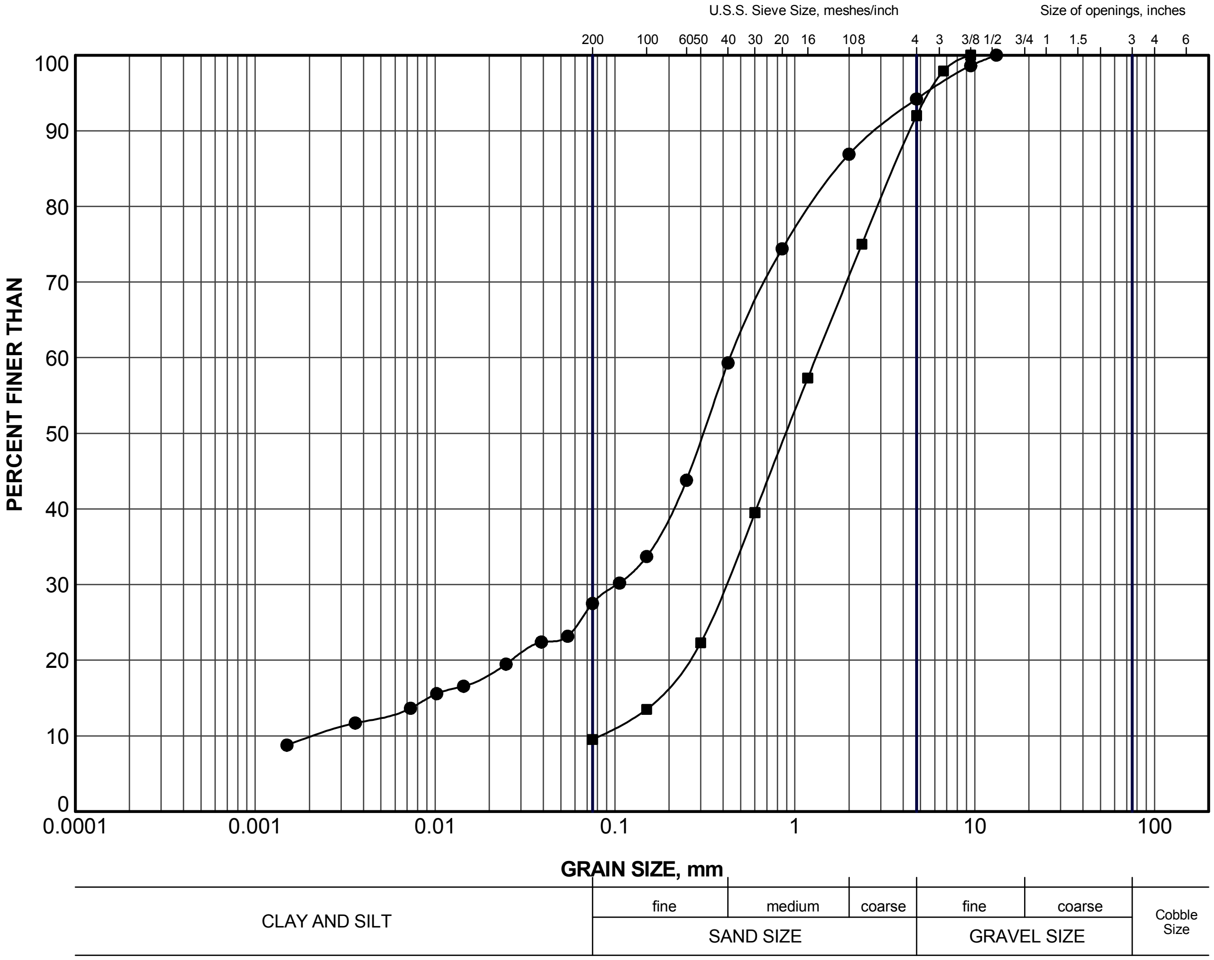
GRAIN SIZE DISTRIBUTION

SANDY SILT TILL

PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R060A7	
			SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-7**	
CHECK				

Golder Associates
LONDON, ONTARIO

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	604	11	170.1
■	604	13	167.1

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

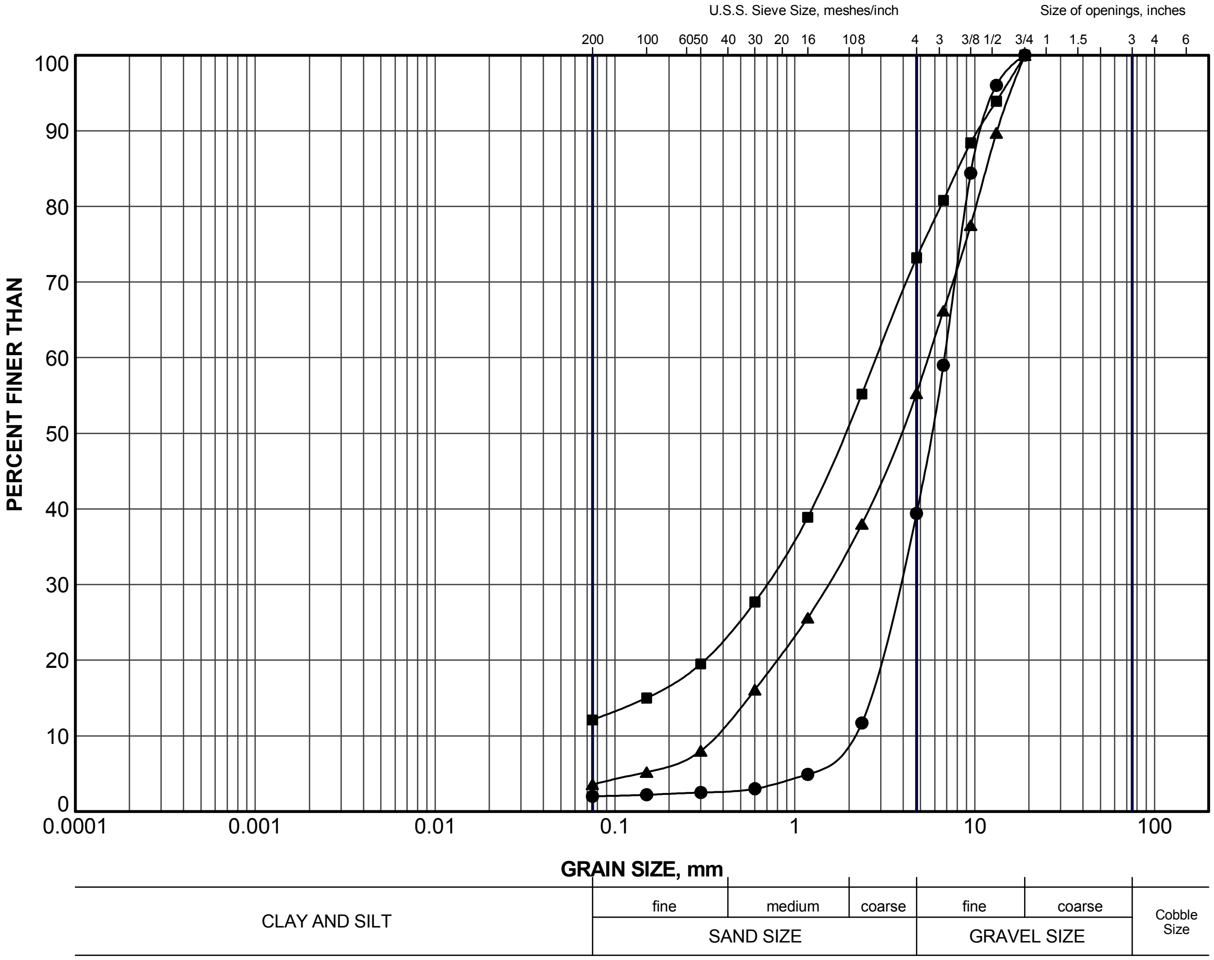
GRAIN SIZE DISTRIBUTION

SAND

PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R060A8
			SCALE N/A — REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-8**
CHECK			

Golder Associates
LONDON, ONTARIO

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	603	11	170.0
■	604	17	161.0
▲	614	10	171.1

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

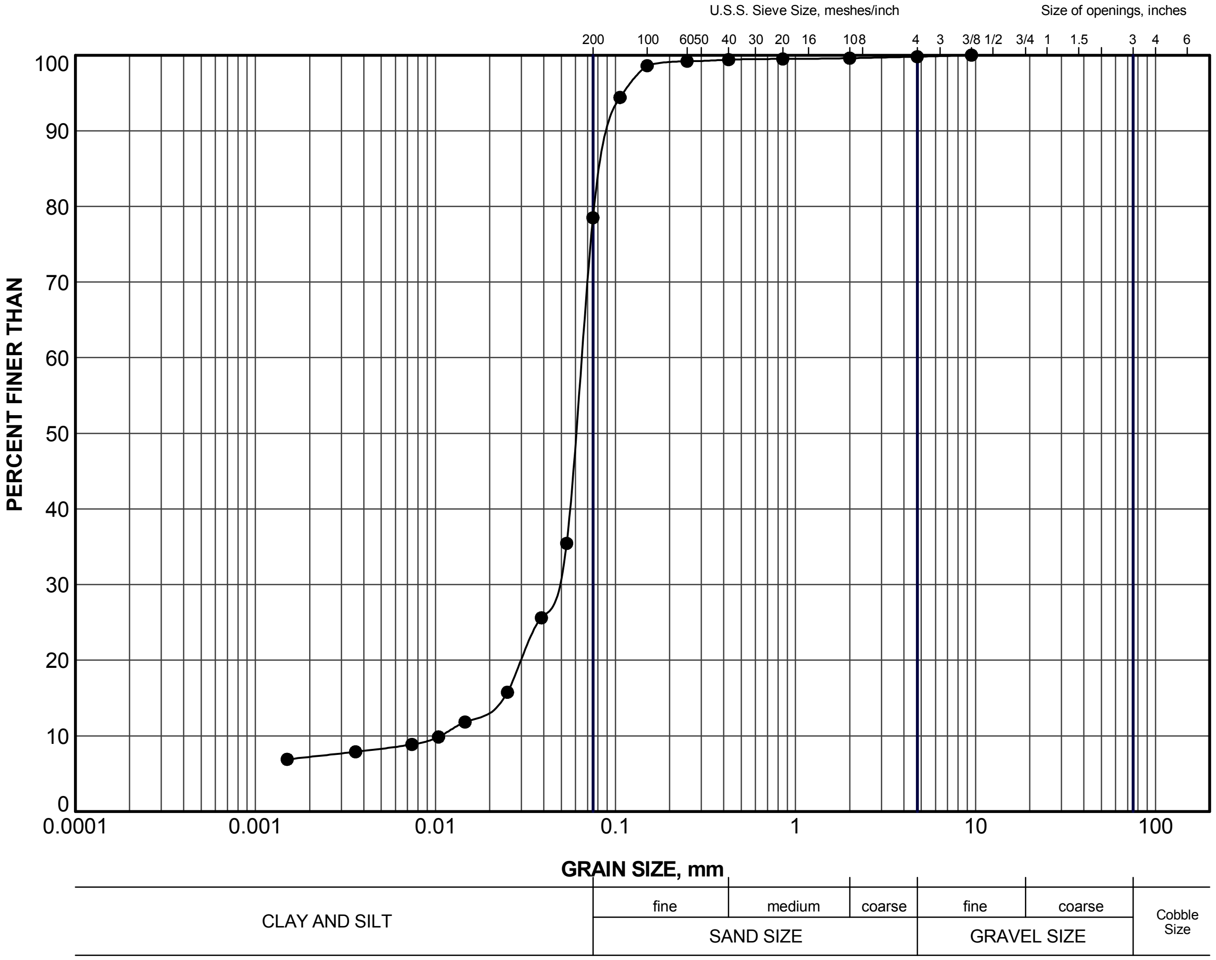
GRAIN SIZE DISTRIBUTION

SAND AND GRAVEL

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R060A9	
			SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-9**	
CHECK				

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	13	166.5

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

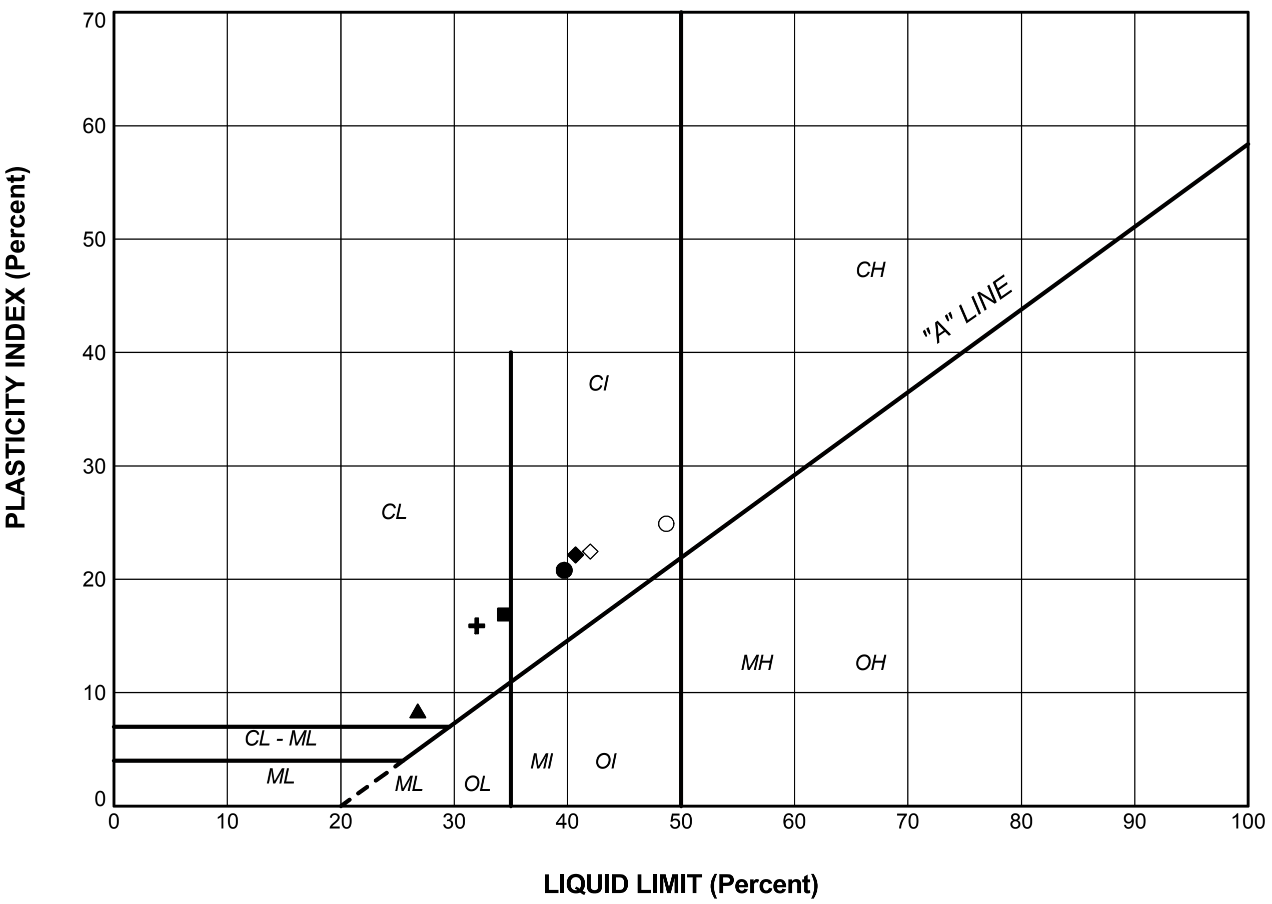
GRAIN SIZE DISTRIBUTION

SILT

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No. 1311320111-1000-R060A10	
			SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-10**	
CHECK				

LDN_MTO_GSD GLDR_LDN.GDT 14/10/14



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI	
●	601	2	39.7	18.9	20.8	(FILL)
■	605	3	34.4	17.5	16.9	(FILL)
▲	606	2	26.8	18.4	8.4	(FILL)
✚	611	2	32.0	16.1	15.9	(FILL)
◆	618	2	40.7	18.6	22.2	(FILL)
◇	619	1	42.0	19.6	22.5	(FILL)
○	622	1	48.7	23.8	24.9	(FILL)

PROJECT
HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

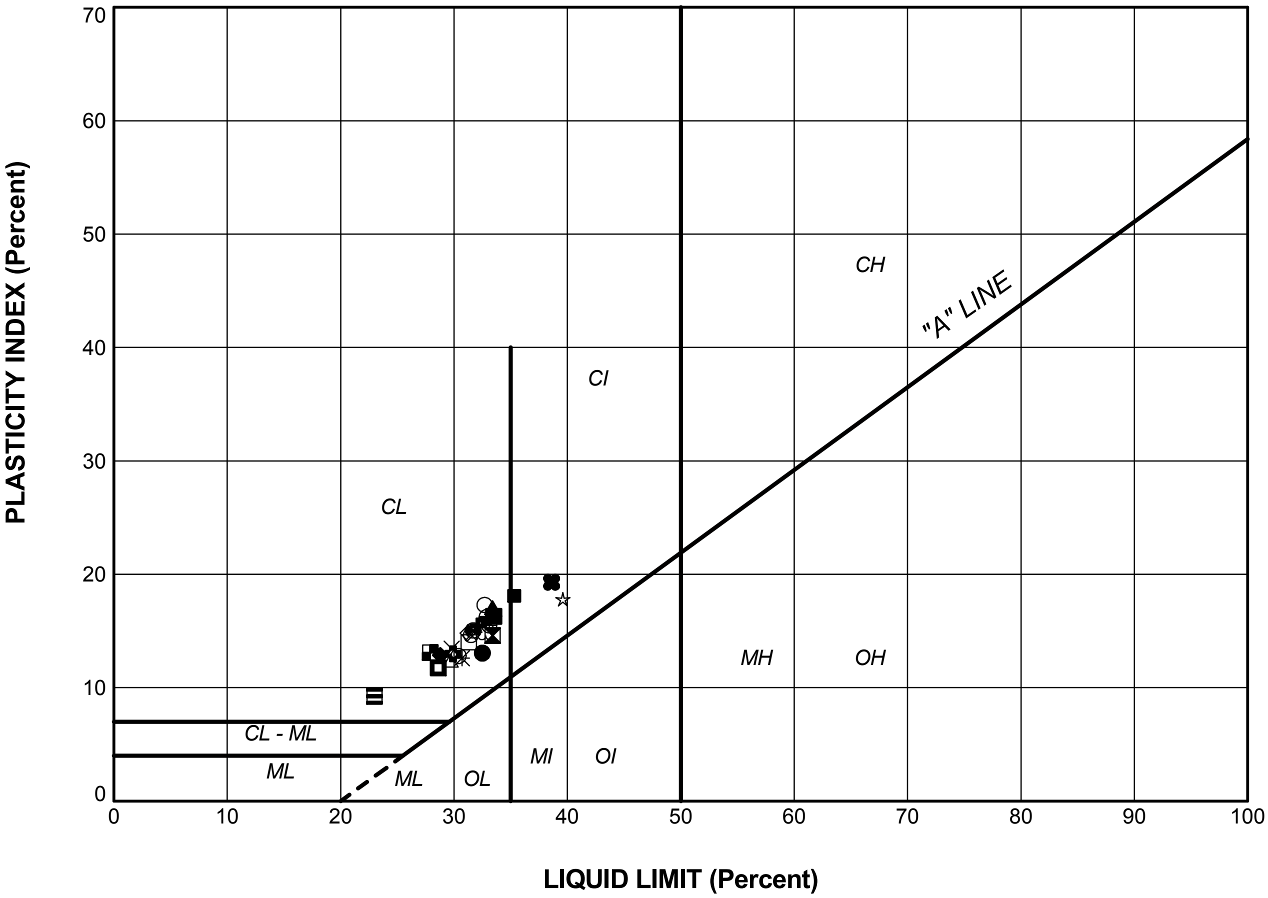
PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No. 13-1132-0111
FILE No. 1311320111-1000-R060A11
SCALE N/A
REV.
DRAWN WDF Oct 14/14
CHECK

FIGURE A-11

LDN_MTO_PI_M GLDR_LDN.GDT 14/10/14



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	403	1	32.5	19.5	13.1
■	403	3	35.3	17.2	18.1
▲	403	8	33.4	16.4	17.1
✚	601	5	32.1	16.7	15.5
◆	602	7	28.8	16.0	12.9
◇	603	6	31.2	16.4	14.8
○	604	7	32.7	15.4	17.3
△	605	6	29.7	17.3	12.4
⊗	606	5	31.5	16.9	14.7
⊕	607	1	33.1	17.6	15.5
□	607	6	31.3	17.3	14.0
ⓧ	608	4	31.7	16.7	15.1
◕	608	7	32.9	16.7	16.3
☆	609	2	39.6	21.8	17.8
✥	609	5	32.8	17.9	14.9
⧗	610	6	33.4	18.8	14.6
⊙	611	6	30.4	17.6	12.8
✜	613	6	29.9	16.9	13.0
✕	614	7	29.8	16.4	13.5
✤	615	4	38.6	19.3	19.3
⊡	615	6	33.5	17.2	16.3
✳	616	2	30.7	18.1	12.6
◘	616	5	28.6	16.9	11.8
▚	616	6	27.9	14.8	13.1
☰	616	10	23.0	13.8	9.3

PROJECT
HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

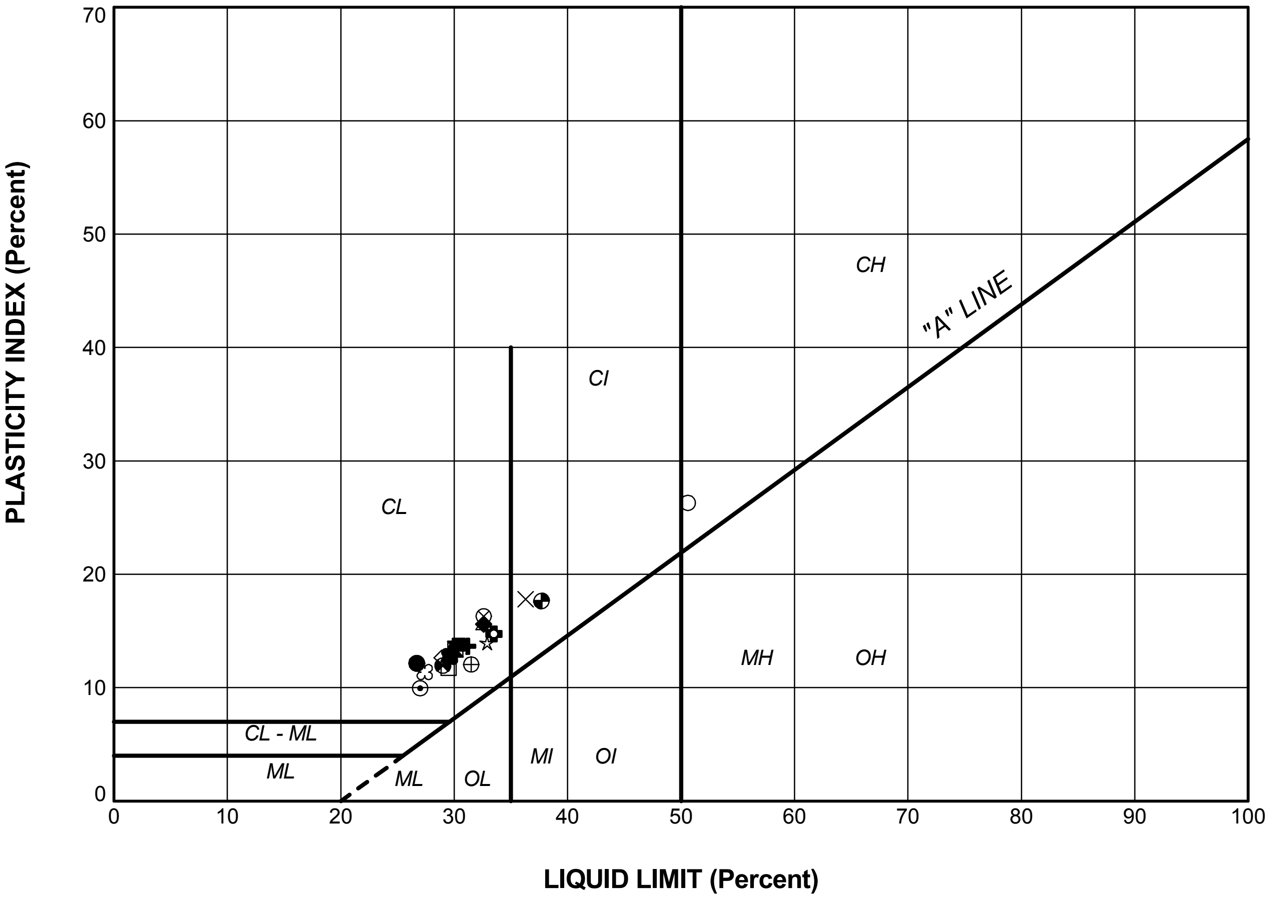
TITLE

PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.	13-1132-0111		FILE No.1311320111-1000-R060A12
			SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-12**
CHECK			

LDN_MTO_PI_25 GLDR_LDN.GDT 14/10/14



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	617	3	26.7	14.6	12.2
■	617	6	30.4	16.6	13.8
▲	617	8	29.7	16.4	13.3
✚	618	7	31.2	17.6	13.7
◆	619	5	32.6	17.0	15.6
◇	619	9	28.9	16.3	12.7
○	620	2	50.6	24.3	26.3
△	620	4	32.6	16.9	15.7
⊗	620	6	32.6	16.3	16.3
⊕	621	2	31.5	19.5	12.1
□	621	3	29.5	17.8	11.8
⊛	621	8	29.0	17.1	12.0
◕	622	2	37.7	20.1	17.7
☆	623	2	32.9	19.0	14.0
⌘	623	4	27.4	16.0	11.4
⧗	623	7	30.1	16.7	13.4
⊙	624	2	27.0	17.1	10.0
✿	624	5	33.5	18.8	14.8
×	625	3	36.3	18.5	17.8
✤	625	6	29.6	16.9	12.8

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

PLASTICITY CHART

Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No. 1311320111-1000-R060A13	
			SCALE N/A	REV.
DRAWN	WDF	Oct 14/14	**FIGURE A-13**	
CHECK				

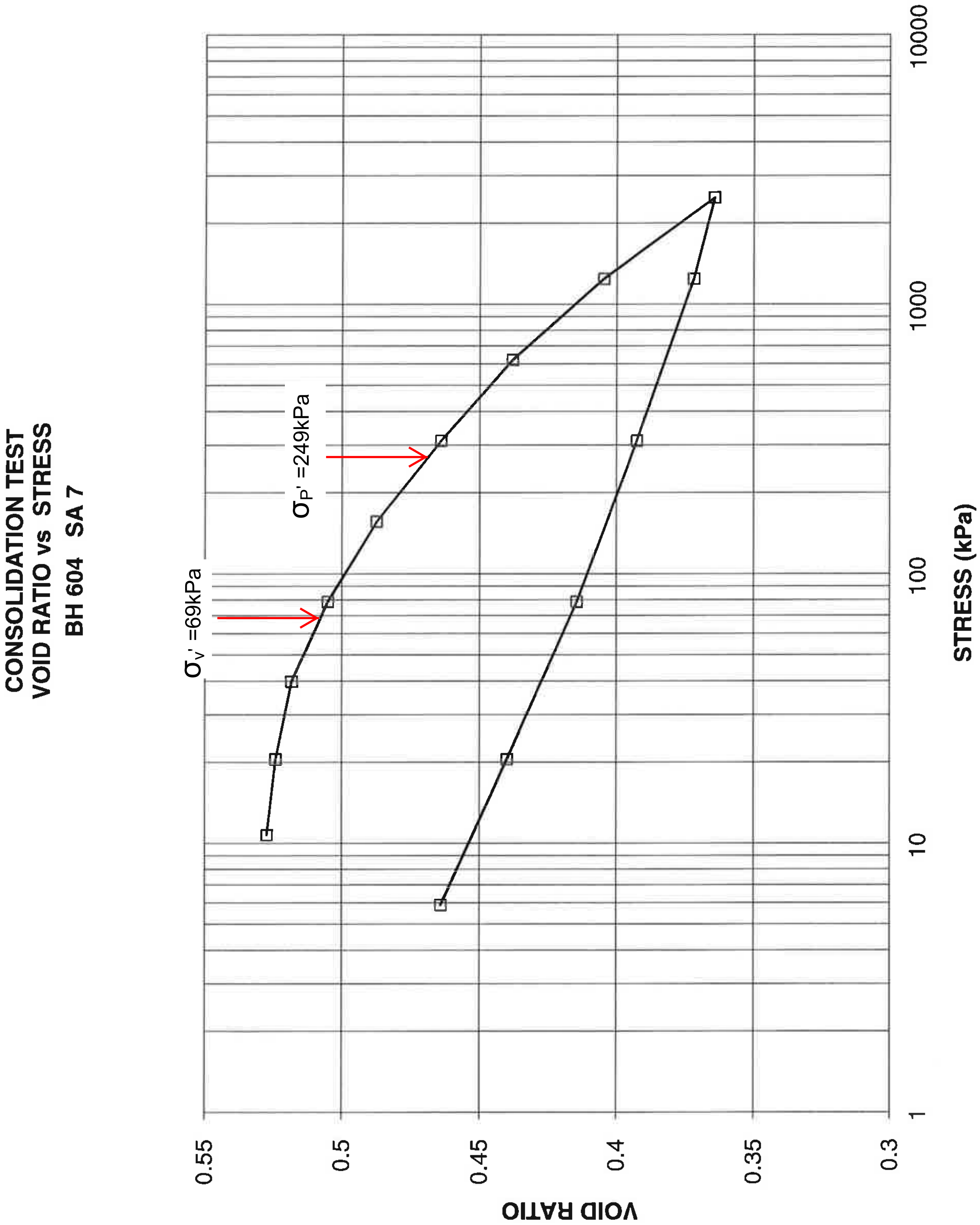
LDN_MTO_PI_25 GLDR_LDN.GDT 14/10/14

APPENDIX B

Laboratory Test Data – Consolidation Testing

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B-1A



Project No. 13-1132-0111

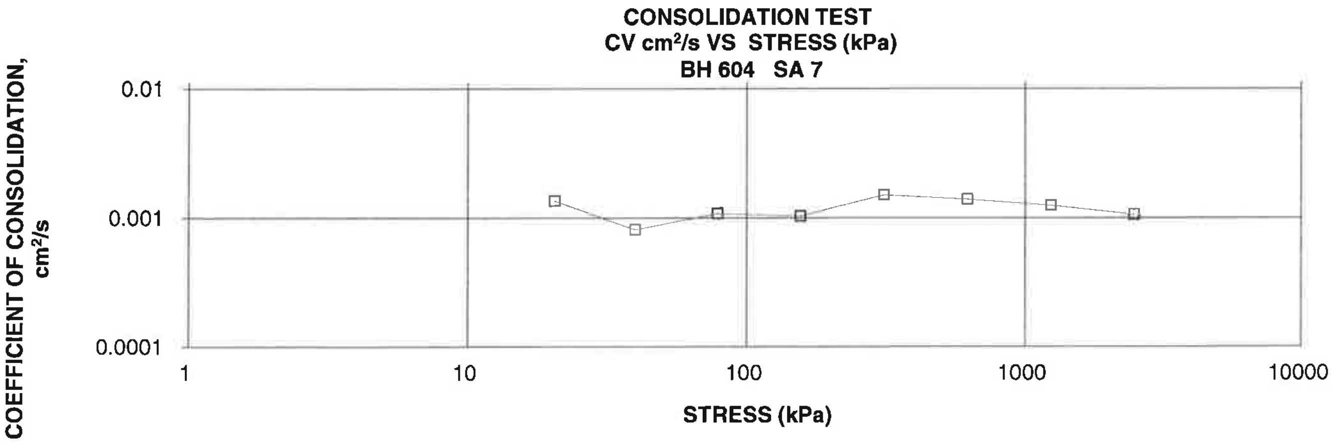
Prepared By: RD

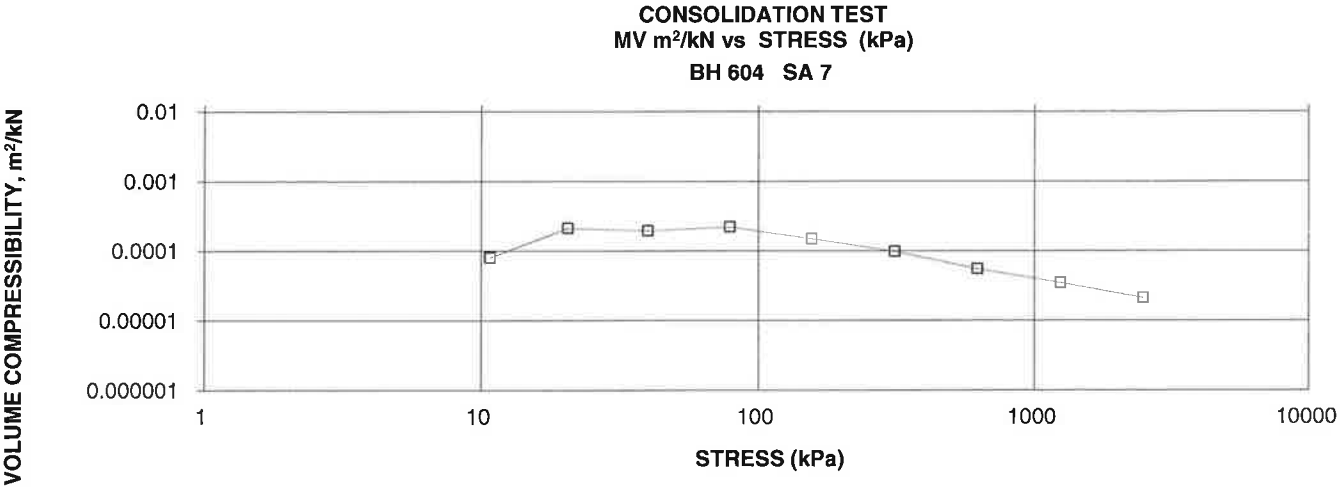
Golder Associates

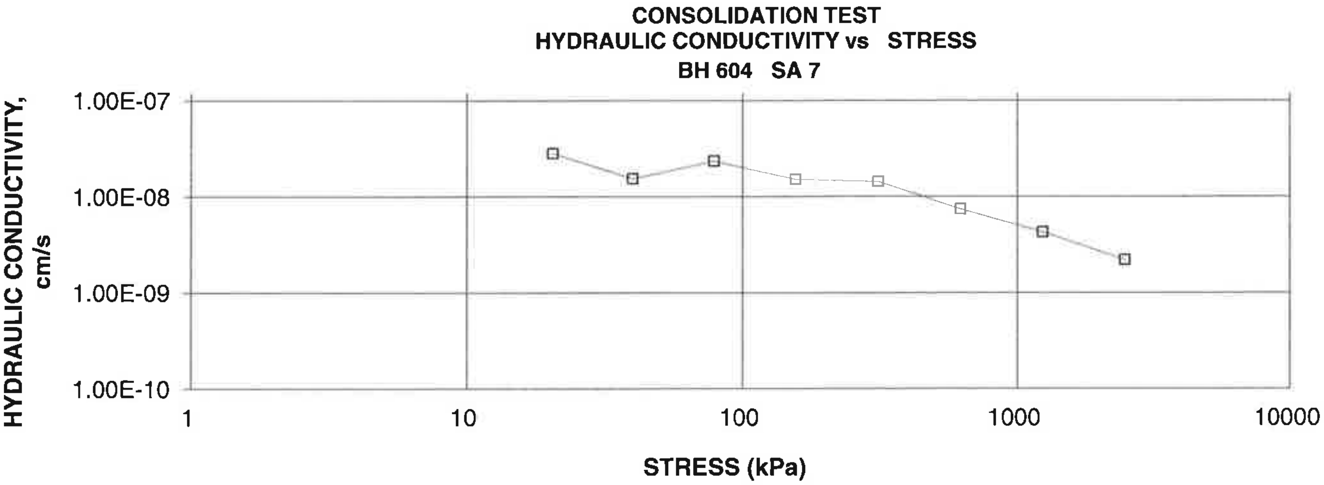
Checked By:

CONSOLIDATION TEST SUMMARY

FIGURE B-1B







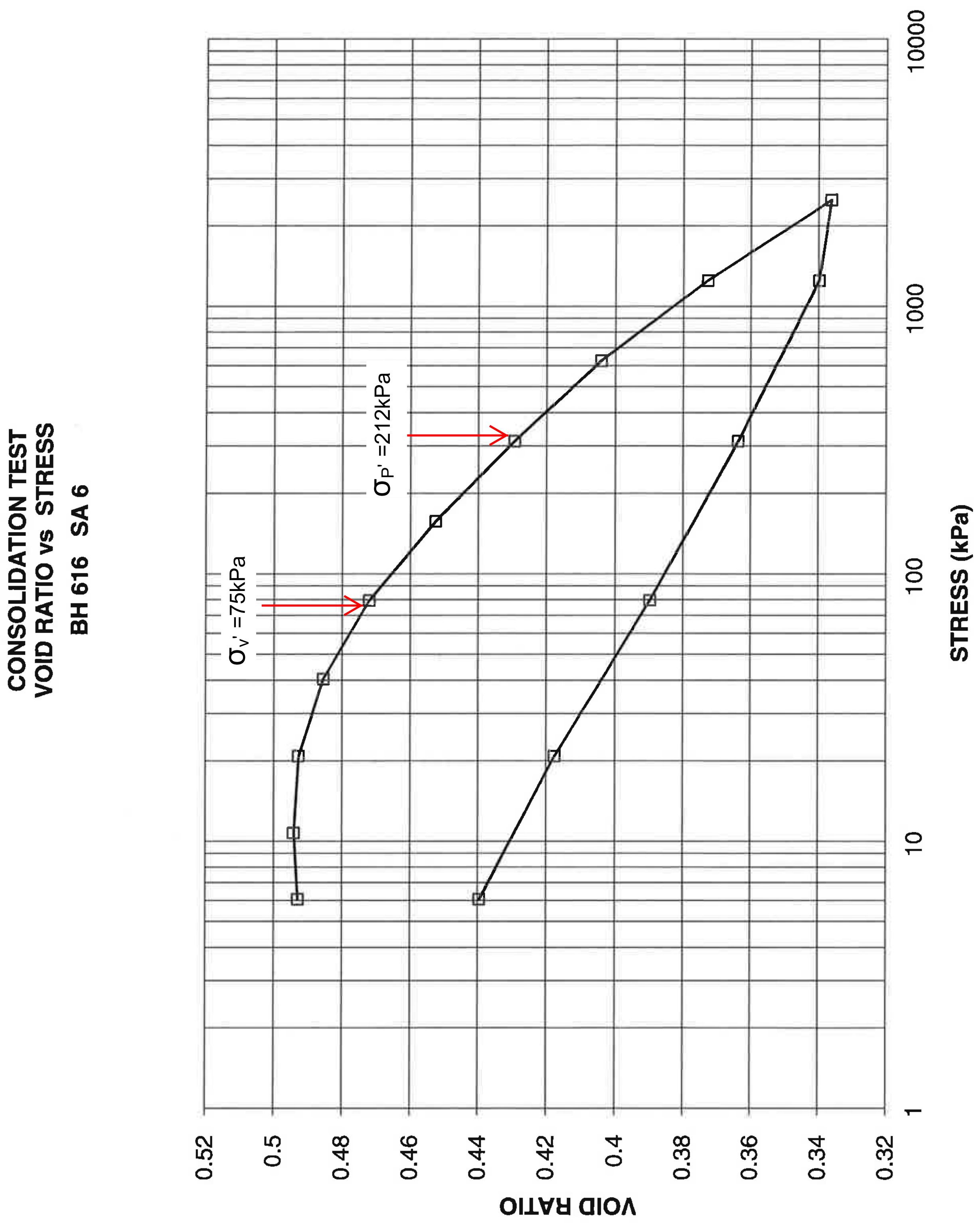
Project No. 13-1132-0111
Prepared By: RD

Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE B-2A



Project No. 13-1132-0111

Prepared By: LG

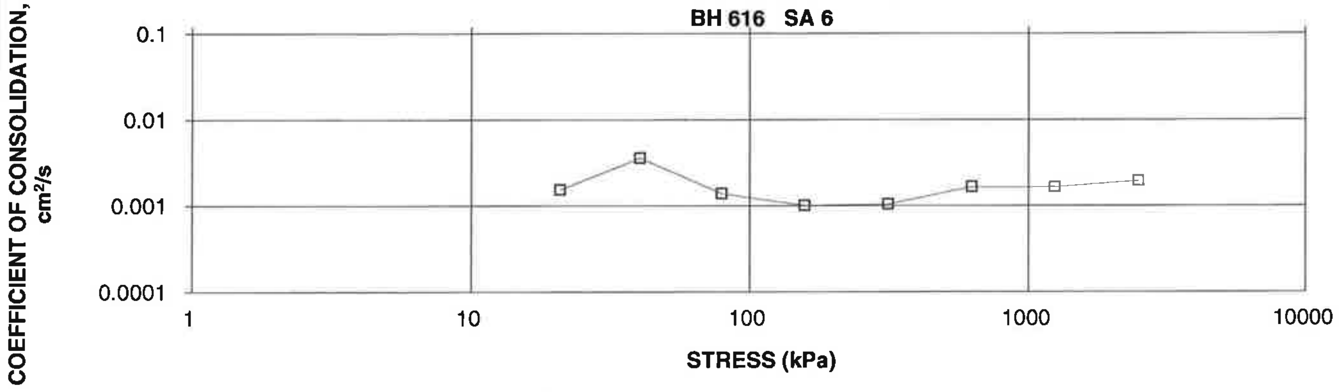
Golder Associates

Checked By:

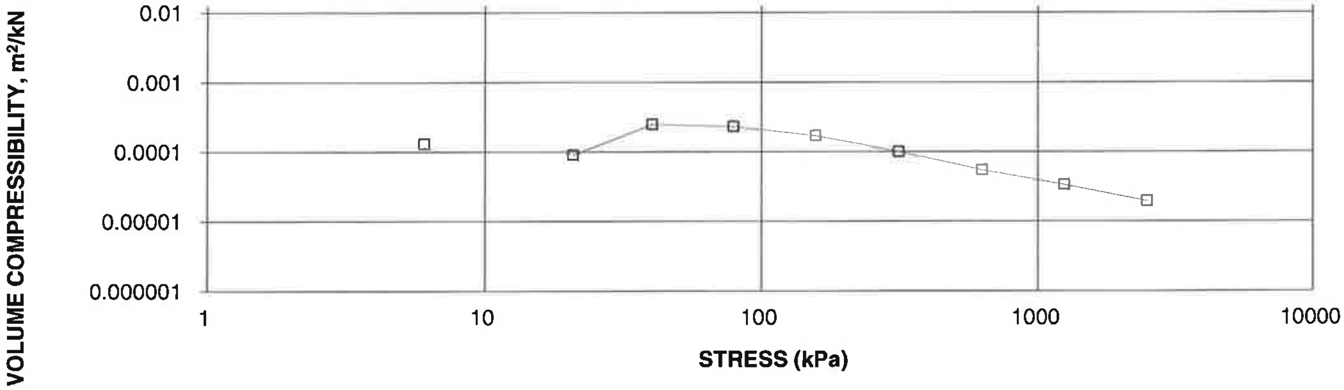
CONSOLIDATION TEST SUMMARY

FIGURE B-2B

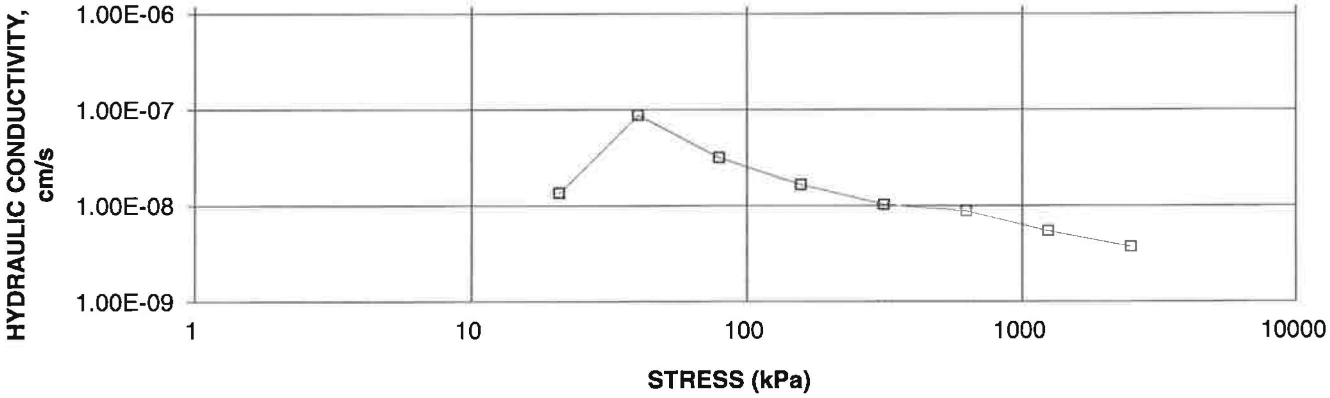
CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH 616 SA 6



CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH 616 SA 6



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 616 SA 6



Project No. 13-1132-0111

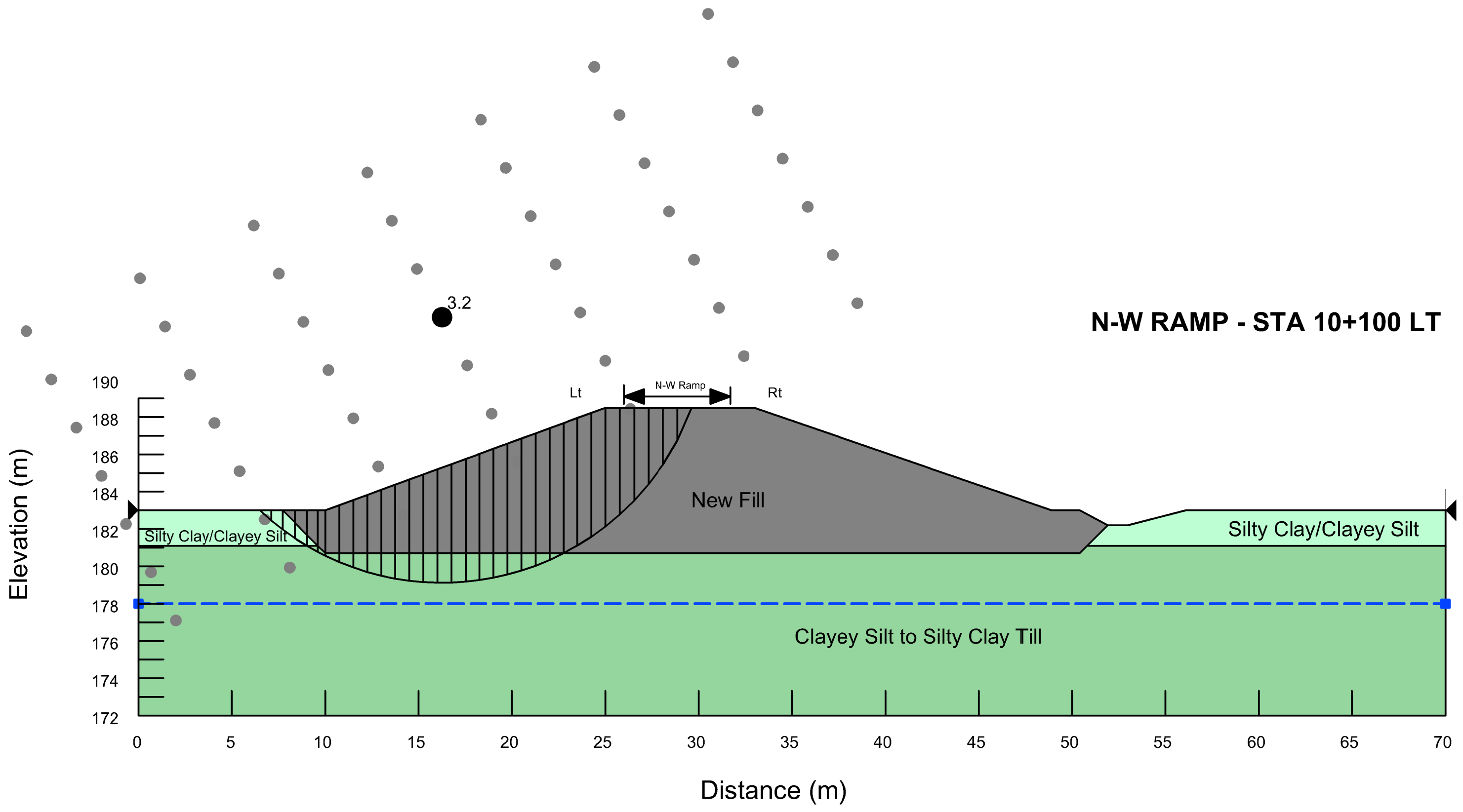
Prepared By: LG

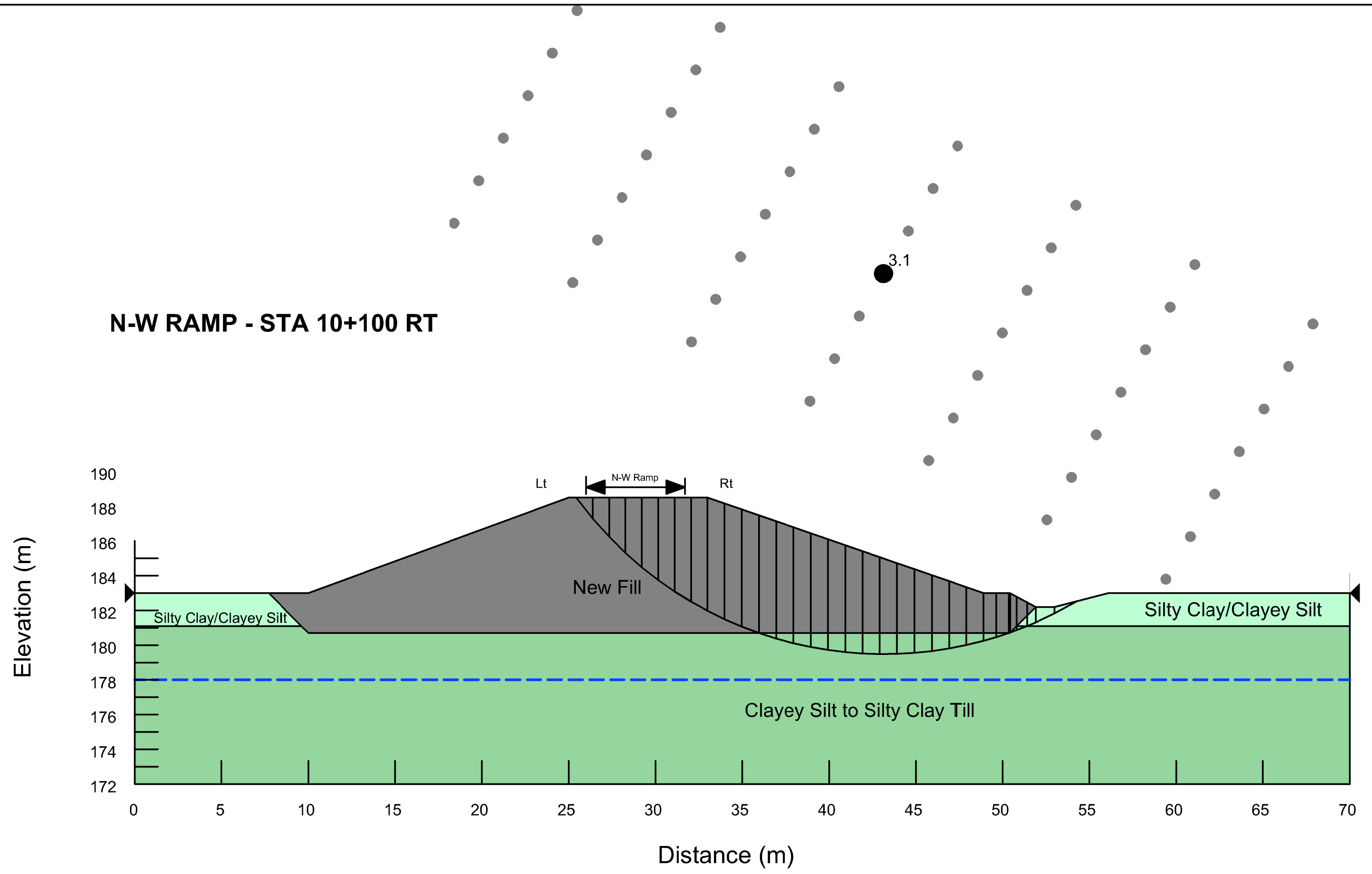
Golder Associates

Checked By:

APPENDIX C

Slope Stability Analyses


N-W RAMP - STA 10+100 LT


N-W RAMP - STA 10+100 RT

SOIL PROPERTIES

Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Silty Clay/Clayey Silt	20	0	28°
Clayey Silt/Silty Clay Till	22	0	32°

NOTES

1. OFFSET = OFFSET FROM ROADWAY CONTROL LINE.
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT
HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

RESULTS OF SLOPE STABILITY ANALYSES

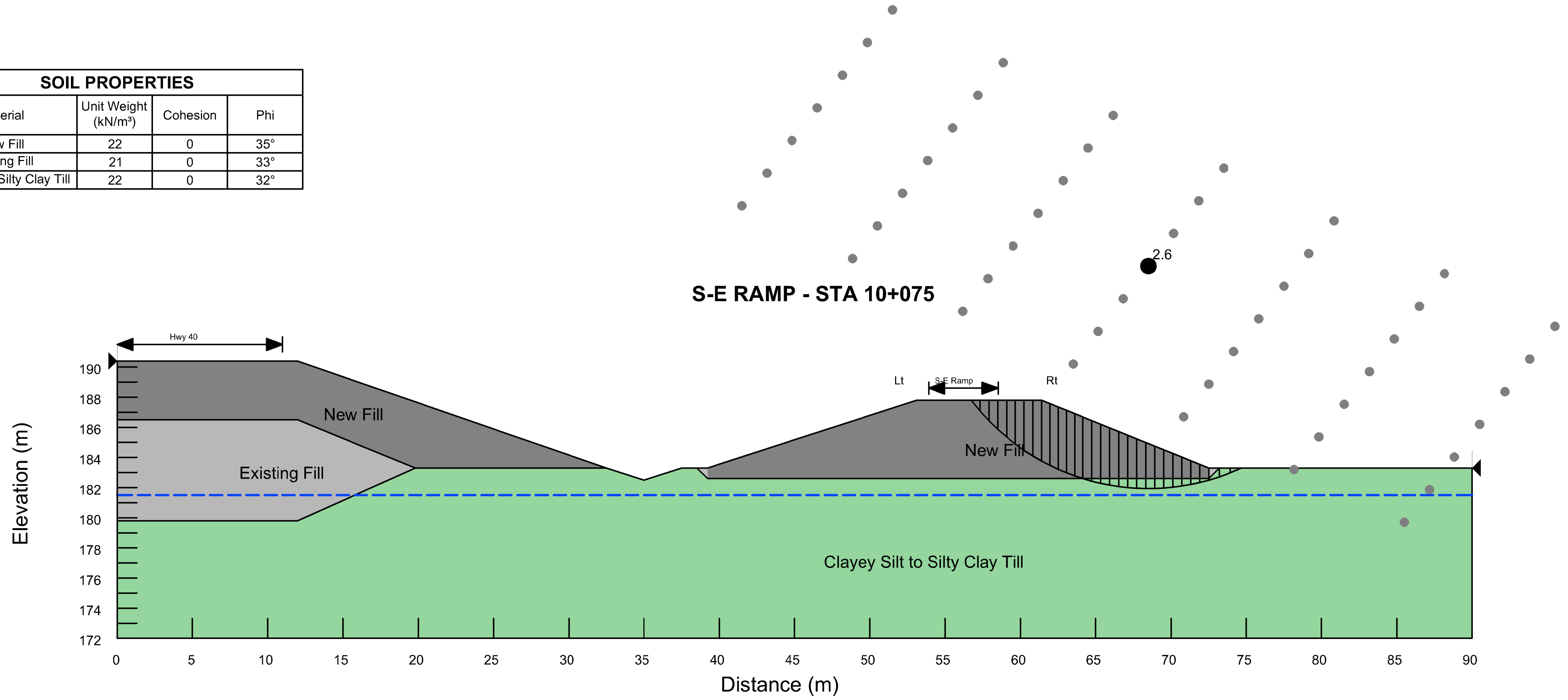
Golder Associates
LONDON, ONTARIO

PROJECT No.		13-1132-0111	FILE No.	1311320111-1000-F060C1
			SCALE AS SHOWN	REV. 0
CADD	WDF/DCH	Nov. 13/14	**FIGURE C-1**	
CHECK				

Drawing file: 1311320111-1000-F060C1.DWG Nov 13, 2014 - 4:25pm

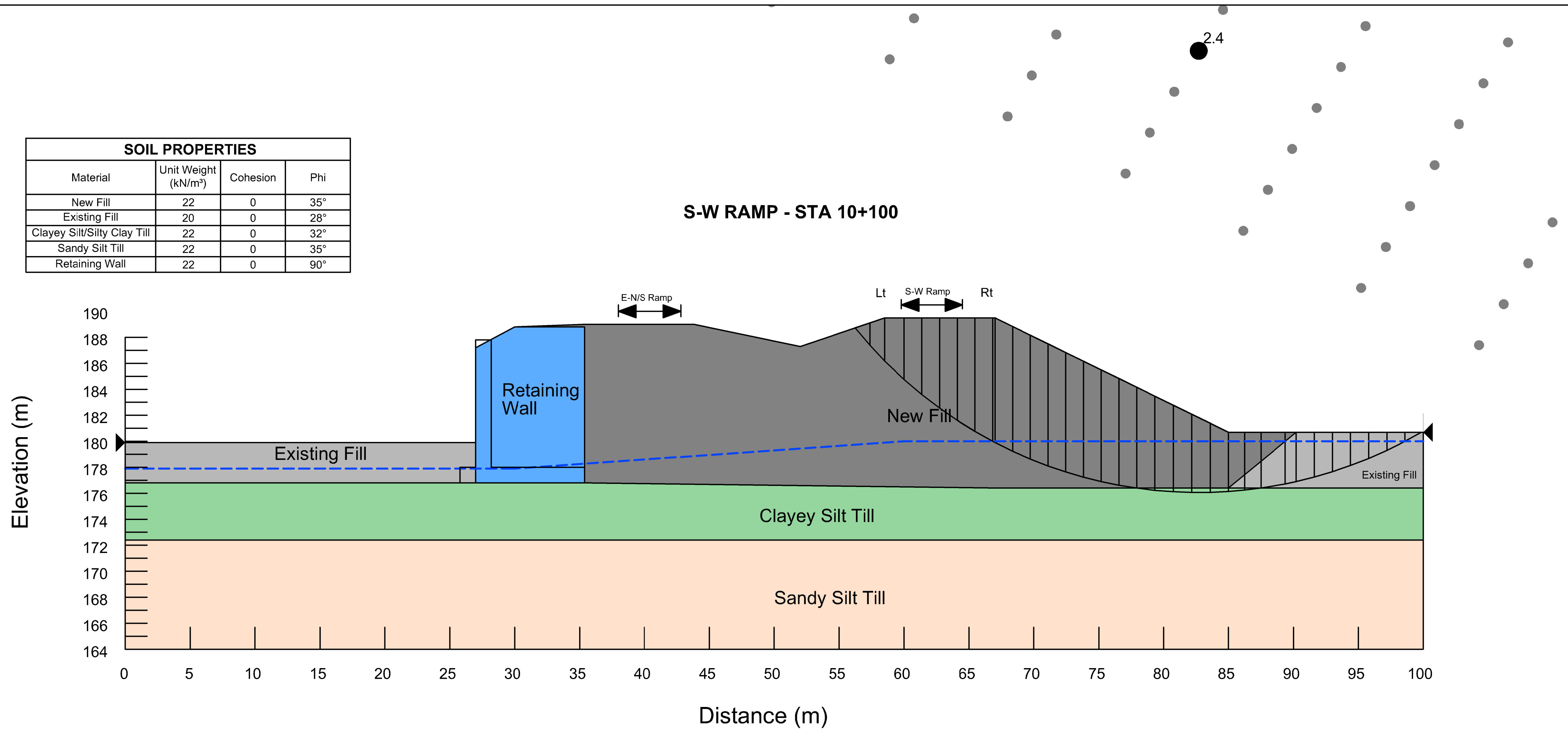
SOIL PROPERTIES			
Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Existing Fill	21	0	33°
Clayey Silt/Silty Clay Till	22	0	32°

S-E RAMP - STA 10+075



SOIL PROPERTIES			
Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Existing Fill	20	0	28°
Clayey Silt/Silty Clay Till	22	0	32°
Sandy Silt Till	22	0	35°
Retaining Wall	22	0	90°

S-W RAMP - STA 10+100



NOTES

1. OFFSET = OFFSET FROM ROADWAY CONTROL LINE.
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT
HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE
RESULTS OF SLOPE STABILITY ANALYSES

Golder Associates
LONDON, ONTARIO

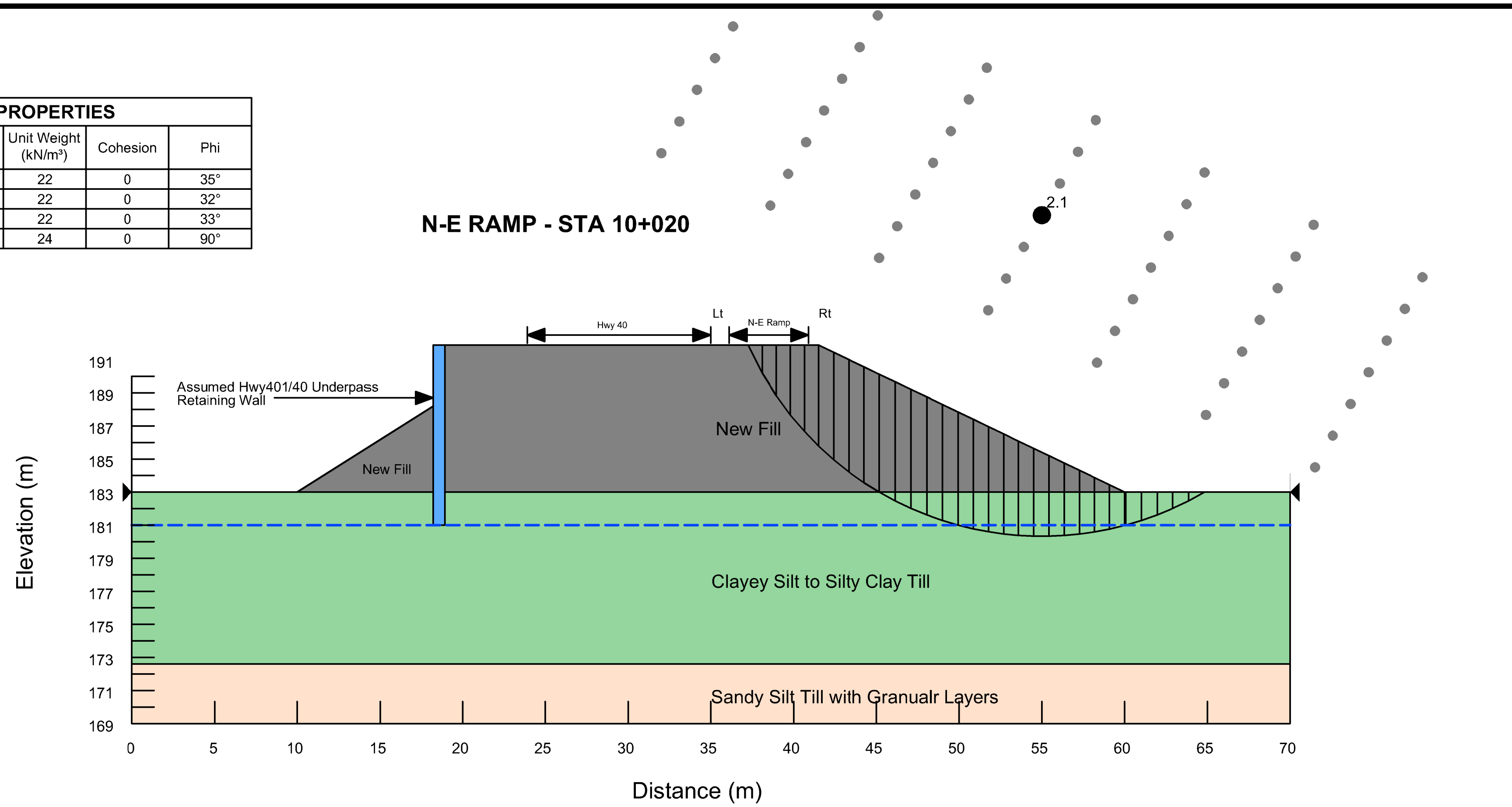
PROJECT No.		13-1132-0111	FILE No.	1311320111-1000-F060C1
			SCALE AS SHOWN	REV. 0
CADD	WDF/DCH	Nov. 13/14	**FIGURE C-2**	
CHECK				

Drawing file: 1311320111-1000-F060C1.DWG   Nov 13, 2014 - 4:24pm

SOIL PROPERTIES

Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Clayey Silt/Silty Clay Till	22	0	32°
Sandy Silt Till, granular layers	22	0	33°
Retaining Wall	24	0	90°

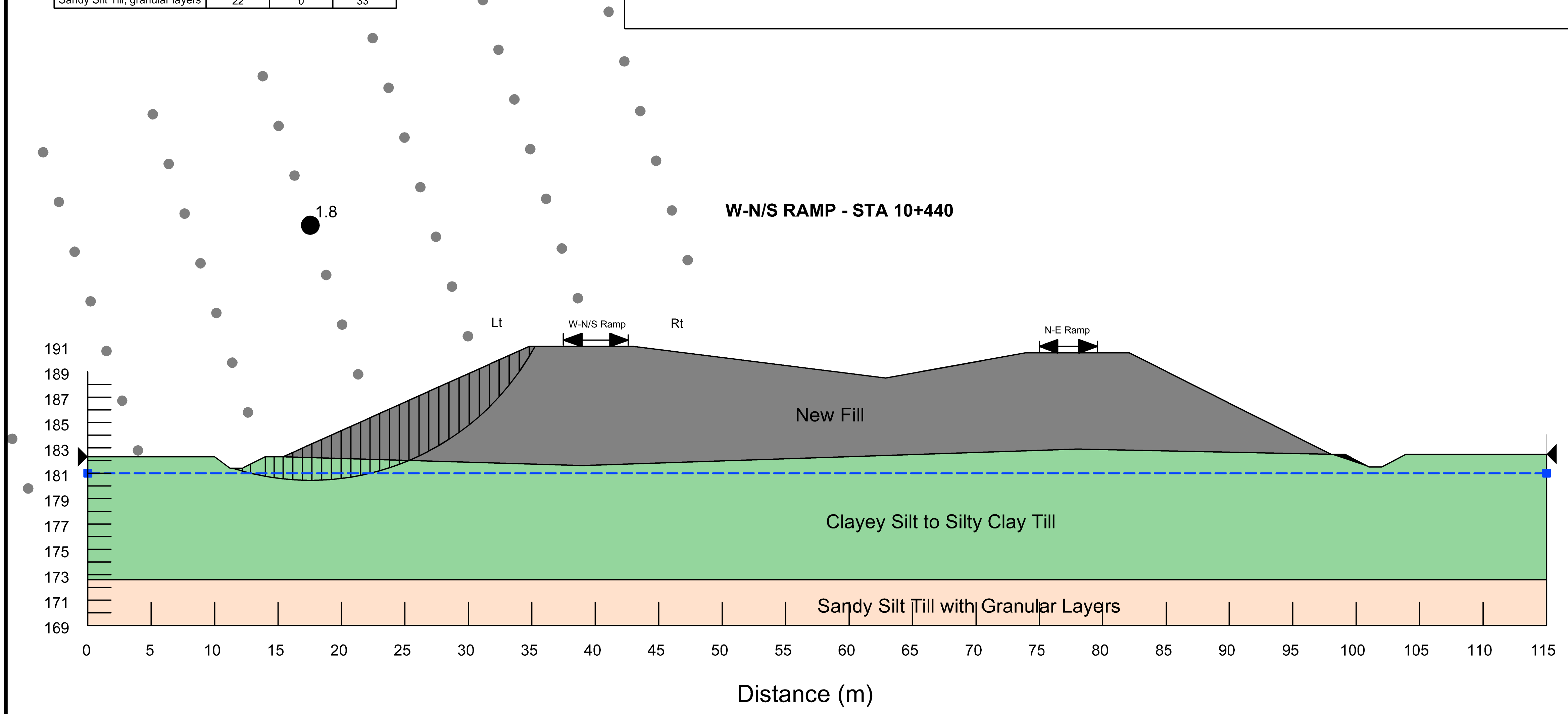
N-E RAMP - STA 10+020



SOIL PROPERTIES

Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Clayey Silt/Silty Clay Till	22	0	32°
Sandy Silt Till, granular layers	22	0	33°

W-N/S RAMP - STA 10+440



NOTES

1. OFFSET = OFFSET FROM ROADWAY CONTROL LINE.
2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT

HIGH FILL RAMP EMBANKMENTS
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

RESULTS OF SLOPE STABILITY ANALYSES

Golder Associates
LONDON, ONTARIO

PROJECT No.	13-1132-0111	
CADD	WDF/DCH	Nov. 13/14
CHECK		

FILE No.	1311320111-1000-F060C1		
SCALE	AS SHOWN	REV.	0

FIGURE C-3

Drawing file: 1311320111-1000-F060C1.DWG Nov 13, 2014 - 4:23pm

APPENDIX D

Special Provisions - Lightweight Fill Materials

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the expanded polystyrene embankment fill, including foundation preparation, excavation, leveling pad, polyethylene sheeting and associated works as shown on the contract drawings.

2.0 REFERENCES

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

National Standards of Canada

CAN/CGSB - 51.20 M87 Thermal Insulation, Polystyrene, Boards and Pipe Covering

American Society for Testing and Materials (ASTM)

ASTM D6817	Standard Specification for Rigid Cellular Polystyrene Geofoam
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Guarded Hot Plate Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Plastics
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS 212	Construction Specification for Borrow
OPSS 501	Construction Specification for Compacting
OPSS 517	Construction Specification for Dewatering of Pipeline, Utility and Associated Structure Excavations
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1605	Material Specification for Extruded Expanded Polystyrene Pavement Insulation
OPSS 1860	Material Specification for Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the geotechnical investigation reports for this Contract.

4.0 DEFINITIONS

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

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

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

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

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure. The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and a method statement that provides full details of the materials and construction procedure.

6.2 Delivery, Storage, Handling and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirements.

6.3 Construction

The contractor shall submit full details of the following.

a) The method of foundation excavation and preparation.
b) Construction of granular leveling pad.
c) The method of placement of expanded polystyrene including temporary ballasting (if required) and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.
d) The method and limits of placement of polyethylene sheeting.
e) The method of placement of protective concrete slab.
f) The method of placement of subbase material.
g) The method of placement of side slope cover.

7. MATERIALS

7.1 Granular Leveling Pad

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

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

a) A general statement as to the type, composition, and method of production of the material.
b) The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
c) Certification of compliance of physical and mechanical properties.
d) An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the expanded polystyrene.
e) The physical and mechanical properties of the rigid expanded polystyrene including:
    1. Geometry
    2. Nominal Density
    3. Compressive Strength
    4. Flexural Strength
    5. Dimensional Stability
    6. Oxygen Index
    7. Water Absorption

f) Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
g) A sample of the expanded polystyrene material to the Contract Administrator for review.

h) To the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents. Certificate to be submitted a minimum of one week prior to commencement of work under this item.

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

7.2.2 Detail Requirements

7.2.2.1 The polystyrene shall meet the requirements for EPS22, as defined by ASTM D6817-02, as follows:

TABLE 1 – MATERIAL PROPERTIES

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	Mm	1200 x 600 x 200 ± 0.5%	
Compressive Strength at 5% strain	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	276	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

The expanded polystyrene shall be supplied in the form of rectangular parallel sheets bundled into minimum acceptable dimensions of 1200 mm x 600 mm x 200 mm.

The maximum deviation from the specified linear dimensions, flatness, squareness and thickness shall be ± 0.5%.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum design permanent stress level must not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 276 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

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

7.2.2.5 Flammability

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

7.2.2.6 Water Absorption

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

7.2.2.7 Chemical Resistance

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

7.2.2.8 Biological Resistance

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

7.2.2.9 Environmental

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

7.3 Polyethylene Sheeting

The polyethylene sheeting shall be 6 mil thick.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer’s name or trademark.
The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer’s recommendation.
Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

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

9.2 Levelling Pad

Place, level and compact a 150 mm thick layer of Granular 'A' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Polystyrene Installation

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

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

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

d) Sloping end adjustments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.

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

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

g) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.

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

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

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

k) The side slope of the rigid expanded polystyrene embankment shall be covered with fill material as detailed elsewhere in this contract.

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

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

10. EQUIPMENT

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

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

11. QUALITY ASSURANCE

11.1 Quality Assurance

Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

11.2 Sampling and Testing

11.2.1 General

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

11.2.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, one (1) block shall be tested for the full suite of tests and three (3) blocks shall be tested for compressive strength.

11.2.3 Acceptance/Rejection

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

12.0 Measurement for Payment

12.1 Actual Measurement

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

13.0 Payment

13.1 Basis of Payment

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

CONCRETE PAD – Item No.

Special Provision

The item Concrete Pad shall refer to the Concrete Pad as shown on the Contract drawings.

1.0 Scope

This special provision covers the requirements for the construction of the concrete pad associated with the expanded polystyrene embankment fill.

2.0 References

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

Ontario Provincial Standard Specifications, Construction:

OPSS 904	Construction Specification for Concrete Structures
OPSS 905	Construction Specification for Steel Reinforcement for Concrete
OPSS 919	Construction Specification for Formwork and Falsework

Ontario Provincial Standard Specifications, Material:

OPSS 1002	Material Specification for Aggregates – Concrete
OPSS 1212	Material Specification for Hot-Poured Rubberized Asphalt Joint Sealing Compound
OPSS 1305	Material Specification for Moisture Vapour Barriers
OPSS 1306	Material Specification for Burlap
OPSS 1308	Material Specification for Joint Filler In Concrete
OPSS 1315	Material Specification for White Pigmented Membrane Curing Compounds for Concrete
OPSS 1350	Material Specification for Concrete - Materials and Production
OPSS 1440	Material Specification for Steel Reinforcement for Concrete

3.0 Submission and Design Requirements

3.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of shop drawings and a method statement that provides full details of materials and the construction procedure.

4.0 Materials

4.01 Concrete and Concrete materials

Concrete and concrete materials shall conform to OPSS 1350 with the following exceptions and/or additions.

Class of Concrete 30 MPa at 28 days
Coarse Aggregate 19 mm nominal maximum size
Air Content 4 - 7%
Maximum Slump 60 mm

4.02 Burlap

Burlap shall conform to OPSS 1306.

4.03 Moisture Vapour Barrier

Moisture vapour barrier for curing shall conform to OPSS 1305.

4.04 Curing Compound

White pigmented membrane curing compounds for concrete shall conform to OPSS 1315.

4.05 Water

Water shall be free of any impurities, which would adversely affect the concrete.

4.06 Joint Materials

Expansion joint filler shall conform to OPSS 1308.

The joint sealing compound shall be hot poured rubberized asphalt conforming to OPSS 1212.

4.07 Reinforcement

The steel reinforcement shall conform to the requirements of OPSS 1440 and shall be placed in accordance with OPSS 905.

5.0 Construction

5.01 General

The work required includes the construction of the concrete pad as detailed in the Contract Drawings in accordance with the requirements of OPSS 904 unless otherwise noted.

5.02 Preparation Work

5.02.01 Setting Forms

Throughout their entire length, forms shall be set true to line and grade and directly in contact with the polyethylene sheeting over the rigid expanded polystyrene. Forms shall be anchored in such a manner so as not to damage the polyethylene or polystyrene.

5.03 Joints

5.03.01 General

Joints shall be of the type and at the locations detailed in the contract. The saw cutting of the joints shall be performed within sufficient time to prevent cracking.

5.03.02 Transverse Joints – Construction

Transverse construction joints shall be made at the end of each day's run or when interruptions occur in the concreting operation. Transverse construction joints shall be formed at a contraction or expansion joint, except in exceptional cases of plant breakdown or adverse weather conditions. In these exceptional cases, a construction joint may be formed in the mid slab area subject to the provision that the portion of the slab placed, and the portion of the slab to be placed, is not less than 3 m in length.

5.04 Tolerance

The surface of the concrete is to be such that when tested with a 3 m long straightedge placed anywhere, in any direction on the surface, except across the crown or drainage gutters, there shall not be a gap greater than 10 mm between the bottom of the straightedge and the surface of the pavement.

5.05 Traffic

Equipment other than rubber-tire sawing equipment shall not be permitted on the concrete until it has attained a minimum compressive strength of 24 MPa.

A lift of Granular B Type II not less than 550 mm thick shall be placed on the concrete pad before traffic is permitted.

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

5.06 Measurement for Payment

5.06.01 Measurement – Concrete Pad

Measurement is by Plan Quantity as may be revised by Adjusted Plan Quantity of the area of concrete pad placed in square metres.

5.07 Basis of Payment

5.07.01 Concrete Pad

Payment at the contract price for the above item(s) shall be full compensation for all labour, equipment and material required to do the work.

CELLULAR CONCRETE – Item No.

Special Provision

The item Cellular Concrete shall refer to Cellular Concrete placed within the zones identified for lightweight fill as shown on the Contract drawings

1.0 Scope

This special provision covers the requirements for the supply and placement of lightweight cellular concrete used as embankment fill. . The provisions of OPSS.PROV 904 apply except as amended or extended herein.

2.0 References

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

Ontario Provincial Standard Specifications, Construction:

OPSS 904	Construction Specification for Concrete Structures

National Standards of Canada

CAN/CSA A3001	Cementitious Materials for Use in Concrete
CSA A23.1	Concrete Materials and Methods of Concrete Construction

American Society for Testing and Materials (ASTM)

ASTM C 869	Standard Specification for Foaming Agents Used in Making Preformed Foam for Cellular Concrete
ASTM C 796	Standard Test Method for Foaming Agents for Use in Producing Cellular Concrete Using Preformed Foam
ASTM C 495-99a	Standard Test Method for Compressive Strength of Lightweight Insulating Concrete Designation: C109/C109M–13
ASTM C109/109M	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)
ASTM D7012	Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

3.0 DEFINITIONS

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

Production Lot: The quantity of cellular concrete produced for a continuously placed lift of cellular concrete.

Quality Verification Engineer: means an Engineer with a minimum of five (5) years' experience related to the design and/or construction of cellular concrete of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the

Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

4.0 QUALIFICATIONS

The cellular concrete supplier shall be certified by the manufacturer of the foaming agent and regularly engaged in the production and placement of cellular concrete. The cellular concrete supplier shall have an adequate number of fully qualified workers who are thoroughly trained and experienced in the production and placement of cellular concrete. The Contractor shall have on site at the commencement of the work a representative of the supplier of the cellular concrete to advise on recommended construction procedure. The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

5.0 SUBMISSION AND DESIGN REQUIREMENTS

5.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and a method statement that provides full details of the materials and construction procedure.

The contractor shall submit full details of the following.

a) The method of foundation excavation and preparation.
b) Method of forming each cellular concrete lift.
c) The method of placement of cellular concrete. The shop drawings shall indicate each planned lift thickness and plan dimensions a layer by layer basis.
d) The method of protecting the top cellular concrete surface from damage during pavement structure placement and compaction.
e) The method of placement of subbase material.
f) The method of placement of side slope cover.

6.0 MATERIALS

6.01 Concrete and Concrete materials

Cellular concrete shall be lightweight engineered fill with the following properties:

Minimum unconfined compressive strength at 28 days of 0.5 MPa.

Wet cast density of 475 kg/m³ (+/-10%).

Portland cement shall conform to the requirements of CSA Standard CAN/CSA A3001, Type GU or HE. Supplementary cementing materials shall conform to the requirements of CSA Standard CAN/CSA A3001.

6.02 Water

Water shall be free of any impurities, which would adversely affect the concrete. Mixing water shall conform to the requirements of CSA Standard A23.1. Water of questionable quality shall not be used unless proven to produce specimens whose 28-day compressive strength is at least 90 % of those made with known acceptable water and an identical material mix.

6.03 Foaming Agent

Foaming agents shall conform to the requirements of ASTM C 869 when tested in accordance with the provisions of ASTM C 796. The Subcontractor shall be pre-qualified and approved in writing by the foaming agent manufacturer referencing this Project.

7.0. EQUIPMENT

The specialized batching, mixing, and placing equipment shall be automated and certified for the purpose by the manufacturer of the cellular concrete material. Drymix equipment must be able to receive bulk cement and produce over 100 cubic metres per hour on-site, continuously, from one piece of equipment, and pump through hoses or pipes up to a flat lineal distance of 1000 metres. Bulk cement shall be weighed on a scale that operates within a tolerance of one and one-half percent (1.5%) per batch. Wet-mix equipment must be able to receive slurry on-site into the equipment and process it continuously during ready-mix supply, and pump through hoses or pipes up to a flat lineal distance of 200 metres. Cellular concrete must be pumped by a positive displacement pump (Peristaltic or similar). A foam generator shall be used to continuously produce pre-formed foam, which shall be injected and mixed with the cementitious slurry downstream of the positive displacement slurry pump. The equipment shall be calibrated to produce a precise and predictable volumetric rate of foam with stable uniform microbubbles.

8.0 CONSTRUCTION

8.01 Foundation Excavation

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

8.02 Cellular Concrete Placement

a) The placement area shall be free of standing water during placement of cellular concrete and until granular material or the next subsequent lift of cellular concrete is placed on top of the completed lift. Snow and ice must be removed from the area prior to placement.

b) Any items to be fully or partially encased in the cellular concrete shall be properly set and stable prior to the installation of the cellular concrete. The Contractor shall provide positive means of preventing uplift and any other movement of embedded items during installation of cellular concrete.

c) Where required, formwork shall be designed and installed to withhold cellular concrete, and may require lining with poly sheeting or similar impermeable membrane to prevent leakage.

d) Cellular concrete may be placed during freezing conditions, provided measures are taken to prevent damage to the cellular concrete until sufficient strength has been attained. Care should be taken to avoid freezing before initial set and insulating systems or heat shall be provided to prevent freezing of the cellular concrete. Cellular concrete must not be placed during precipitation.

e) Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling. Initial discharge of cellular concrete that has accumulated in the discharge lines during prior placements or any cellular concrete mix that has not been fully aerated shall be wasted prior to discharge into the intended lift. Cellular concrete shall not be discharged into the intended lift after the foam generator has been turned off.

f) The maximum lift thickness shall be determined based on density and any other considerations that may affect placement. Cellular concrete shall be cast in a formed area within 1 to 2 hours, to permit undisturbed curing. Foot traffic within the cellular concrete mass shall not be permitted.

g) Finished surface elevation shall be within ±25 mm of the design grades shown on the drawings. Cellular Concrete can be placed with a maximum slope of 1%. Slopes greater than 1% will require profiling by creating steps for the Cellular Concrete with formwork.

h) Loading of, or traffic on the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfill can commence with cellular concrete supports foot traffic without leaving an indentation.

9. QUALITY ASSURANCE

9.01 Quality Assurance

a) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted prior to the installation of the cellular concrete. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor’s Quality Verification Engineer.

b) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer, a minimum of one week prior to the commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Upon completion of the cellular concrete embankment filling the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the cellular concrete has been constructed in conformance with the installation procedures and specifications of the contract documents.

9.02 Sampling and Testing

9.02.1 General

a) The Contract Administrator may undertake an independent testing program of the cellular concrete. Sampling and testing will be carried out in conformance with the relevant test procedure. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

b) Quality test certificates for each production lot of supplied cement and any additives showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator.

9.02.2 Sampling Frequency and Methods

a) Cellular concrete samples must be captured, cured, and tested to verify the compressive strength requirement is satisfied. One sample is comprised of one set of three cellular concrete cylinders. One sample should be taken for each placement, or every 100 m^3, whichever is more frequent.

b) Test cylinders shall be cast in 75 mm by 150 mm cylindrical plastic molds. The sample mold must be lined with “freezer paper” with the plastic side against the cellular concrete. Cellular concrete cylinders shall be cured and tested as per ASTM C495-99a, modified to represent the field curing conditions for geotechnical applications.

c) Fresh cellular concrete density shall be measured and recorded once per production run, or once for every 50 cubic metres, or once per 20 minutes, whichever is more frequent. The density shall be maintained within +/- 10 % of the design density.

d) A minimum of three cube or core samples of the in-place cellular concrete shall be cut by manual methods for each lift prior to placement of any subsequent lift. Core samples shall be tested for compressive strength in accordance with ASTM D 7012. Manually cut samples shall be tested for compressive strength in accordance with ASTM C109/109M. Wet and dry unit weight shall be tested for all samples. Samples shall be taken at top, middle and bottom of each lift.

e) In the event of disagreement between the measurements of unit weight or compressive obtained from the test cylinders or those cut/cored from the in-place materials the test results from the in-place materials shall be considered representative.

9.03 Acceptance/Rejection

Failure of any one of the samples to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the production lot or any alternative mitigation accepted by the Contract Administrator shall be at the Contractor’s expense.

10.0 Measurement for Payment

10.1 Actual Measurement

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

11.0 Payment

11.1 Basis of Payment

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

LIGHTWEIGHT MATERIAL - Item No.

Non Standard Special Provision

SCOPE

This non standard special provision covers the requirements for the supply and placement of the lightweight blast furnace slag.

DEFINITIONS

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

SUBMISSION AND DESIGN REQUIREMENTS

The Contractor shall submit to the Contract Administrator Certificates of Conformance sealed and signed by the Quality Verification Engineer as follows:

1. Prior to the placement of the lightweight fill material on the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the material properties specified in Table 1. The material properties shall be determined using the test procedure specified in Table 1.

2. Following embankment construction, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the requirements of this specification and that the work has been carried out in general conformance with the contract documents and specifications.

In addition, the Contractor shall submit to the Contract Administrator, for information only, all Quality Control Test Results.

MATERIAL

The Lightweight Blast Furnace Slag shall satisfy the physical, mechanical and chemical property requirements specified in Table 1:

LIGHTWEIGHT MATERIAL - Item No.

Table 1: Material Properties and Construction Requirements

Property	Requirement	Test Method
Angle of Internal Friction	> 35 °	ASTM 2850-95
Hydraulic Conductivity	> 8 E-03 cm/s	ASTM 5856-95, Method A
Chemical Composition	The material shall meet the Leachate Criteria Established Under Ontario Regulation 347.	
In-Situ Wet Unit Weight, maximum when placed and compacted in accordance with the requirements of this Special Provision	< 14.5 kN/m^3	ASTM D2922

The Contractor shall retain a laboratory that has been inspected and accepted by the MTO under the "Soil and Rock - High Complexity Testing" to undertake the testing of the material properties. Laboratory testing shall be signed and sealed by an Engineer, licensed to practice in the Province of Ontario

CONSTRUCTION

The Contractor is advised that the lightweight blast furnace slag is susceptible to crushing if overcompacted and that careful construction supervision is required.

The Contractor shall place the lightweight fill material and shall achieve compaction without crushing the material since crushing increases its unit weight.

The Contractor shall place the lightweight fill material without exceeding the specified in-situ unit weight and maintaining crushing of the material below 5%.

To prevent overcrushing and overcompaction, the lightweight fill shall be placed as follows:

1. For embankments, the lightweight fill shall be placed in lifts of 300 mm and compacted by three (3) passes using single drum vibratory equipment such as a Bomag 142 or equivalent.
2. For backfill to structures, the lightweight fill shall be placed in lifts of 300 mm and compacted with 8 passes of manually guided tamper such as a Bomag BPR 30/38 D or equivalent.
3. The Contractor shall place and spread the loose lifts using a rubber tire front-end loader such as a Caterpillar 980 F or equivalent.

Compaction equipment technical details are provided in Table 2.

Table 2 – Compaction Equipment Technical Details

	Bomag 142 D	Bomag BPR 30/38 D
Weights		
▪ Operating weight (kg)	4690±	175±
▪ Mass per square metre of base plate (kg/m^2)	N/A	1439
Dimensions		
▪ Drum width (mm)	1426±	N/A
▪ Drum diameter (mm)	1058±	N/A
▪ Width of Base Plate (mm)	N/A	380
▪ Length of Base Plate (mm)	N/A	730
Drive		
▪ Performance DIN 6271 IFN (kW)	37±	3.7
▪ Performance SAE (Kw)	39.5	N/A
▪ Speed (rpm)	2300	3600
Vibratory System		
▪ Frequency (Hz)	32±	68±
▪ Amplitude (mm)	1.24±	N/A
▪ Centrifugal force (Kn)	66±	30±

QUALITY CONTROL

General

Quality Control (QC) testing shall be carried out by the Contractor for purposes of ensuring that the lightweight fill material is placed and compacted to the requirements specified in the Contract. Field density and field moisture determination shall be made in accordance with ASTM D2922 and ASTM D3017.

Acceptability of compaction shall be based on achieving the target in situ unit weight.

Control Strip

Under the Supervision of the Quality Verification Engineer, the Contractor shall build a control strip to verify that the placement and compaction procedure will achieve the requirements of this Special Provision without evidence of crushing and without exceeding the specified maximum in-situ unit weight of 14.5 kN/m^3.

LIGHTWEIGHT MATERIAL - Item No.

Prior to incorporating any of the material into the work the Contractor shall build a minimum trial area of 400 m^2 in area consisting of two equal lifts of 300 mm thickness. The Contractor shall give the Contract Administrator written notice of the construction of the control strip 48 hours prior to commencement of this work.

Material placed in the control strip shall have the moisture content that will yield the specified in-situ unit weight.

After the trial area is complete, samples for moisture content and in-situ unit weight determination testing shall be as per ASTM D2922.

In addition, Gradation as per ASTM D422-63 before and after compaction effort shall be performed to determine that crushing is kept within 5%.

All test results will be used to determine compliance with the specification. Any proposed changes to the specified compaction method shall be reviewed and approved by the Contract Administrator prior to implementation. The requirements of the control strip must be satisfied as part of the acceptance criteria of any proposed change to the specified compaction method of this Special Provision.

MEASUREMENT OF PAYMENT

The unit measurement will be cubic metres for the lightweight fill material placed in situ as per the requirements of the contract.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour equipment and materials required to do the work.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
309 Exeter Road, Unit #1
London, Ontario, N6L 1C1
Canada
T: +1 (519) 652 0099