



July 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 401/Highway 40 Underpass Replacement
Site No. 13-238

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



A world of
capabilities
delivered locally

Report Number: 13-1132-0111-1000-R01

Geocres No. 40J8-59

Distribution:

8 Copies - Dillon Consulting Limited

1 Copy - Golder Associates Ltd.





Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	2
2.1 General.....	2
2.2 Site Geology.....	2
3.0 INVESTIGATION PROCEDURES.....	4
4.0 SUBSURFACE CONDITIONS.....	6
4.1 Site Stratigraphy.....	6
4.1.1 Pavements.....	6
4.1.2 Topsoil.....	6
4.1.3 Fill.....	7
4.1.4 Clayey Silt.....	7
4.1.5 Silty Clay.....	7
4.1.6 Clayey Silt Glacial Till.....	7
4.1.7 Silty Clay Glacial Till.....	8
4.1.8 Sandy Silt Glacial Till.....	8
4.1.9 Sand.....	9
4.1.10 Sand and Gravel.....	9
4.1.11 Silt.....	9
4.2 Groundwater Conditions.....	10
5.0 MISCELLANEOUS.....	11

PART B - FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS.....	12
6.1 General.....	12
6.2 Bridge Foundations.....	13
6.2.1 Shallow Foundations.....	13
6.2.2 Deep Foundations.....	18
6.3 Retaining Walls.....	22
6.3.1 Retaining Wall Foundations.....	23



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

6.3.2	Resistance to Lateral Forces	24
6.4	Liquefaction Potential and Seismic Analysis	24
6.4.1	Seismic Parameters	24
6.4.2	Seismic Hazard Assessment	25
6.5	Lateral Earth Pressures for Design	25
6.6	Construction Considerations	26
6.6.1	Shallow Foundations	26
6.6.2	Deep Foundations	27
6.7	Embankments	27
6.7.1	Settlement	28
6.7.2	Settlement Mitigation	30
6.7.3	Instrumentation and Monitoring	33
6.8	Excavations and Temporary Cut Slopes	34
6.8.1	General	34
6.8.2	Temporary Roadway Protection	34
7.0	MISCELLANEOUS	36
TABLE I - Comparison of Foundation Alternatives		
LIST OF ABBREVIATIONS		
LIST OF SYMBOLS		
RECORD OF BOREHOLE SHEETS		
FIGURE 1 - Key Plan		
DRAWINGS 1 to 3 - Borehole Locations and Soil Strata		
APPENDICES		
APPENDIX A		
Laboratory Test Data – Routine Soils		
APPENDIX B		
Laboratory Test Data – Consolidation Testing		
APPENDIX C		
Site Photographs		
APPENDIX D		
Special Provisions - Lightweight Fill Materials		



PART A

FOUNDATION INVESTIGATION REPORT

**HIGHWAY 401/HIGHWAY 40 UNDERPASS REPLACEMENT, SITE 13-238
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 reconstruction of the Highway 401/Highway 40 underpass, Site 13-238.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed replacement structure by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal (RFP) and in Golder'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 were 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. The Highway 401 pavement surface has an elevation of about 184.5 metres at the interchange. Highway 40 has a pavement surface at the underpass near elevation 189.7 metres. The existing underpass was constructed in 1961 and rehabilitated in 1989. The bridge is a four-span concrete tee beam (boat type) bridge with a total length of about 79 metres and accommodates two lanes of Highway 40 traffic. The area immediately surrounding the interchange generally consists of flat-lying agricultural lands and commercial properties on the south side of Highway 401, and the McGregor Creek wetland and residential properties on the north side. Site photographs are provided in Appendix C.

2.2 Site Geology

This project lies within the physiographic region of southwestern Ontario known as the Bothwell Sand Plain which was the delta of the Thames River in prehistoric glacial Lake Warren. The Bothwell Sand Plain primarily consists of a thin layer of sand, approximately 1 metre thick, over the clay floor.¹ Quaternary geology mapping indicates that 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 muck. Based on geologic mapping the underlying bedrock surface is 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

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.



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

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.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between May 6 and June 25, 2014 during which time seven boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations, and borehole depths.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
101	4 693 678	338 414	182.81	22.40
102	4 693 656	338 404	189.78	15.70
103	4 693 725	338 307	189.72	17.22
104	4 693 653	338 373	182.96	22.55
105	4 693 680	338 336	183.00	23.93
106	4 693 721	338 348	182.80	22.63
107	4 693 700	338 298	183.49	19.63

The investigation was carried out using truck and track-mounted drilling equipment supplied and operated by specialist drilling contractors. 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 addition, field vane shear strength testing was carried out in accordance with ASTM D2573 to determine the undrained shear strength of softer cohesive soils encountered in the boreholes.

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 standard 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 penetration, a penetration resistance representing the number of blows to drive the sampler for the full penetration distance 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.



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

The boreholes were terminated between 15.7 and 23.9 metres below the existing pavement or ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. A groundwater observation piezometer was installed in borehole 101 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 a selected Shelby tube sample. 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.



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 Appendix 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 and rock types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered surficial topsoil or pavement structure overlying fill materials, underlain by a deposit of clayey silt till over sandy silt till with varying layers of sand, sand and gravel and silt.

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 401/Highway 40 structure. A distinctly different layer of firm to stiff silty clay was identified at and below approximately elevation 167 metres. The transition between the firm to stiff silty clay found below elevation 167 metres near the Highway 40/Lucas Drain bridge and the granular layers below elevation 167 metres at the Highway 401/Highway 40 structure is not known but may occur within the length of the high fill embankment leading to the Highway 401/Highway 40 structure.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawings 1 to 3. Detailed descriptions of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and are summarized in subsequent report sections.

4.1.1 Pavements

Boreholes 102 and 103 were advanced through the existing pavement structure of Highway 40 and encountered 240 millimetres of asphaltic concrete, underlain by 210 millimetres of granular road base material and 300 and 400 millimetres, respectively, of granular sub-base material. Borehole 105 was advanced in the centre median of Highway 401 and encountered 300 and 340 millimetres of granular base and subbase materials, respectively, from the ground surface. A single measured N value from standard penetration testing carried out in the granular subbase material in borehole 103 was 7 blows per 0.3 metres.

4.1.2 Topsoil

Boreholes 101, 104, 106 and 107 were advanced in each quadrant of the Highway 401/Highway 40 interchange in grassed areas of the site and encountered 150 to 270 millimetres of surficial topsoil. Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.



4.1.3 Fill

Boreholes 102 and 103 were advanced through the existing approach embankments and encountered 6.4 and 6.6 metres, respectively, of silty clay and clayey silt embankment fill beneath the pavement structure. The embankment fill was encountered to elevations 182.6 and 182.3 metres. Measured N values in the embankment fill ranged from 5 to 15 blows per 0.3 metres indicating a firm to stiff consistency. Samples of the silty clay embankment fill had moisture contents of 17 and 18 per cent, liquid limits of 36 and 40 per cent, plastic limits of 20 and 29 per cent, and plasticity indices of 17 and 20 per cent, indicating intermediate plasticity. Samples of the clayey silt embankment fill had moisture contents of 20 and 26 per cent, liquid limits of 30 and 34 per cent, plastic limits of 17 and 18 per cent, and plasticity indices of 13 and 16 per cent, indicating low plasticity.

Fill materials were encountered beneath the topsoil in borehole 104 and beneath the pavement structure in borehole 105. The fill in borehole 104 was 2.0 metres thick and consisted of silty clay and clayey silt. Measured N values in the silty clay fill were 8 blows per 0.3 metres with a corresponding sample having a water content of 25 per cent, liquid and plastic limits of 56 and 24 per cent, respectively, and a plasticity index of 32 per cent, indicating high plasticity. The fill in borehole 105 was 0.7 metres thick and consisted of silt. A single N value from the silt fill was 17 blows per 0.3 metres indicating a compact relative density.

Grain size distribution curves for samples of the fill materials are provided on Figure A-1. The results of the Atterberg limits determinations are shown on Figure A-9.

4.1.4 Clayey Silt

Stiff to very stiff clayey silt, 1.1 and 0.8 metres thick, was encountered beneath the topsoil in borehole 101 and beneath the fill material in borehole 105 at elevations 182.5 and 181.6 metres, respectively. Measured N values in the clayey silt were 12 and 21 blows per 0.3 metres. A sample of the clayey silt obtained in borehole 105 had a water content of 19 per cent. The grain size distribution curve for the sample of the clayey silt is provided on Figure A-2.

4.1.5 Silty Clay

Stiff to very stiff silty clay, 1.5 and 0.8 metres thick, was encountered beneath the fill materials in borehole 103 and beneath the clayey silt in borehole 105, at elevations 182.5 and 181.6 metres, respectively. Measured N values in the silty clay ranged from 9 to 12 blows per 0.3 metres. A sample of the silty clay obtained in borehole 103 had a water content of 24 per cent, liquid and plastic limits of 43 and 22 per cent, respectively, and a plasticity index of 21 per cent, indicating intermediate plasticity. The results of the Atterberg limits determination are shown on Figure A-9. The grain size distribution curve for the sample of the silty clay is provided on Figure A-3.

4.1.6 Clayey Silt Glacial Till

A deposit of stiff to very stiff clayey silt glacial till was encountered beneath the clayey silt and silty clay in boreholes 101, 103 and 105, beneath the fill materials in boreholes 102 and 104, and beneath the topsoil in



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

boreholes 106 and 107, between elevations 180.1 and 183.3 metres. Where fully penetrated, the clayey silt till deposit was between 6.6 and 9.4 metres thick. Borehole 102 was terminated in the clayey silt till after penetrating the layer 8.5 metres. A layer of silty clay till was encountered within the clayey silt till in borehole 101 as described below. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the clayey silt glacial till due to its depositional history.

Measured N values in the clayey silt till ranged from 8 to 30 blows per 0.3 metres. Field vane shear strength testing carried out in the clayey silt till in borehole 105 indicated undrained shear strengths of greater than 144 kilopascals (kPa) indicating very stiff consistency. Triaxial shear strength testing was carried out on seven samples retrieved from between elevations 175 and 181 metres for Geocres Report No. 40J8-15, the Foundation Investigation and Design Report prepared in 1960 for the existing structure. The shear strength of the samples ranged from 195 to 228 kPa indicating a very stiff to hard material.

Samples of the clayey silt till had measured water contents ranging from 11 to 21 per cent. Seven Atterberg limits determinations were carried out on the clayey silt till, the results of which are shown on Figure A-9. Samples of the clayey silt till had liquid limits ranging from 24 to 33 per cent, plastic limits ranging from 13 to 19 per cent, and plasticity indices ranging from 8 to 14 per cent, indicating low plasticity. Grain size distribution curves for the samples of the clayey silt till are provided on Figure A-4.

Consolidation testing carried out on a sample of the clayey silt till indicated the geotechnical engineering properties summarized below. 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)
104	9	7.3	106	0.46	0.012	0.123	232

4.1.7 Silty Clay Glacial Till

Layers of very stiff silty clay glacial till, 1.1 and 1.5 metres thick, were encountered in borehole 101 within and immediately below the clayey silt till at elevations 173.1 and 179.0 metres. Measured N values from the silty clay till ranged from 17 to 27 blows per 0.3 metres. Cobbles and boulders should be anticipated within the silty clay glacial till deposit.

4.1.8 Sandy Silt Glacial Till

Layers of sandy silt glacial till were encountered beneath the clayey silt till in boreholes 103, 104, 105, 106 and 107, and at depth beneath layers of silt and sand in boreholes 101, 104, 105, 106 and 107. The upper layers of sandy silt till were encountered between elevations 172.6 and 173.9 metres while the lower layers were encountered between elevations 163.6 and 166.4 metres. The sandy silt till layers were between 0.8 and 3.2 metres thick where fully penetrated. Boreholes 101 and 103 were terminated in the sandy silt till after exploring the layers for 3.8 and 0.8 metres, respectively.



Samples of the upper sandy silt till layers had measured N values ranging from 30 to 87 blows per 0.3 metres indicating dense to very dense relative density. The lower sandy silt till layers had N values ranging from 71 to greater than 100 blows per 0.3 metres indicating very dense relative density. Water contents of samples of the sandy silt till typically ranged from 9 to 12 per cent with a single sample having a water content of 19 per cent.

Based on eight grain size distribution analyses, the results of which are provided on Figure A-5, the sandy silt till ranged in gradation from sandy silt to sand and silt, with varying amounts of gravel and clay.

4.1.9 Sand

Layers of compact to very dense sand were encountered beneath the silty clay till in borehole 101, beneath layers of sandy silt till in boreholes 104 and 107, and in borehole 105 beneath layers of sand and gravel and silt between elevations 159.8 and 172.0 metres. Where fully penetrated, the sand layers were between 2.9 and 7.8 metres thick. Boreholes 105 and 107 were terminated in sand after exploring the layers for 0.8 and 0.9 metres, respectively. The sand was found to vary in gradation from fine silty sand to coarse sand.

Measured N values in the sand layers ranged from 8 to greater than 100 blows per 0.3 metres, with typical values of 42 blows per 0.3 metres and greater. Samples of the sand had water contents ranging from 11 to 15 per cent. Grain size distribution curves for samples of the sand are provided on Figure A-6.

4.1.10 Sand and Gravel

Layers of dense to very dense sand and gravel were encountered beneath sandy silt till layers in boreholes 105 and 106 at elevation 171.5 metres. The sand and gravel layers were between 3.2 and 5.8 metres thick where fully penetrated. Borehole 106 was terminated in a lower layer of sand and gravel after exploring the layer for 1.6 metres. Cobbles and boulders were encountered within the sand and gravel in borehole 106. Measured N values in the sand and gravel ranged from 31 to greater than 100 blows per 0.3 metres. Samples of the sand and gravel had water contents of 11 and 13 per cent. Grain size distribution curves for samples of the sand and gravel are provided on Figure A-7.

4.1.11 Silt

Silt layers, between 0.9 and 1.5 metres thick, were encountered beneath layers of sandy silt till, sand, and sand and gravel in boreholes 104, 105 and 106 between elevations 161.3 and 165.7 metres. Borehole 104 was terminated after exploring the silt layer for 0.9 metres. Measured N values from the silt layers ranged from 64 to greater than 100 blows per 0.3 metres, indicating very dense relative density. Samples of the silt had water contents of 10 and 18 per cent. Grain size distribution curves for samples of the silt are provided on Figure A-8.



4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. The groundwater level was not established in boreholes 102 and 103 prior to completion of drilling, nor in boreholes 104, 105 and 106 prior to mud rotary drilling. The groundwater level in boreholes 101 and 107 was encountered during drilling at elevations 172.0 and 172.5 metres, respectively. Also, a piezometer was installed in borehole 101 as shown on the Record of Borehole sheets. The encountered and measured groundwater levels from borehole 101 are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level		Measured Groundwater Elevation (m)					
		Depth (m)	Elevation (m)	May 6, 2014	June 4, 2014	June 12, 2014	July 10, 2014	August 12, 2014	September 24, 2014
101	182.81	10.8	172.0	175.46	175.49	175.53	175.46	175.60	175.37

Based on the measured and encountered groundwater levels and the soil colour change from brown to grey, the groundwater level in the cohesive till layer is inferred to be at about elevation 181 metres for design. A groundwater pressure within the sand/sand and gravel layer is equivalent to a water surface at about elevation 175.5 metres indicating that the lower granular deposit, and possibly the underlying bedrock, represents a confined aquifer. The difference in interpreted groundwater levels indicates that there is an overall downward hydraulic gradient at this site. 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. and Henderson Drilling Inc., Ontario Ministry of Environment and Climate Change licensed well contractors. 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 Manager, Mr. David J. Mitchell.


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. 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.


Dirka U. Prout, P.Eng.
Project Engineer




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


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



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

NG/SJB/FJH/cr

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

n:\active\2013\1132-geol\1132-0100\13-1132-0111 dillon-gwp 3093-09-00-hwy 401-40\ph 1000-fdns\vrpts\vr01 hwy 401 & 40 up\1311320111-1000-r01 jul 31 15 (final) fnds part a&b
hwy401&40 up.docx



PART B

FOUNDATION DESIGN REPORT

**HIGHWAY 401/HIGHWAY 40 UNDERPASS REPLACEMENT, SITE 13-238
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
CHATHAM-KENT, ONTARIO
GWP 3093-09-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the Highway 401/Highway 40 underpass replacement. 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.

The existing Highway 401/Highway 40 underpass is a four span concrete T-beam structure constructed in 1961. Based on the original General Layout Drawings dated July 1960, the bridge was designed to have four spans of 14, 25.5, 25.5 and 14 metres, a width of 8.5 metres, and a skew angle of about 35 degrees to the Highway 401 centreline. The abutments were reportedly founded on HP310 x 94 (12 BP 53) piles which were driven to approximate elevation 173.1 metres for both abutments, elevation 174.9 metres for the north abutment wing walls and elevation 174.8 metres for the south abutment wing walls. According to Department of Highways Ontario Drawing No. D-4585-2 dated July 1960, the design load for the piles was 356 kilonewtons. According to Geocres Report No. 40J8-15, the recommended minimum allowable footing pressure (working stress design) for spread footings was 287 kilopascals for footings with a minimum width of 2 to 3 metres founded within the very stiff clayey silt to silty clay till at elevation 181.36 metres. The approach slabs were designed to extend approximately 4.8 metres beyond the abutments where two driven piles provided support to each end of the slab. The pier was to have consisted of four columns supported on a continuous spread/strip footing at about elevation 181.5 metres; the founding soil for the shallow footings was not specified. The bridge currently carries one lane each of Highway 40 northbound and southbound traffic over Highway 401.

Based on the preliminary design information provided by Dillon, it is understood that the replacement underpass will be a two span precast, prestressed concrete girder structure with semi-integral abutments, constructed at approximately the same location and alignment as the existing structure. The bridge is to accommodate a widened highway platform that will carry one lane each of Highway 40 north and southbound traffic as well as the S-W and N-E Ramps traffic. The bridge will have a total width of about 25 metres, two spans of about 44 and 41 metres for a total length of 85 metres, and will be constructed at an approximately 35 degree skew angle from the Highway 401 centreline. Based on the design information provided by Dillon, the abutments are to be supported on conventional shallow foundations at about elevation 182 metres. The central pier is to be constructed at the same location of the existing central pier and will consist of five columns supported on a strip footing at about elevation 182.5 metres. The profile grade of the Highway 40 pavements will be increased by about 2 metres at the abutments. The proposed Highway 40 pavement elevation at the abutments is about 192 metres. The existing embankments and central pier will be widened approximately 13.5 metres to the west and between 2.5 to 4.5 metres to the east.



6.2 Bridge Foundations

The subsurface soil conditions at the site typically consist of surficial topsoil or pavement structure overlying fill materials, underlain by a deposit of clayey silt till over sandy silt till with varying layers of sand, sand and gravel and silt, over bedrock. It is noted that the bedrock elevation was not proved by coring in the boreholes advanced for this investigation. The elevation of Highway 401 at the site is known to be approximately 184.5 metres. The inferred groundwater level at the site is near elevation 181 metres.

Semi-integral or conventional abutments may be founded on conventional shallow foundations, driven steel H-piles, or driven concrete-filled steel tube piles. The central pier may be founded on conventional shallow foundations, drilled shafts (caissons), or any of the other deep foundation alternatives. If an integral abutment design is selected, the abutments may be founded on steel H-piles with lateral loads in the direction of the weak axis, or concrete-filled steel tube piles, provided the additional resistance to lateral loads is adequately considered during structural design. It is understood that supporting the replacement bridge abutments and pier on shallow foundations is the preferred technical alternative from a structural engineering perspective. Recommendations for each of these foundation systems are provided in subsequent report sections below.

A comparison of foundation alternatives is presented in Table I following the text of this report. The relative costs are compared using the most economical foundation option (shallow foundations) as the base cost. The estimated relative costs are meant to provide an order of magnitude comparison amongst the alternatives and are not indicative of actual construction costs.

6.2.1 Shallow Foundations

Conventional spread/strip footings may be constructed on the stiff to very stiff clayey silt till at or below about elevation 181 metres. In areas where competent native soils are not encountered at the design founding elevation, footings may be constructed on properly constructed engineered fill or lowered to bear on the native undisturbed soils.

Geotechnical Axial Resistance

The assessment of the factored geotechnical resistance at Ultimate Limit States (ULS) and the geotechnical reaction at Serviceability Limit States (SLS) was carried out for the abutments considering the following three load cases provided by Dillon:

- Case 1: ULS in-service condition – uniform distribution;
- Case 2: ULS in-service condition – trapezoidal distribution; and
- Case 3: SLS in-service condition – trapezoidal distribution.



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

Case	H _f (kN)	V _f (kN)	Eccentricity from Centreline (mm)	Load Inclination (°)	Factored ULS Uniform Pressure (kPa) ¹	Trapezoidal Toe Pressure (kPa)	Trapezoidal Heel Pressure (kPa)
1	750	1,990	570	22	310	-	-
2	750	1,990	570	-	-	400	65
3	490	1,870	570	-	-	130	310

Note: 1) Where factored, the uniform ULS pressures as provided by Dillon exclude load inclination.

The general bearing capacity equation was used to calculate the factored geotechnical resistances at ULS and the geotechnical reactions at SLS. An effective underside of footing elevation of 181 metres, footing width of 8.5 metres and embedment depth of 2.7 metres (to underside of footing) were assumed for the abutment footings. Effective stresses, or drained conditions, were found to apply to in-service ULS conditions (Cases 1 and 2). Total stresses, or undrained analyses, were used for the SLS condition (Case 3). One-way eccentricity was considered for all load cases and inclination was accounted for where required. The results were checked using a computer-assisted settlement analysis that considered staged construction as described in the following section. The clayey silt till was assigned the following properties for calculation of the geotechnical resistances.

Unit Weight, γ (kN/m ³)	Effective Stress		Total Stress
	Cohesion, c (kPa)	Effective Angle of Friction, φ' (°)	Undrained Shear Strength, s _u (kPa)
21	0	32	160

The recommended factored geotechnical resistances at ULS and geotechnical reaction at SLS (considering 25 millimetres of settlement) are summarized in the table below.

Load Case	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
1	570	-
2	2,000	-
3	-	325

In accordance with Section 6.7.4 of the current edition of the Canadian Highway Bridge Design Code (CHBDC), it is not necessary to apply the load reduction factors to the geotechnical resistances at ULS provided above, since the effects of inclined loads have been accounted for in calculating the above values. The difference



between the factored ULS resistances for the different cases and SLS resistance values is the result of the assumed speed of loading and response of the underlying soils. Case 3, for example, assumes an idealized instantaneous loading on a saturated and plastic clay soil (undrained loading), whereas Cases 1 and 2 assume drainage of pore water pressures has occurred. Actual loading during construction will be between these two simplifying assumptions.

Settlement – Bridge Structure

Settlement of the embankment and bridge footings was evaluated 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 table below summarizes the engineering parameters and simplified stratigraphy used in the analysis.

Material	Elevation (m)	γ (kN/m³)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Silty Clay to Clayey Silt	182 to 183	19	-	-	0.49	0.146	0.016	400	0.47
Clayey Silt to Silty Clay Till	179 to 182	21	-	-	0.49	0.127	0.014	740	0.46
	176 to 179				0.49	0.127	0.014	655	0.46
	173 to 176				0.49	0.127	0.014	735	0.46
Sandy Silt Till	171 to 173	22	100,500	100,500	0.3	-	-	-	-
Sand/Sand and Gravel	Below 171	22	87,000	87,000	0.3	-	-	-	-

The ground surface was taken as elevation 184.0 metres. A reference stage was established in the model whereby any settlements induced by the existing embankment and pier footings were considered to have already occurred and all settlement values discussed further in this memorandum are representative of new settlements.

The first simulated construction stage included construction of the west embankment widening. The second simulated construction and backfilling of footings for the abutments and median pier for the west side of the structure. The third stage included construction and backfilling of the abutment and median pier footings for the east portion of the structure. The fourth and final construction stage simulated increasing the highway grades and widening the east portion of the embankment. Based on preliminary staging considerations, it is understood that the abutment foundations may be constructed in advance of the embankment widening to minimize the need for excavation of newly placed fill. These simplified construction stages did not, however, account for time differences between the various stages or account for construction of embankments for the adjacent S-W and N-E Ramps.

The dead load of the abutment structures and their backfill represents more than 60 to 70 per cent of the load. The majority of any remaining settlement is expected to be complete within 1 to 3 months after completion of fill placement. It is recommended that erection of the superstructure, particularly for the west side, proceed as late



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

as practicable after completion of the substructure and backfilling of the embankment in order to minimize post-construction differential superstructure settlement.

Total and differential settlement should be within acceptable values. Estimates of settlement at the abutments and centre pier have been provided to aid the structural designers in proportioning the foundations and optimizing the design to provide relatively uniform settlement along the continuous span structure. Post-construction settlement values consider placement of the abutment backfill and approach embankment fill above the bridge bearing elevations to the ultimate elevation and addition of the bridge deck.

Stage	Abutment Foundation Settlement (mm)					
	North Abutment			South Abutment		
	West	Middle	East	West	Middle	East
West Side of New Structure and Embankment Widening						
Settlement to completion of backfill to bridge bearing elevation – west side	25	15	<10	30	20	<10
Post-construction settlement after completion of widening and bridge girder and deck installation (Stage 2)	10	<10	<10	15	10	<10
East Side of New Structure and Embankment Widening						
Settlement to completion of backfill to bridge bearing elevation – east side	35	30	15	45	40	15
Post-construction settlement after completion of widening and bridge girder and deck installation (Stage 4)	<10	10	10	<10	<10	10

It is expected that the post-construction settlement in the area of the median pier will be relatively unaffected by the embankment modifications. For this project, settlement at the abutment foundation locations is driven primarily by the embankment loads whereas settlement at the pier is driven largely by the superstructure loads. Because of the sequencing of construction, it is further expected that the abutments and approach embankments will be constructed before the superstructure and deck are in place and, given the nature of the soils at the site, a significant proportion of the settlement at these locations should occur as the load is placed, as discussed above. Therefore, the median pier may settle more than the abutments once the approach embankments have been constructed. It is expected that this differential settlement will be of particular concern if the pier and abutments are supported on different foundation types (i.e. median pier on shallow foundation and abutments on deep foundations). The estimated magnitude of settlement of the median pier is summarized below.



Stage	Pier Foundation Settlement (mm)		
	West	Middle	East
Completion of superstructure (Stage 2)	25	20	<10
Completion of superstructure (Stage 4)	25	25	15

Given that a continuous span structure is planned, it may be necessary to proportion the median pier foundations to promote more uniform settlement magnitudes among all foundation elements if spread footings are to be used for the abutments and pier. The settlement estimates summarized above are based on the full abutment foundations (combined east and west halves) being approximately 8.5 metres wide and 30 metres long, and the median pier foundation being about 4.5 metres wide and 30 metres long. The differential settlement between the abutments and piers, for the western half of the structure, should be on the order of about 15 millimetres in a sag profile from north to south. If this differential settlement magnitude or direction is not acceptable, the median pier width could be increased to reduce the applied stresses on the soils at and below the foundation elevation. For example, if the median pier width is increased to approximately 7 metres, the anticipated pier settlement would be about 20 per cent less than the values tabulated above.

Resistance to Lateral Forces

Resistance to lateral forces/sliding between the cast-in-place concrete spread/strip footings and the native, undisturbed subsoil should be calculated in accordance with Section 6.7.5 of the CHBDC. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following effective angles of internal friction and corresponding unfactored coefficients of friction, $\tan \phi'$, may be used.

Founding Soil	Effective Internal Angle of Friction, ϕ' (°)	Unfactored Coefficient of Friction, $\tan \phi'$
Stiff to very stiff silty clay and clayey silt	28	0.53
Stiff to very stiff clayey silt till	32	0.62

Frost Protection

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



6.2.2 Deep Foundations

If a semi-integral or integral abutment design is selected, consideration may be given to supporting the replacement structure on driven steel H piles or concrete filled steel tube piles. Consideration may also be given to founding the pier or on drilled shafts (caissons). Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement.

Geotechnical Axial Resistance – Driven Piles

Steel HP 310 x 110 piles and 324 millimetre outside diameter (OD) by 9.5 millimetre wall thickness, concrete-filled steel tube piles driven closed ended should be driven to practical refusal. Although the MTO considers practical refusal to be at elevations where three consecutive SPT N values of 100 blows per 0.3 metres have been obtained, this criteria does not take into account the greater efficiency and energy of the modern automatic hammers that were used in this investigation. It is considered that practical refusal at this site may be achieved in the granular layers at or below elevation 168.5 metres at the south abutment to 173.5 metres at the north abutment. For planning, to minimize the potential for having to drive the piles deeper, the maximum depth of refusal may be assumed to be between elevation 164.0 metres at the south abutment and elevation 168.0 metres at the north abutment in the dense to very dense sandy silt till at depth. For design, the factored axial geotechnical resistances at ULS and geotechnical reactions at SLS for H-piles and steel tube piles as described above driven to practical refusal at or below the shallower anticipated elevations are provided in the following table. The SLS values correspond to an estimated total settlement of 25 millimetres.

Pile Type and Location	Assumed Cut-off Elevation (m)	Founding Strata	Maximum Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
HP 310 x 110:	182.3	Dense to very dense sandy silt till, sand and sand and gravel		1,500	1,000
– North Abutment			173.5		
– Central Pier			172.0		
– South Abutment			168.5		
324 x 9.5 mm OD steel tube:				750	500
– North Abutment			173.5		
– Central Pier			172.0		
– South Abutment			168.5		

Alternatively, if the piles are driven to the depths where SPT refusal was obtained, greater resistances may be achieved as summarized in the following table.



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

Pile Type and Location	Assumed Cut-off Elevation (m)	Founding Strata	Maximum Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
HP 310 x 110:	182.3	Very dense sandy silt till		1,800	1,500
– North Abutment			168.0		
– Central Pier			164.0		
– South Abutment			164.0		
324 x 9.5 mm OD steel tube:				900	600
– North Abutment			168.0		
– Central Pier			164.0		
– South Abutment			164.0		

The above cut-off elevations have been assumed based on the proposed shallow footing elevations shown on the preliminary design drawing and an assumed pile cap embedment depth of 0.3 metres. Higher cut-off elevations may be used if an integral abutment design is selected resulting in longer piles.

The pile driving hammer should be selected such that the energy delivered to the piles is sufficient to achieve the termination criteria (ultimate geotechnical resistances and tip elevations) stated in the Contract Documents. The actual pile penetration and pile set characteristics will be dependent, to some extent, upon the driving equipment selected by the contractor. It is recommended that, following the selection of the driving equipment, the piling contractor submit for review to the geotechnical engineer the proposed pile driving criteria based on the characteristics of the hammer and equipment intended for use.

Geotechnical Axial Resistance – Drilled Shafts (Caissons)

The vertical load carrying capacity of caissons derived from side resistance may be calculated using the following equation.

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

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

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

$$Q_b = q_{BN} A_t$$



Where Q_b is the toe resistance in kN, q_{BN} is the nominal unit base resistance in kPa, and A_t is the cross-sectional area of the caisson in square metres. Cast-in-place concrete drilled piers founded in the stiff to very stiff clayey silt till at the elevations shown below may be designed using the nominal unit side and base resistances provided in the following table. The stratigraphy presented in the table below has been simplified for the purposes of this report.

Soil Type	Elevation (m)	f_{SN} (kPa)	q_{BN} (kPa)	Unit Weight (kN/m ³)
Central Pier				
– Fill	Above 181.0	-	-	19.0
– Clayey silt till	173.6 to 181.0	40	1,200	21.0
– Sandy silt till	170.9 to 173.6	125	3,000	21.0

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

Frost Protection

Pile caps should be provided with a minimum of 1.2 metres of soil cover or thermal equivalent above the underside of pile cap elevation for frost protection.

Downdrag Load (Negative Skin Friction)

The embankment foundation soils are lightly to moderately overconsolidated. It is expected that the proposed grade raise of about 2 metres at the abutments and embankment widening will result in negligible downdrag loads on piles.

Resistance to Lateral Loads

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

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

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

where:

$$d = \text{pile width or diameter (m)}$$



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

n_h = constant of horizontal subgrade reaction (MPa/m)

S_u = undrained shear strength of the soil (MPa)

z = depth below ground surface grade (m)

The stratigraphy presented in the table below has been simplified for the purposes of this report. The range in values reflects the variability in subsurface conditions as well as the two extremes of design: the requirement for flexibility if integral abutments are selected, and the requirement for lateral support in the cases of non-integral abutments or pier foundations.

Location	Soil Type	Elevation (m)	n_h (MPa/m)	S_u (MPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 - 10	-
North Abutment	Stiff to very stiff clayey silt till	174.4 to 182.5	-	200
	Very dense sandy silt till	171.0 to 174.4	15 - 20	-
	Dense to very dense sand/sand and gravel	166.1 to 171.0	10 - 20	-
	Very dense sandy silt till	Below 166.1	25 - 30	-
Central Pier	Stiff to very stiff clayey silt/clayey silt till	173.6 to 181.6	-	200
	Very dense sandy silt till	170.5 to 173.6	18 - 19	-
	Very dense sand/sand and gravel	164.4 to 170.5	18 - 20	-
	Very dense sandy silt till	Below 164.4	19 - 30	-
South Abutment	Stiff to very stiff clayey silt/clayey silt till	172.0 to 181.0	-	200
	Compact to very dense sand	164.3 to 172.0	6 - 16	-
	Very dense sandy silt till	Below 164.3	25 - 30	-

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

Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
HP 310 x 110, weak axis bending (for integral abutments)	40	*
HP 310 x 110, strong axis bending (for semi-integral or conventional abutments)	260	200
324 mm OD x 9.5 mm tube (for integral abutments)	55	*



Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
324 mm OD x 9.5 mm tube (for semi-integral or conventional abutments)	275	210
Drilled Shafts	2,500	330

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

The lateral resistances are based on Brom's Method from "Design and Construction of Driven Pile Foundations Workshop Manual – Volume 1, FHWA, Pub. No. FHWA H1 97-013, Revised November 1998" and referenced to Table C6.4 of the Commentary for the CHBDC. The SLS values were checked using LPILE Version 6.0, software for analyzing the lateral loading of piles produced by ENSOFT Inc. Free-head piles were assumed for H-piles and steel tube piles, while fixed-head piles were assumed for drilled shafts. For semi-integral or conventional abutments the load was assumed to be applied at the underside of abutment, or the proposed design founding elevation. For integral abutments the horizontal load was assumed to be applied at a height of 3 metres above the proposed founding elevation. An ultimate compressive strength of 32 megapascals (MPa) was assumed for the concrete-filled steel tube piles and caissons. The SLS values are based on 10 millimetres of deflection at the ground surface.

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

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

6.3 Retaining Walls

It is anticipated that retaining walls will be required to retain the approach fills behind each abutment. Consideration may be given to designing the retaining walls as Reinforced Soil System (RSS) walls or concrete cantilever or gravity walls.



RSS Walls

Retained soil system walls utilizing pre-cast concrete panels and geogrid or metal strip reinforcement is a geotechnically feasible alternative. RSS walls are proprietary systems which are to be designed by the supplier and constructed in accordance with their specifications. The internal stability of the mechanically-reinforced soil walls should be verified by the RSS supplier/designer. If a RSS design is selected, the geotechnical aspects of the global stability of the detailed retaining wall design should be reviewed prior to construction. An initial assessment of global wall stability was completed assuming a reinforced soil zone of width equal to 0.7 times the wall height and global stability should be acceptable for this underpass with factors of safety exceeding the minimum MTO requirement of 1.5. Depending on the design approach selected, an embedment depth equivalent to the frost depth may not be required for foundations of RSS walls. This wall type can be constructed relatively quickly and inexpensively using small equipment. RSS walls must be designed in accordance with the MTO RSS Design Guidelines (2008). Special Provision (SP) 599S22 and SP 599S23 dated December 2014 should be included in the Contract Documents.

Reinforced Concrete Gravity and Cantilever Walls

Construction of reinforced concrete gravity or cantilever walls is geotechnically feasible. Footings for concrete gravity and cantilever walls must be constructed with a frost cover of 1.2 metres. This may result in a longer foundation construction time compared to RSS walls, particularly if cast-in-place walls are constructed. Concrete gravity walls could consist of pre-cast or cast-in-place elements.

6.3.1 Retaining Wall Foundations

Retained soil system walls may be designed such that the facing blocks are constructed on a levelling pad constructed with Granular A to a minimum thickness of 300 millimetres. Depending on the design selected by the RSS supplier, it may not be necessary to provide 1.2 metres of earth cover or thermal equivalent for frost protection; however, the foundations must have adequate embedment to provide a stable structure. Typically the embedment depth, defined as the distance between the top of the levelling pad and the top of the adjoining finished grade, is a minimum of 500 millimetres.

Concrete gravity and cantilever walls must be provided with 1.2 metres of frost cover or thermal equivalent. Pre-cast elements should be placed on a levelling pad constructed with Granular A to a minimum thickness of 300 millimetres.

Each wall type may be founded in the stiff to very stiff clayey silt till at or below the elevations noted in the following table.

Wall Location	Elevation (m)
Northwest	183.3
Northeast	182.6
Southeast	182.5



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

Wall Location	Elevation (m)
Southwest	180.8

Concrete gravity and cantilever walls and RSS walls founded on the stiff to very stiff clayey silt till may be designed using a factored geotechnical resistance at ULS of 375 kPa and a geotechnical reaction at SLS of 250 kPa. The SLS value corresponds to an estimated total settlement of 25 millimetres.

6.3.2 Resistance to Lateral Forces

The resistance to lateral forces/sliding resistance between the retaining walls and the subgrade soils should be calculated in accordance with Section 6.7.5 of the CHBDC. Each retaining wall shall be checked for overturning. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angles of friction and corresponding unfactored coefficients of friction may be used for the interaction between the base of the wall and the founding soil.

Wall Type	Interaction	Effective Internal Angle of Friction, ϕ' (°)	Coefficient of Internal Friction, $\tan \phi'$	Angle of Interface Friction, δ (°)	Coefficient of Interface Friction, $\tan \delta$
Concrete Gravity or Cantilever Wall	Cast-in-place concrete strip footing on stiff to very stiff clayey silt till	32	0.62	-	-
	Pre-cast concrete footing on Granular A levelling pad	-	-	33	0.65
RSS Block System Wall	Pre-cast concrete block facing units on Granular A levelling pad	-	-	33	0.65

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

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

The replacement bridge is in Seismic Performance Zone (SPZ) 1, based on a CHBDC classification as an "Emergency Route Bridge". Based on the site stratigraphy, the soil profile type is categorized as Type II with a seismic site response coefficient, S , of 1.2 based on the CHBDC criteria. Analysis of bridges in SPZ 1 is not a



requirement of the CHBDC; however, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

6.4.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.⁵ The liquefaction potential is considered to be relatively low based on the soil profile type, age of the deposits, relative density and the historically low regional seismicity; therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

6.5 Lateral Earth Pressures for Design

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

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

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



- For Case a, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM).

Soil unit weight: 20 kN/m³

Coefficients of lateral earth pressure:

At rest, K_o 0.50

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

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

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

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

6.6 Construction Considerations

6.6.1 Shallow Foundations

The founding soils are sensitive to disturbance and loosening due to water seepage and/or ponding and construction equipment or foot traffic when damp to wet. In the event that the footing concrete cannot be placed in the same working day as completion of the excavation, placement of a concrete working slab (100 millimetres thick of 20 MPa concrete) will be required at the base of the excavations for the footing areas. Exposure without protection using the working slabs may result in loosening or softening of the founding soils. Prior to placing the working slab the cleaned excavation bases should be inspected by a Quality Verification Engineer (QVE) qualified in geotechnical engineering. It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with the QVE on site and the working slab or footing concrete be placed immediately after footing inspection.



6.6.2 Deep Foundations

It should be noted that cobbles and boulders are present in the glacial till and native granular soils at the site and may affect pile driving/caisson drilling operations. A non-standard special provision (NSSP) should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile installation. It is anticipated that the abutments for the new structure will be offset from the existing abutment locations in order to accommodate the lengthening of the bridge span. It is possible that the new abutments may be placed in the vicinity of the existing deck columns and, therefore, may encounter the associated existing piles. Also, if piles are to be driven through the existing embankment fill near the present abutments, they may encounter remnants of temporary works buried in the fill.

Deep foundations should be installed and monitored in accordance with OPSS 903, as well as OPSD 3000.150, 3001.150, and SS103-11 (Pile Driving Control) for the driven piles. The H-piles and steel tube piles should be equipped with Type I driving shoes as shown in OPSD 3000.100 and 3001.100, respectively. In accordance with Section 3.3.3 of the MTO Structural Manual (April 2011) Pile Note No. 2 should be included in the project drawings, indicating the piles are to be driven to below the elevations indicated in Section 6.3.2, above, using twice the provided Factored Geotechnical Resistances at ULS.

6.7 Embankments

All surficial topsoil, organic, loose, soft, and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. Prior to placement of embankment fill material, the exposed subgrade should be proofrolled under the direction of a geotechnical QVE. During this work, particular attention should be given for potentially encountering soft or unsuitable areas in the vicinity of the former Lucas Drain (approximately 110 metres south of the south abutment) and a former meander channel of McGregor Creek (approximately 50 metres north of the north abutment). Design and construction considerations for the high fills in these areas are addressed in a separate report (Geocres No. 40J8-61).

In order to reduce the potential for buildup of excess pore water pressure in the embankments, the use of clayey fill material should be avoided. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 metres where pavement base and subbase materials will be placed. Granular embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and 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 two horizontal to one vertical or flatter. Embankments no steeper than 2H:1V constructed on the near surface stiff to very stiff clayey silt till are considered to be stable and are expected to achieve a Factor of Safety against a deep seated, rotational slope failure of at least 1.3 for embankments. Embankments greater than 8 metres in height should be constructed with a 2 metre wide bench at mid-height. All grading and embankment construction should be conducted in accordance with OPSS 206 and SP 105S10 (amendment to OPSS 501).



6.7.1 Settlement

Settlement Performance Requirements

The MTO restricts the allowable post-construction settlement of the paved portion of embankment widenings on non-freeways to 75 millimetres. 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. The differential settlement rate of embankment widenings on non-freeways is restricted to a maximum of 100 horizontal to 1 vertical. It is anticipated that the embankments will be raised by approximately 2 metres and the new crests will be approximately 26 metres wide.

Data Interpretation

Estimates of engineering parameters used in the settlement analysis were based on SPT N values obtained in boreholes 101 to 107, and an oedometer test carried out on a sample from borehole 104. In addition to direct estimation from the single oedometer test, the compression index, C_c , undrained shear strength, s_u , and preconsolidation pressures, σ'_p , were interpreted using correlations developed for Geocres Report No. 40J6-28 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:

$$\begin{aligned} s_{u(\text{SPT})} &= 9 \text{ to } 10 \text{ times } N_{\text{field}} \\ \text{where: } s_{u(\text{SPT})} &= \text{undrained shear strength as derived from the SPT (kPa)} \\ N_{\text{field}} &= \text{field SPT N value using automatic hammer} \end{aligned}$$

This correlation between the SPT values is an approximation due to the inherent variability of the energy delivered during the SPT procedure; however, the approximation 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. Where available, however, field vane shear tests were considered to better represent shear strength measurements.

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

$$\begin{aligned} C_c &= 0.0086 w_n - 0.0086 \\ C_r &= 0.11 C_c \\ w_n &= \text{natural water content expressed as a per cent} \end{aligned}$$



The preconsolidation pressure was estimated using the undrained shear strength values based on cone penetration test (CPT) data obtained at other structure locations within the interchange. The following well-known relationship was used to estimate the preconsolidation pressure based on undrained shear strength measurements.

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

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 for the Highway 401/Highway 40 interchange site were consistent with the correlation between water content and C_c identified in Geocres No. 40J6-28. 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, the cohesive soils at the site would exhibit recompression behaviour and that the preconsolidation pressure would have little if any effect on the magnitude of settlement.

It should be noted that at the site of the Lucas Drain bridge, Geocres Report No. 40J8-64, located approximately 240 metres south of the underpass, a distinctly different layer of firm to stiff silty clay was identified at and below approximately elevation 167 metres. The northern extent of this deposit is not known. It is expected that the new embankment fill materials combined with the existing in situ stresses in are close to exceeding the estimated preconsolidation pressure in the Lucas Drain bridge area.

Settlement Analysis

Settlement of the founding soils due to the proposed embankment loading was analysed using Settle^{3D} Version 2.0. The existing and future embankment loads were inferred from the drawings provided by Dillon and it was assumed that the new embankment widening and grade raise will consist of conventional engineered fill, with a unit weight between 21 and 22 kilonewtons per cubic metre (kN/m³). The engineering parameters and simplified stratigraphy provided in Section 6.2.1 were used in the embankment settlement analysis.

The total settlement due to the new embankment widening and grade raise within 75 metres of the bridge abutments is estimated to vary as follows:

- from about 100 millimetres at the south abutment decreasing southward to about 30 millimetres at a point 75 metres south of the abutment, depending on the northward extent of the compressible silty clay layer found below elevation 167 metres at the Lucas Drain bridge; and
- from about 80 millimetres at the north abutment decreasing northward to about 25 millimetres at a point 75 metres north of the abutment.

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



Total estimated settlements within 20 metres of the north and south abutments exceed the MTO's post-construction settlement criteria for longitudinal transition zones when staging and construction duration are ignored. The differential settlement rate is estimated to be on the order of 1 vertical to 1,500 horizontal and would therefore be within the MTO differential settlement criteria. As discussed above, most of the settlement for the widened and raised embankments should occur during construction as the dead load is applied and proportional to the dead load. Therefore, once the embankments have been fully constructed and after final paving, differential longitudinal settlement should be within the MTO tolerances.

Though the softer cohesive deposit encountered at depth at the Lucas Drain bridge was not encountered in the boreholes advanced for the underpass structure, the northern extent of the deposit is not known and the transition between this and the granular deposits beneath the Highway 401/Highway 40 underpass structure may extend to within the footprint of the southern approach embankment. This may result in higher than anticipated total settlement and differential settlement rates of the south approach embankment. At the location of the planned highest grade raise between the Highway 401/Highway 40 underpass and the Highway 40/Lucas Drain structure, the maximum estimated total settlement of the embankment would be about 120 millimetres if the clay layer encountered below elevation 167 metres remained at its maximum estimated compressibility and maximum encountered thickness. This condition would result in a differential settlement slope of less than 1/1500 metres and, therefore, should be within the MTO criteria for non-freeway conditions.

6.7.2 Settlement Mitigation

It is expected that settlement mitigation may only be required within about 20 metres of the proposed abutments where the settlement estimates exceed the MTO tolerances and only if staging does not permit a period of settlement prior to erecting the deck superstructure. Consideration may be given to constructing the embankment widening and grade raise and backfilling the abutments with lightweight fill consisting of:

- water-cooled blast-furnace slag;
- cellular concrete; or
- expanded polystyrene blocks.

If settlement mitigation is deemed necessary for the approach embankment widening between the Highway 401/Highway 40 underpass and the Lucas Drain bridge, consideration may be given to the following general options:

- constructing the proposed embankment widening well in advance of the bridge abutments if the project schedule and staging allows;
- use of sand drains to accelerate the rate of settlement; and
- constructing the embankment widening and grade raises using lightweight fill, as described above.



While each of these settlement mitigation options is described below, it is anticipated that the transition between the clay deposit found below elevation 167 metres near the Highway 40/Lucas Drain structure and the granular deposits near the Highway 401/Highway 40 underpass structure should be relatively gradual. Therefore, total and differential settlements should also be gradual and may be within the MTOs criteria. During construction, it will be essential to monitor settlement of the new embankments so that settlement mitigation measures need only be implemented if and as necessary.

Early Embankment Construction/Preloading

Construction of the widened and higher embankments in advance of bridge construction will assist in minimizing the effects of settlement such as negative skin friction (down drag) loads on the piles and differential settlement between the bridge abutments and approach embankments. 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 underpass. In order to minimize or eliminate differential movement between the new structure and embankments solely with early embankment construction it would be necessary to build the embankments to near their full height and use vertical drains through the underlying clayey silt till. However, depending on the planned staging and traffic considerations (e.g., site distances) this approach may not be possible. Therefore, in the vicinity of the replacement underpass the embankment should be widened to its future full width and filled as high as possible in the construction year preceding underpass construction if at all possible. At the time the underpass is constructed, consideration may be given to removal of the embankment materials and reinstatement along with lightweight fill, if necessary, as described below.

Vertical Drains

The use of prefabricated vertical drains (wick drains) at the site was considered as a method to accelerate settlement; however, installation of vertical drains through the stiff to very stiff glacial till soils at the site may be problematic and, given the small site, may not be practical, especially given the unknown northerly extent of the clay layer that exists below elevation 167 metres near the Highway 40/Lucas Drain structure. Generally, installation of wick drains in cohesive soils with N values in the range of 10 to 15 blows per 0.3 metres is expected to be difficult. Further, layers of granular soils, cobbles and/or boulders which are anticipated at the site may further obstruct drain installation. As such, the use of prefabricated vertical drains at the site is not considered appropriate.

While use of pushed-in wick drains may not be practical at this site, sand drains could be used instead. Sand drains, which are the precursor to wick drains prior to their invention, are generally constructed using conventional borehole drilling techniques and equipment where the boreholes are simply filled with sand. In general, sand drains are more costly on a per metre basis but for small sites the overall cost can be less as compared to wick drains. In this particular case, if there is a compelling case for early limitation of settlement effects (e.g., downdrag loads on piles cannot be accommodated or roadway settlement cannot be otherwise managed), sand drains should be extended to approximately elevation 175 metres and be constructed on a uniform triangular grid spacing of 1.5 to 2 metres. Care will be needed to ensure the drill holes are fully filled with water during drilling to minimize the effects of puncturing into the underlying granular layers. If acceleration



of settlement is considered necessary, however, additional exploration should be carried out to better define the northerly extent of the compressible clay layer. A combination of early embankment widening followed by placement of lightweight fill (as described below) should, however, eliminate the need for use of sand drains.

Lightweight Fill

An alternative for reducing the magnitude of long-term settlement is to use lightweight fill for embankment construction and abutment backfill. 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.

The disadvantages of this option are:

- embankments should be constructed with 2 horizontal to 1 vertical or flatter side slopes given the need for granular fill for a levelling pad and conventional soil cover on side slopes;
- significant additional expense of lightweight fill (depending on the type and volume required);
- it is not feasible to install lightweight fill below surface or groundwater levels (due to buoyancy forces); and
- low-height embankments may not practically be built with lightweight fill due to the need for a minimum conventional soil cover or pavement structure on top of the lightweight fill.

The overall goal of using lightweight fill for the underpass site is to minimize the new embankment net loads on the underlying soil to the degree practicable. 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 kN/m³, though EPS meeting the minimum compressive strength criteria for roadway applications is typically about 0.2 kN/m³;
- 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/m³ 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/m³ ("ultralightweight blast furnace slag") to about 14.5 kN/m³ or less.

Non-standard special provisions for each of these materials are included in Appendix D of this report that include relevant definitions, qualifications, material and placement specifications and quality control testing requirements.



The thickness of lightweight fill should be determined based on balancing:

- the total vertical stress induced by the existing fill and pavement thickness above the native soil interface elevation, using an assumed unit weight of about 21 kN/m^3 ; and
- as compared to the total vertical stress of the combined thicknesses of new, controlled granular fill and pavement structure with an assumed unit weight of 22 kN/m^3 and one of the lightweight fill materials defined above.

Beyond the 20 metre abutment distance limits described above, the thickness of lightweight fill should be uniformly tapered at a maximum average slope of 3 horizontal to 1 vertical to the point at which the minimum 1 metre combined pavement and granular fill cover is achieved.

For the EPS lightweight fill option, a levelling pad comprised of at least 300 millimetres of Granular A should be constructed prior to the installation of the EPS. Further, a minimum 125 millimetre thick reinforced concrete pad should also be constructed on top of the EPS prior to placement of the pavement structure to avoid reflective cracking associated with the joints between EPS blocks. All lightweight fill should be covered with a 1 metre thick conventional soil cover on the side slopes.

6.7.3 Instrumentation and Monitoring

For areas where the existing embankments will be widened (preloading) the magnitude and time-rate of settlement should be measured. Such monitoring would consist of installing settlement plates (SPs) below the embankment at the native soil/engineered fill interface. Regular survey measurements of the settlement plates should be taken at given intervals of time during and after construction of the embankment for the duration of the period between initial fill placement (first construction season) and removal of the first stage of fill. In general, at least four settlement plates should be installed for each of the two approach embankment widening sections (northwest and southwest quadrants). In each of the two quadrants, two settlement plates should be located as close to the existing embankment as possible with the other two below the future embankment crest alignments. These monitoring points should be surveyed twice within the first day of installation, and weekly thereafter for three weeks, and monthly thereafter unless the readings indicate that an alternate schedule is appropriate. Surveying should be sufficiently accurate and precise to be repeatable within 2 millimetres from a stable benchmark located at least 50 metres from the nearest construction area. The contractor should be made responsible for immediately replacing any settlement plates that are damaged. The settlement plates 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.



6.8 Excavations and Temporary Cut Slopes

6.8.1 General

Excavations for pile caps or shallow foundations will penetrate the existing pavement structure, fill and/or topsoil and extend into the underlying clayey silt till, and may encounter existing foundations and/or remnants of temporary structures used during the original construction. The groundwater level is expected to be at about elevation 181 metres and will fluctuate seasonally and due to climatic variations. Excavations for shallow footings and pile caps may extend below the groundwater level. Minimal groundwater seepage from the native cohesive soils is expected. Groundwater control may be achieved by pumping from properly constructed and filtered sumps. Sumps should be maintained outside of the actual pile cap and/or abutment limits. Surface water runoff should be directed away from the excavations at all times. All excavations should be carried out 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 native cohesive till and silty clay and clayey silt would be classified as Type 2 soils and the fill materials would be classified as Type 3 soils.

6.8.2 Temporary Roadway Protection

Where space is restricted and will not permit open cuts, temporary road protection systems will be required to support the sides of the excavation and permit the use of vertical cuts. These systems are to be designed by the Contractor to Performance Level 2 as specified by OPSS 539. The design and limits of the systems are to be determined by the contractor.

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

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter provided that the centre to centre pile spacing is greater than three times the pile socket diameter. The unfactored triangular earth pressure distribution (p in kN/m^2 ; increasing with depth), can be calculated as follows:

$$p = K_a (\gamma H + q)$$

where:

H = the height of the excavation at any point (m)

K_a = active coefficient of earth pressure

γ = soil unit weight (kN/m^3)



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 401/HIGHWAY 40 UNDERPASS, SITE 13-238

q = surcharge for traffic and other loading (kPa)

The support systems may be designed using the following parameters:

Soil Type	Coefficients of Earth Pressure			Angle of Internal Friction (°)	Unit Weight (kN/m ³)
	Active, K_a	At Rest, K_o	Passive, K_p		
Fill	0.33	0.50	3.0	30	19.0
Silty Clay to Clayey Silt	0.36	0.53	2.8	28	19.0
Clayey Silt Till	0.31	0.47	3.3	32	21.0

These parameters are provided to assist with design for the unfactored ultimate resistance and loading conditions and may not result in a temporary support design that adequately controls ground and structure displacements. Achieving adequate displacement control in accordance with the MTO performance criteria may require designs that result in a system that is stiffer than might otherwise be required based on the soil parameters provided in the table above. The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficients should be adjusted accordingly.

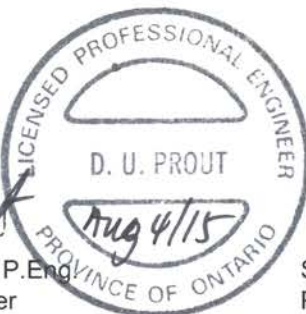


7.0 MISCELLANEOUS

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.


Dirka U. Prout, P.Eng.
Project Engineer





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


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



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

NG/SJB/FJH/cr

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

n:\active\2013\1132-geo\1132-0100\13-1132-0111 dillon-gwp 3093-09-00-hwy 401-40\ph 1000-fdns\reports\101 hwy 401 & 40 up\1311320111-1000-r01 jul 31 15 (final) fnds part a&b hwy401&40 up.docx

TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

Highway 401/Highway 40 Underpass Replacement, Site 13-238
 Highway 401 and Highway 40 Interchange Reconfiguration
GWP 3093-09-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Spread footings supported on stiff to very stiff clayey silt to silty clay till, silty clay or clayey silt	<ul style="list-style-type: none"> • Feasible • Preferred geotechnical alternative 	<ul style="list-style-type: none"> • Least expensive option. • Ease of construction. 	<ul style="list-style-type: none"> • Not compatible with integral abutments. • More settlement expected than with deep foundations. • Larger work area required compared to driven piles and caissons. • Sensitivity of continuous span structures to differential settlement may be of concern. 	<ul style="list-style-type: none"> • Low 	<ul style="list-style-type: none"> • Relatively low risk. • Deeper excavations required if soil at founding elevation is unsuitable.
End bearing steel H-piles or steel tube piles driven to practical refusal in native granular soils	<ul style="list-style-type: none"> • Feasible for abutments 	<ul style="list-style-type: none"> • High bearing resistance. • Negligible settlement. • Compatible with all abutments types; however, steel tube piles may have insufficient flexibility for integral abutment design. • Depending on abutment design, may require less extensive excavations compared to shallow foundations. 	<ul style="list-style-type: none"> • More expensive than shallow foundations. • Can be damaged and deflected by cobbles and boulders within glacial till deposits. • More construction noise and vibration compared to shallow foundations or caissons. • Cannot be visually inspected at depth. • Integrity inspection requires specialty dynamic testing. 	<ul style="list-style-type: none"> • Moderate 	<ul style="list-style-type: none"> • Possible pile tip damage if piles are not adequately protected while driving through till deposits. • Variation in pile tip elevations.

COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Concrete caissons drilled into stiff to very stiff clayey silt to silty clay till	<ul style="list-style-type: none"> • Feasible for pier. 	<ul style="list-style-type: none"> • Less construction noise and vibration compared to driven piles. Faster construction and less work space required compared to shallow foundations. • Less potential for caissons to be impeded by cobbles in native till deposits compared to driven piles. 	<ul style="list-style-type: none"> • Potential for greater settlement compared to driven piles. • Not compatible with integral abutments. • Cannot be visually inspected at depth due to health and safety regulations. 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Cleaning of base could be problematic or overlooked during construction.

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

Prepared By: NG
Checked By: SJB



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 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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



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
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

w	water content
w_l 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 = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$

RECORD OF BOREHOLE No 101

1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693678.4 , E 338414.3 ORIGINATED BY MA/BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE COMPILED BY WDF
DATUM GEODETIC DATE May 5, 2014 - May 6, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _P W W _L WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
182.81	GROUND SURFACE							20 40 60 80 100						
0.00	TOPSOIL, clay, roots Brown													
0.27	CLAYEY SILT, trace sand Stiff Brown and grey													
181.44			1	SS	12		182							
1.37	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Grey		2	SS	14		181							
			3	SS	14		180			10	20	30	2	25 46 27
			4	SS	16									
179.00							179							
3.81	SILTY CLAY TILL, some sand, trace gravel Very stiff Grey		5	SS	17		178							
			6	SS	18									
177.48							177			10	20	30	8	17 41 34
5.33	CLAYEY SILT TILL, some sand to sandy, trace to some gravel Stiff to very stiff Grey		7	SS	15		176							
			8	SS	15		175							
							174			10	20	30	2	20 46 34
			9	SS	14									
173.06							173							
9.75	SILTY CLAY TILL, some sand, trace gravel Very stiff Grey		10	SS	27									
171.99							172							
10.82	SAND, medium to coarse, some gravel, with cobbles Dense to very dense Grey		11	SS	47		171							
							170							
			12	SS	52		169							
							168							
			13	SS	46									

Continued Next Page

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

RECORD OF BOREHOLE No 101

2 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693678.4 , E 338414.3 ORIGINATED BY MA/BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE COMPILED BY WDF
DATUM GEODETIC DATE May 5, 2014 - May 6, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE							
							20	40	60	80	100		10	20	30		GR SA SI CL	
166.96	SAND, medium to coarse, some gravel, with cobbles Dense to very dense Grey						Piezometer										40 26 24 10	
15.85	SAND, fine to medium, trace gravel Very dense Grey		14	SS	50		167											
							166											
			15	SS	55		165											
164.22							Bentonite											
18.59	SANDY SILT TILL, trace to some clay, some gravel, with cobbles Very dense Grey		16	SS	109/ 250mm		164							○				
							163											
			17	SS	104/ 225mm		162											
							161											
160.41			18	SS	100/ 140mm													
22.40	END OF BOREHOLE																	
	Groundwater encountered at about elev. 172.0m during drilling on May 5, 2014.																	
	Water level measured at elev. 175.46m following installation on May 6, 2014.																	
	Water level measured in Piezometer at elev. 175.49m on June 4, 2014.																	
	Water level measured in Piezometer at elev. 175.53m on June 12, 2014.																	
	Water level measured in Piezometer at elev. 175.46m on July 10, 2014.																	
	Water level measured in Piezometer at elev. 175.60m on Aug. 12, 2014.																	
	Water level measured in Piezometer at elev. 175.37m on Sept. 24, 2014.																	

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

RECORD OF BOREHOLE No 102

1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693655.6 , E 338403.6 ORIGINATED BY BT
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	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _P W W _L						
SHEAR STRENGTH kPa								WATER CONTENT (%)							
							○ UNCONFINED + FIELD VANE								
							● QUICK TRIAXIAL × LAB VANE								
189.78	PAVEMENT SURFACE						20 40 60 80 100	10 20 30					kN/m ³	GR SA SI CL	
0.00	ASPHALTIC CONCRETE														
0.24	FILL, granular base, sand and gravel, crushed Brown														
0.45															
0.75	FILL, granular subbase, sand and gravel, some silt Brown		1	SS	7										
	FILL, silty clay, some sand, trace gravel, trace topsoil Firm to stiff Brown and grey		2	SS	6										
			3	SS	9										
			4	SS	12										
			5	SS	9										
			6	SS	5										
			7	SS	8										
			8	SS	8										
			9	SS	15										
182.62	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Brown to grey below about elev. 180.6m		10	SS	13										
7.16			11	SS	16										
			12	SS	17										
			13	SS	13										
			14	SS	12										
			15	SS	12										

Continued Next Page

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

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

RECORD OF BOREHOLE No 103

1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693725.0 , E 338307.2 ORIGINATED BY BT
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	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE								● QUICK TRIAXIAL		
189.72	PAVEMENT SURFACE						20	40	60	80	100						GR	SA	SI	CL
0.00	ASPHALTIC CONCRETE																			
0.24	FILL, granular base, sand and gravel, crushed Brown																			
0.45																				
188.87	FILL, granular subbase, sand and gravel, some silt, with cobbles Brown		1	SS	7															
0.85	FILL, clayey silt, some sand, trace gravel Firm to stiff Brown		2	SS	9															
			3	SS	9															
			4	SS	8															
			5	SS	9															
			6	SS	10															
			7	SS	9															
			8	SS	7															
			9	SS	5															
182.25																				
7.47	SILTY CLAY, sandy, trace gravel Stiff Brown and dark grey		10	SS	10															
			11	SS	9															
180.73																				
8.99	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Brown to grey below about elev. 178.8m		12	SS	21															
			13	SS	17															
			14	SS	16															
			15	SS	10															

Continued Next Page

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

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 103		2 OF 2	METRIC
W.P. <u>3093-09-00</u>		LOCATION <u>N 4693725.0 , E 338307.2</u>		ORIGINATED BY <u>BT</u>	
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>WDF</u>	
DATUM <u>GEODETIC</u>		DATE <u>May 13, 2014</u>		CHECKED BY <u> </u>	

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

RECORD OF BOREHOLE No 104

1 OF 2

METRIC

PROJECT 13-1132-0111

W.P. 3093-09-00

LOCATION N 4693653.3 , E 338372.5

ORIGINATED BY SL

DIST HWY 401

BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE

COMPILED BY WDF

DATUM GEODETIC

DATE May 14, 2014 - May 15, 2014

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100						
								20 40 60 80 100						
													</	

Continued Next Page

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

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 23/12/14

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 104		2 OF 2		METRIC	
W.P. <u>3093-09-00</u>		LOCATION <u>N 4693653.3 , E 338372.5</u>		ORIGINATED BY <u>SL</u>			
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE</u>		COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>		DATE <u>May 14, 2014 - May 15, 2014</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	w _p	w	w _L						
167.11	SAND, medium to coarse, trace to some gravel, trace to some silt Loose to very dense Grey						167														
15.85	SAND, fine to medium, some gravel, trace silt Dense to very dense Grey		15	SS	46																
			16	SS	66																
164.37																					
18.59	SANDY SILT TILL, some clay, trace gravel Very dense Grey		17	SS	100/ 150mm		164														
			18	SS	119																
161.32																					
21.64	SILT, trace sand, trace gravel Very dense Grey						162														
160.41																					
22.55	END OF BOREHOLE Groundwater not established during drilling on May 14, 2014.																				

Continued Next Page

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

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 105		2 OF 2	METRIC
W.P. <u>3093-09-00</u>	LOCATION <u>N 4693680.3 , E 338336.3</u>	ORIGINATED BY <u>SL</u>			
DIST <u> </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE</u>	COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>	DATE <u>May 20, 2014</u>	CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100	20 40 60 80 100	W _p W W _L						
							168										
167.30	SAND, fine to coarse, trace to some gravel, trace to some silt Very dense Grey						167										
15.70			14	SS	62		166										
							165										
			15	SS	60		164										
164.41	SILT Very dense Grey						163										
18.59							162										
163.55	SANDY SILT TILL, some clay, trace gravel Very dense Grey		16	SS	71		161										
19.45							160										
			17	SS	107												
161.36	SILT, some clay, trace sand Very dense Grey																
21.64																	
			18	SS	50/ 50mm												
159.84	SAND, fine to medium, some silt Very dense Grey																
23.16																	
159.07	END OF BOREHOLE Groundwater not established during drilling on May 20, 2014.		19	SS	100/ 150mm												
23.93																	

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 23/12/14

RECORD OF BOREHOLE No 106

1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693720.9 , E 338347.7 ORIGINATED BY SL
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE COMPILED BY WDF
DATUM GEODETIC DATE June 3, 2014 - June 4, 2014 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L	WATER CONTENT (%)	GR SA SI CL		
182.80	GROUND SURFACE													
0.00	TOPSOIL, clayey silt, some sand, roots Dark brown													
0.18	CLAYEY SILT TILL, some sand to sandy, trace to some gravel Very stiff Brown to grey below about elev. 179.1m		1	SS	15									
			2	SS	18									
			3	SS	18									1 21 44 34
			4	SS	19									
			5	SS	20									
			6	SS	18									
			7	SS	16									2 16 46 36
		8	SS	17										
		9	SS	18										
173.81	SANDY SILT TILL, trace to some clay, trace gravel Very dense Grey	10	SS	50									5 45 36 14	
8.99														
		11	SS	59										
171.52	SAND AND GRAVEL, trace to some silt, cobbles and boulders Dense to very dense Grey													
11.28			12	SS	83									
			13	SS	31									29 61 (10)

Continued Next Page

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

PROJECT 13-1132-0111

W.P. 3093-09-00

LOCATION N 4693720.9 , E 338347.7

ORIGINATED BY SL

DIST _____ HWY 401BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE

COMPILED BY WDF

DATUM GEODETIC

DATE June 3, 2014 - June 4, 2014

CHECKED BY _____

[illegible]

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

RECORD OF BOREHOLE No 107

1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693700.3 , E 338298.0 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE COMPILED BY WDF
DATUM GEODETIC DATE June 25, 2014 CHECKED BY

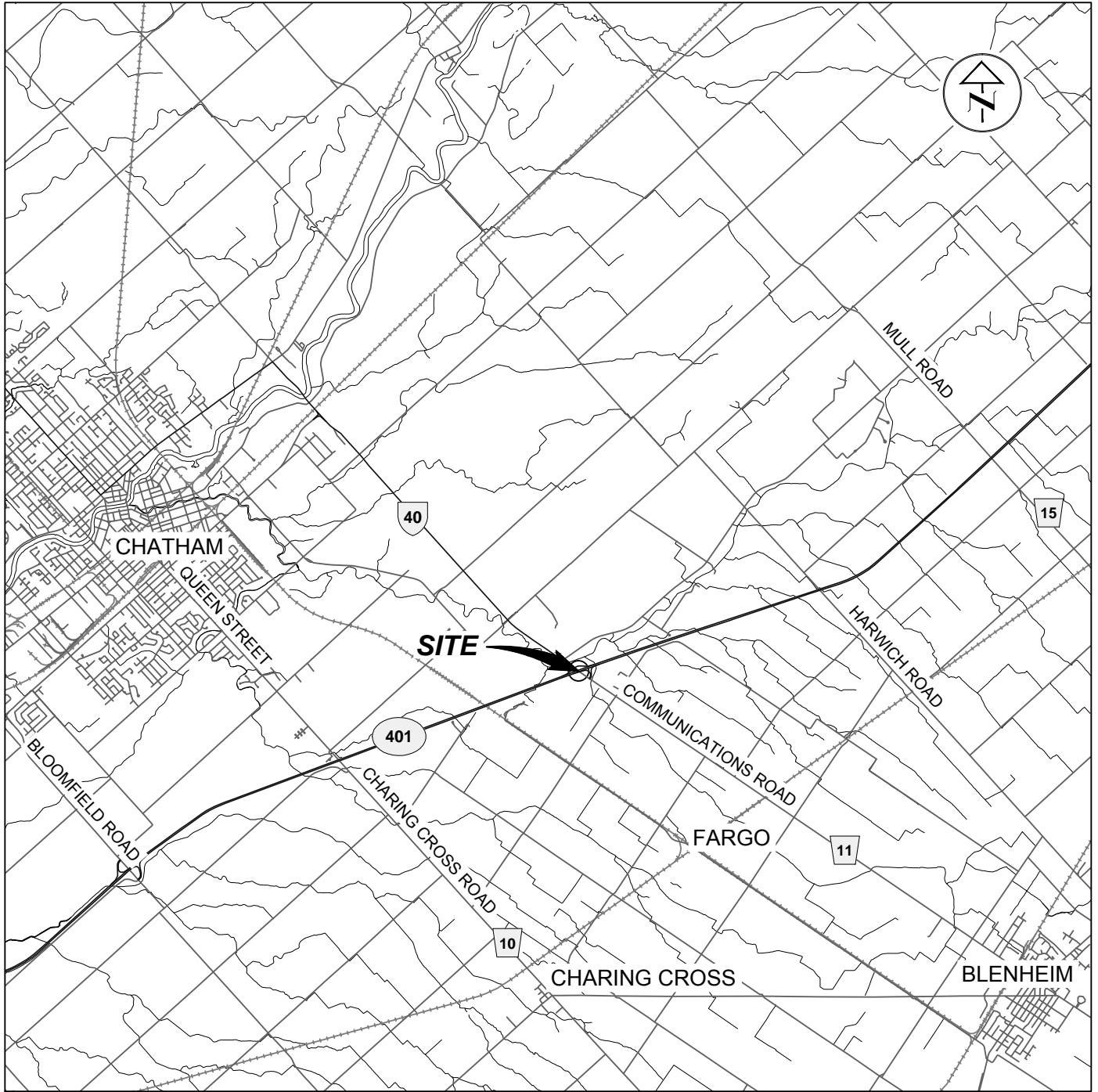
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
183.49	GROUND SURFACE																
0.00	TOPSOIL, silt, roots																
0.18	Brown																
	CLAYEY SILT TILL, some sand, trace gravel																
	Stiff																
	Brown and grey		1	SS	8									0 3 70 27			
			2	SS	10												
181.36																	
2.13	CLAYEY SILT TILL, some sand, trace gravel		3	SS	23												
	Stiff to very stiff		4	SS	16												
	Brown to grey below about elev. 180.6m		5	SS	12												
			6	SS	14												
			7	SS	14												
			8	TO	PH												
			9	SS	16												
			10	SS	8												
173.89																	
9.60	SANDY SILT TILL, trace to some clay, trace gravel		11	SS	87												
	Very dense																
	Grey		12	SS	84									1 32 58 9			
170.69																	
12.80	SAND, medium to coarse, some gravel, trace to some silt		13	SS	68									15 76 (9)			
	Very dense																
	Grey																
169.47																	
14.02	SAND, fine to medium, trace gravel																
	Dense																
	Grey		14	SS	42												

Continued Next Page

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

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 107		2 OF 2	METRIC
W.P. <u>3093-09-00</u>	LOCATION <u>N 4693700.3 , E 338298.0</u>	ORIGINATED BY <u>BT</u>			
DIST <u> </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE</u>	COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>	DATE <u>June 25, 2014</u>	CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _p	W	W _L						
167.95							168														
15.54	SILTY FINE SAND Very dense Grey		15	SS	87		167														
166.42																					
17.07	SANDY SILT TILL, trace clay, trace to some gravel Very dense Grey		16	SS	88		166														
164.74																					
18.75	SAND, fine, trace silt Very dense Grey						165														
163.86			17	SS	100/ 275mm		164														
19.63	END OF BOREHOLE																				
	Groundwater encountered at about elev. 172.5m during drilling on June 25, 2014.																				



0 SCALE 2000 4000m
1:100,000

REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

NOTE

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

PROJECT

HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT
HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

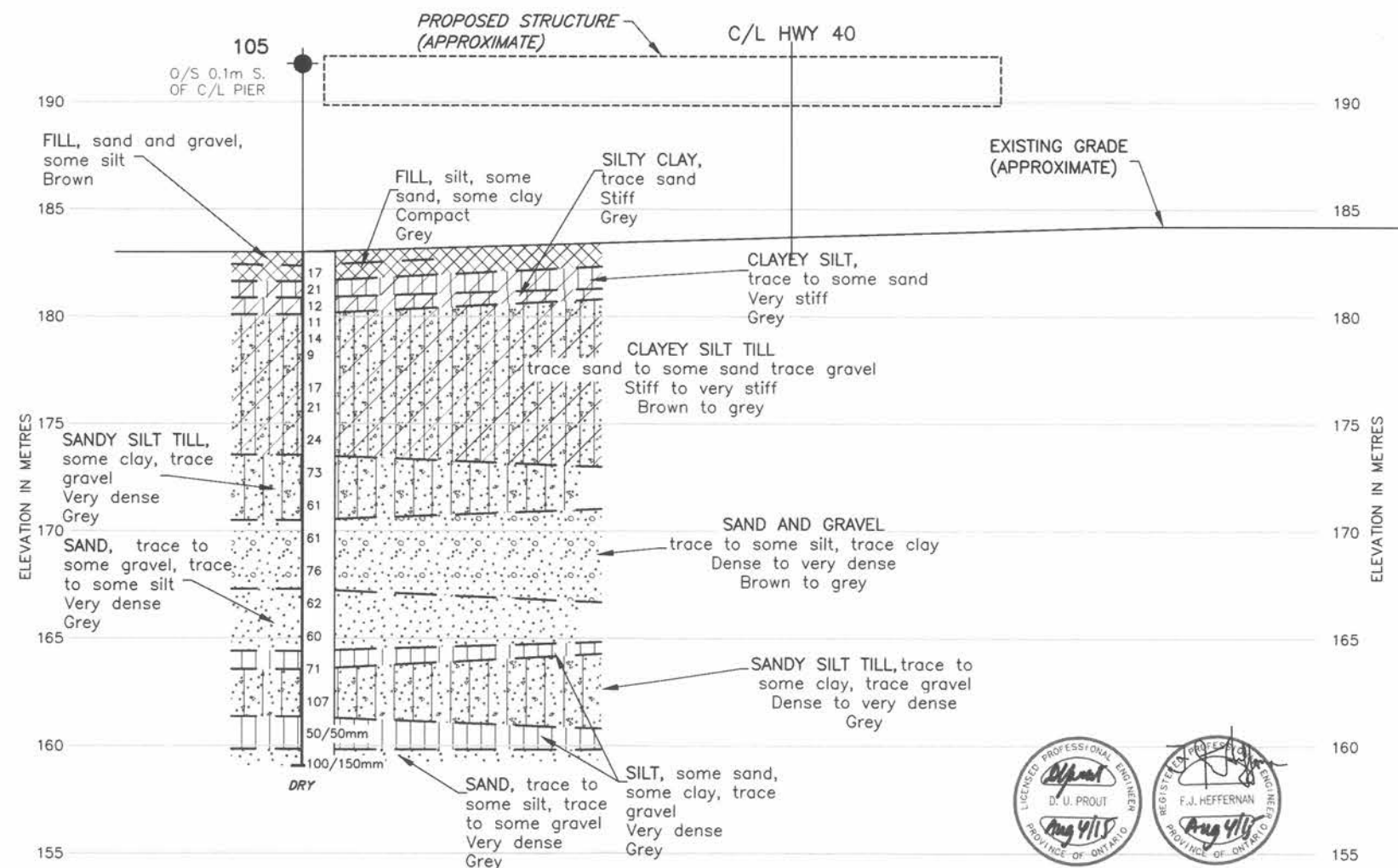
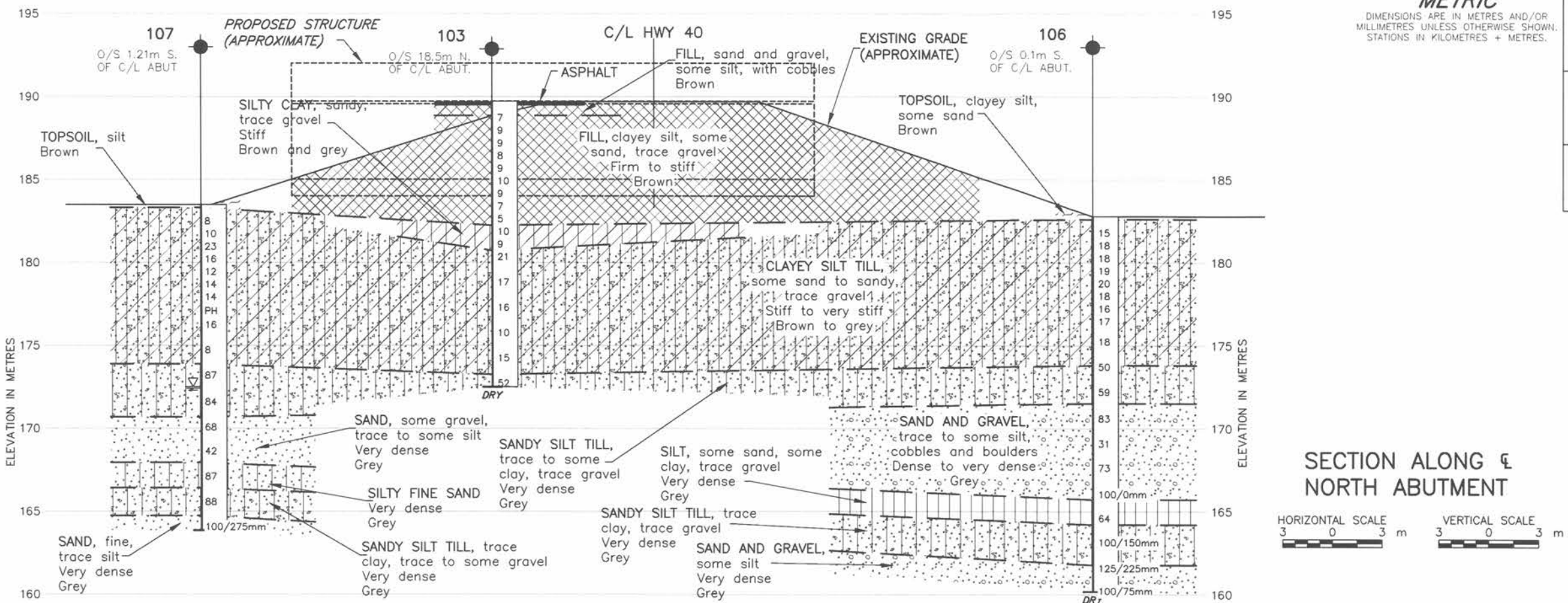
TITLE

KEY PLAN



PROJECT No. 13-1132-0111		FILE No. 1311320111-1000-F01001	
CADD	LMK	July 17/14	SCALE AS SHOWN REV. 0
CHECK			

FIGURE 1



CONT No.
WP No. 3093-09-00

UNDERPASS REPLACEMENT
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN

SCALE IN KILOMETRES

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.
- DRY WL not established during drilling.

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
103	189.72	4 693 725.0	338 307.2
105	183.00	4 693 680.3	338 336.3
106	182.80	4 693 720.9	338 347.7
107	183.49	4 693 700.3	338 298.0

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 McCORMICK RANKIN

NO.	DATE	BY	REVISION
Geocres No.	40J8-59		
HWY.	401	PROJECT NO.	13-1132-0111
SUBM'D.	NG	CHKD.	NG
DRAWN:	WDF	CHKD.	DUP
DATE:	Nov. 24/14	APPD.	FJH
DIST.		SITE:	13-238
DWG.	2		



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

CONT No.
WP No. 3093-09-00

UNDERPASS REPLACEMENT
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole - Current Investigation
- Seal
- Standpipe
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL encountered during drilling.
- WL in piezometer, on Sept. 12, 2014.
- DRY WL not established during drilling.

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
101	182.81	4 693 678.4	338 414.3
102	189.78	4 693 655.6	338 403.6
104	182.96	4 693 653.3	338 372.5

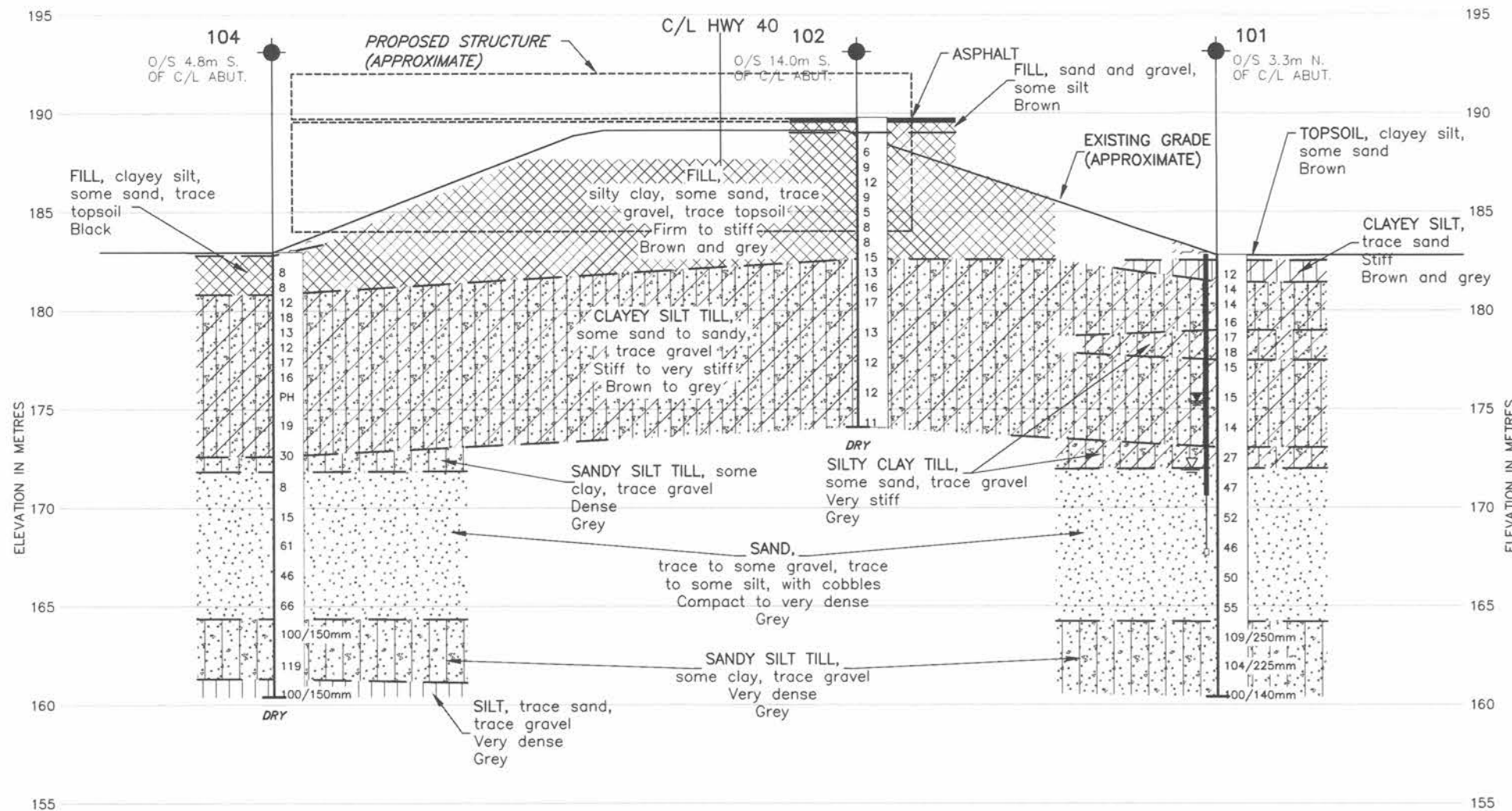
NOTES

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

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

REFERENCE

Base plans provided in digital format by McCORMICK RANKIN



SECTION ALONG & SOUTH ABUTMENT

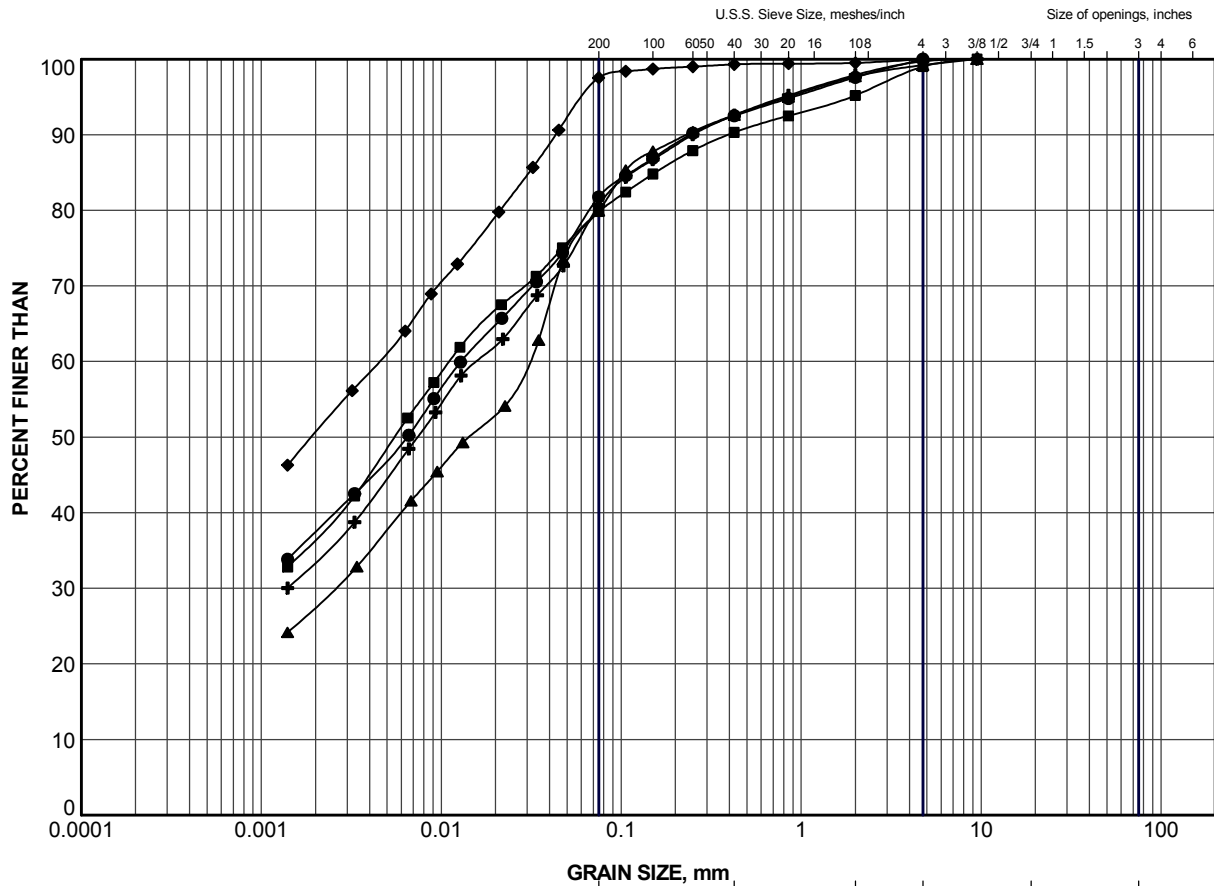


NO.	DATE	BY	REVISION
Geocres No. 40JB-59			
HWY.	401	PROJECT NO. 13-1132-0111	DIST.
SUBM'D.	NG	CHKD. NG	DATE: Nov. 24/14
DRAWN:	WDF	CHKD. DUP	APPD. FJH
		SITE: 13-238	
		DWG. 3	



APPENDIX A

Laboratory Test Data – Routine Soils



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	102	3	187.3
■	102	7	184.2
▲	103	6	184.9
+	103	9	182.6
◆	104	2	181.2

PROJECT
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT
HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

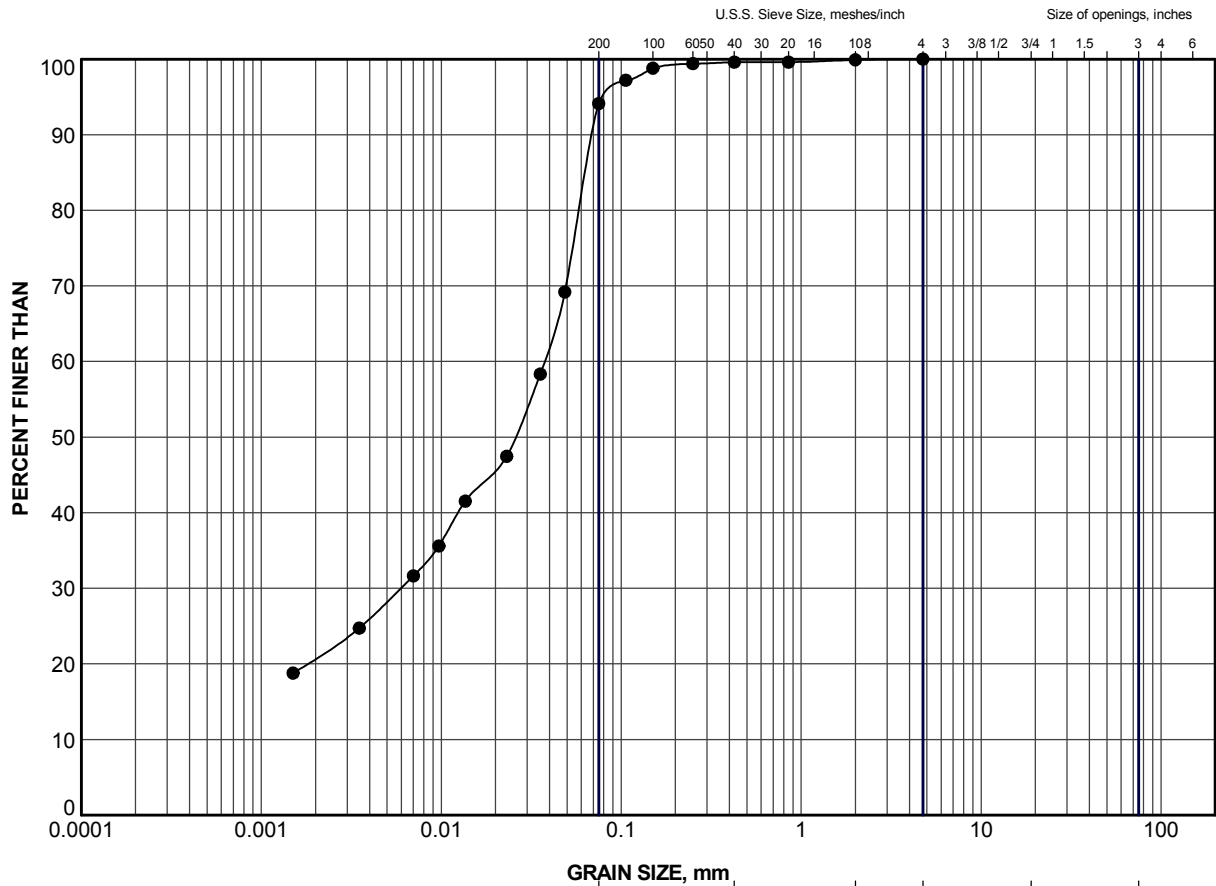
TITLE

GRAIN SIZE DISTRIBUTION FILL



PROJECT No.	13-1132-0111	FILE No.1311320111-1000-R010A1
DRAWN	WDF	Aug 07/14
CHECK		
SCALE N/A REV.		

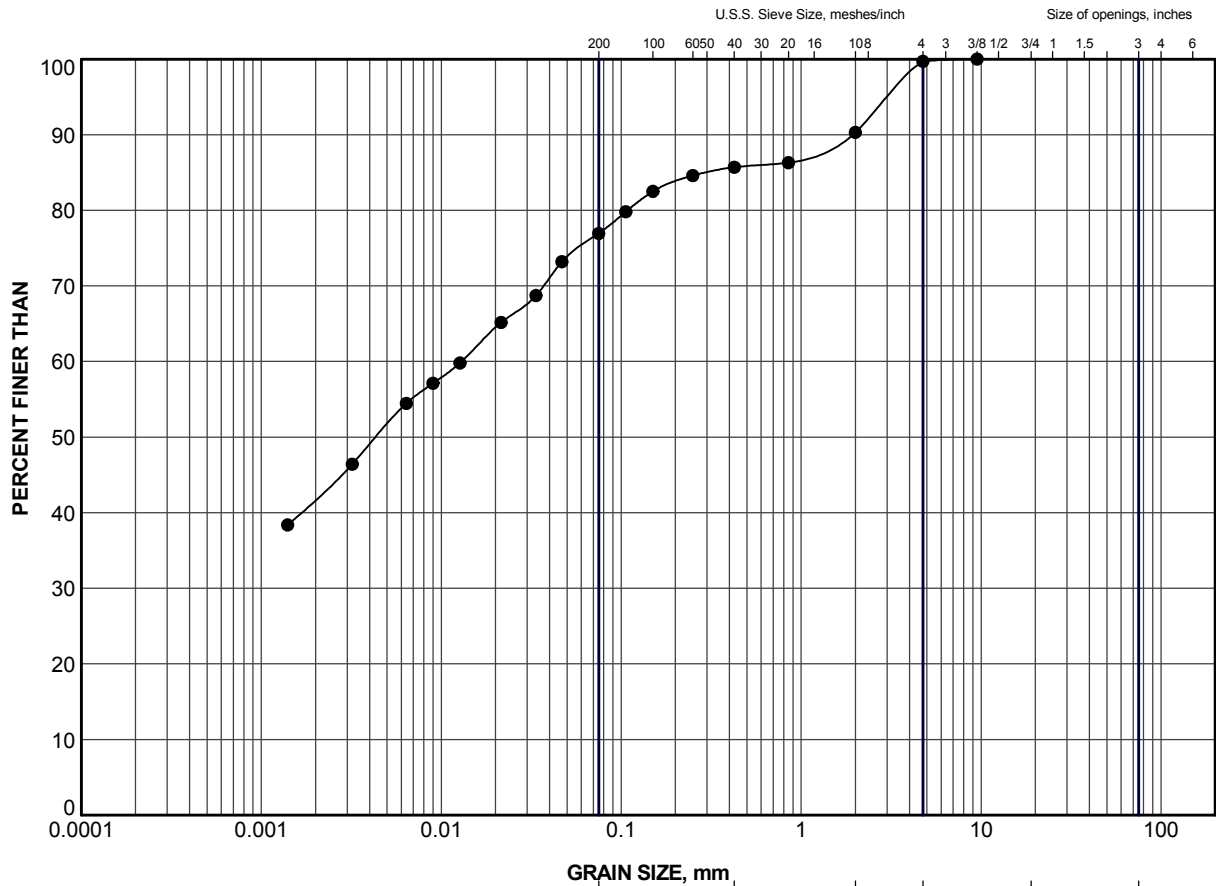
FIGURE A-1



LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	2	181.3

PROJECT			
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		13-1132-0111	
FILE No.		1311320111-1000-R010A2	
DRAWN	WDF	Aug 07/14	SCALE N/A REV.
CHECK			
Golder Associates LONDON, ONTARIO			FIGURE A-2

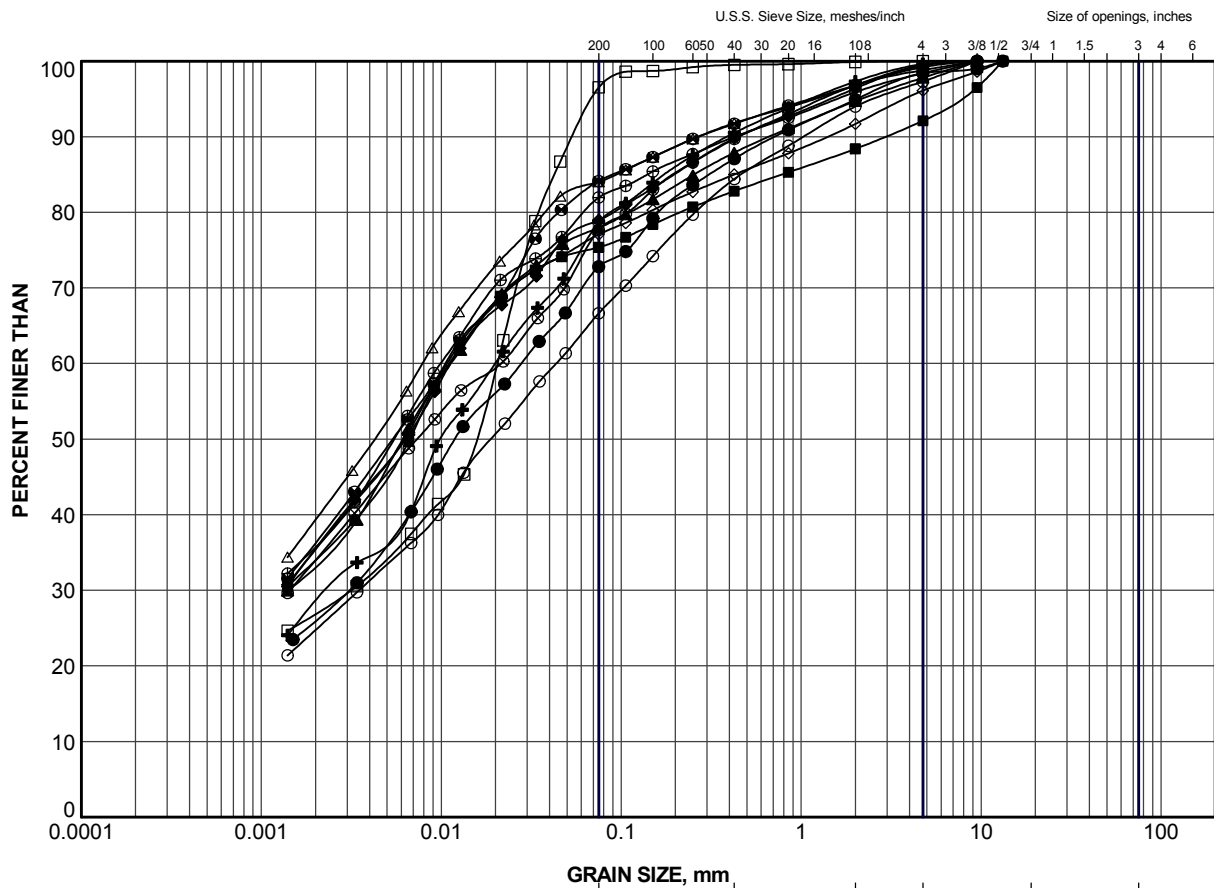


CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	103	11	181.1

PROJECT HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE GRAIN SIZE DISTRIBUTION SILTY CLAY			
	PROJECT No.	13-1132-0111	FILE No.1311320111-1000-R010A3
	DRAWN	WDF	Aug 07/14
	CHECK		
			SCALE N/A REV.
			FIGURE A-3


LDN_MTO_GSD-15 GLDR_LDN.GDT 07/08/14

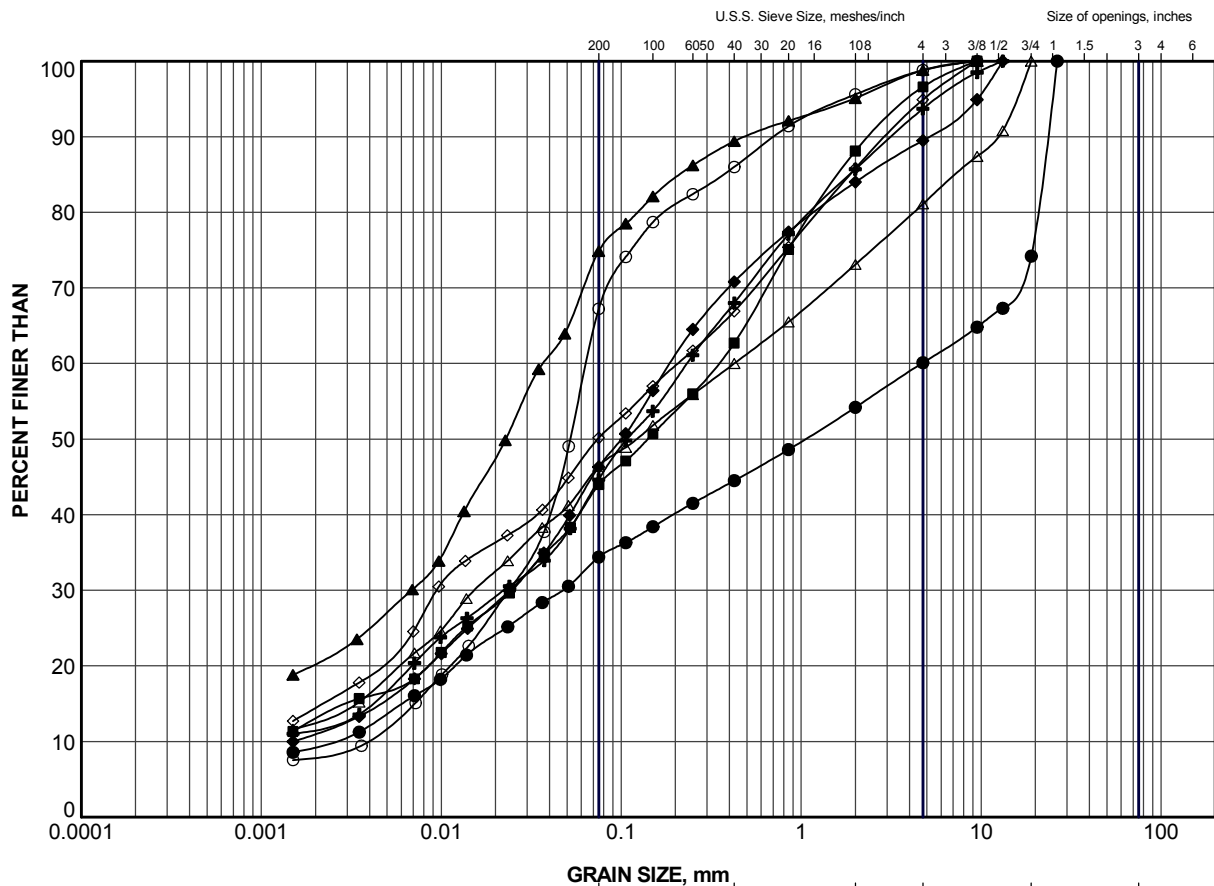


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	3	180.3
■	101	7	177.1
▲	101	9	174.1
+	102	12	180.4
◆	102	15	175.8
◇	103	14	177.3
○	104	4	179.7
△	104	8	176.6
⊗	106	3	180.3
⊕	106	7	177.2
□	107	1	182.5
⊙	107	7	177.9

PROJECT			
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		13-1132-0111	
FILE No.		1311320111-1000-R010A4	
DRAWN	WDF	Aug 07/14	REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-4

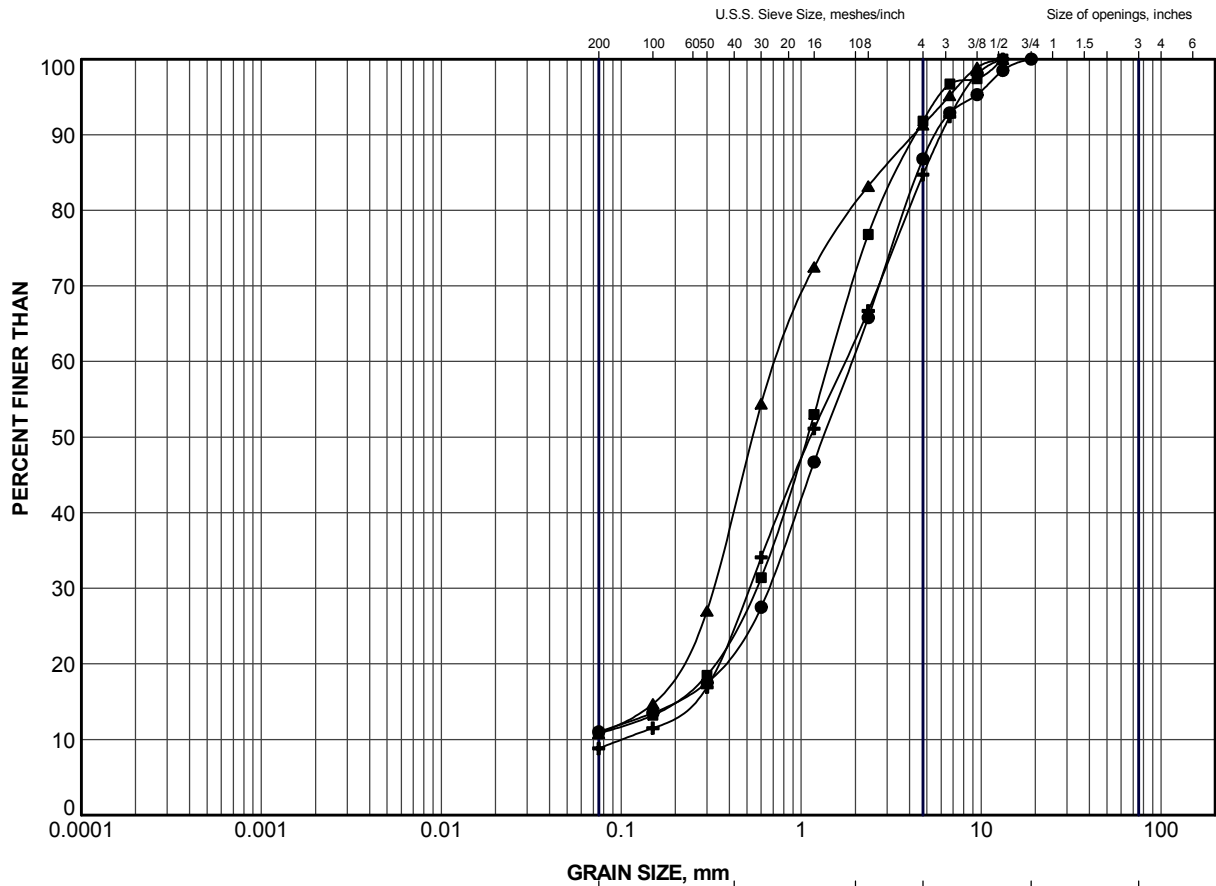


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	16	163.4
■	103	17	172.7
▲	104	18	162.0
+	105	10	172.7
◆	105	17	162.0
◇	106	10	173.7
○	107	12	171.6
△	107	16	165.6

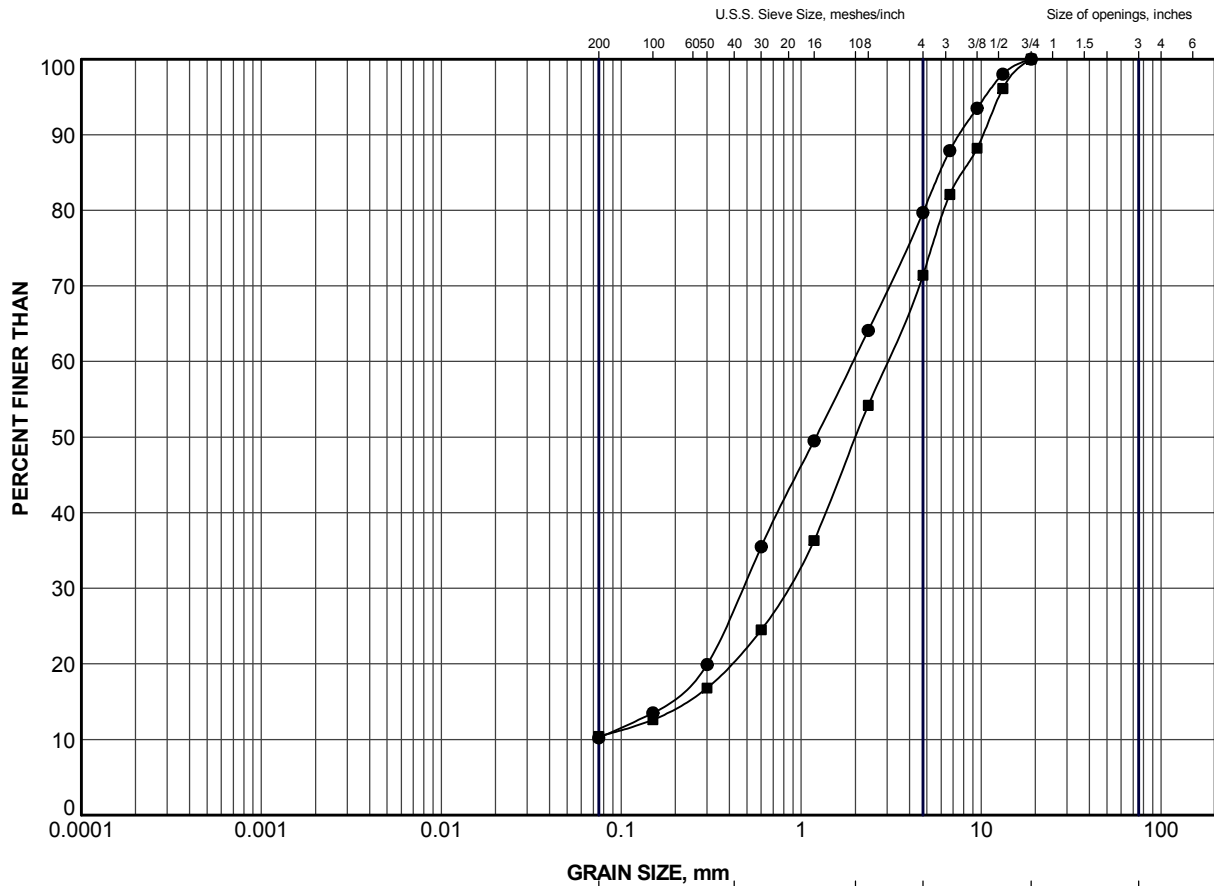
PROJECT			
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No.		13-1132-0111	
FILE No.		1311320111-1000-R010A5	
DRAWN	WDF	Aug 07/14	SCALE N/A REV.
CHECK			
Golder Associates LONDON, ONTARIO		FIGURE A-5	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	101	13	168.0
■	104	12	171.1
▲	105	15	165.1
+	107	13	170.1


PROJECT			
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
GRAIN SIZE DISTRIBUTION SAND			
PROJECT No.		13-1132-0111	
FILE No.		1311320111-1000-R010A6	
DRAWN	WDF	Aug 07/14	SCALE N/A
CHECK			REV.
Golder Associates LONDON, ONTARIO			FIGURE A-6

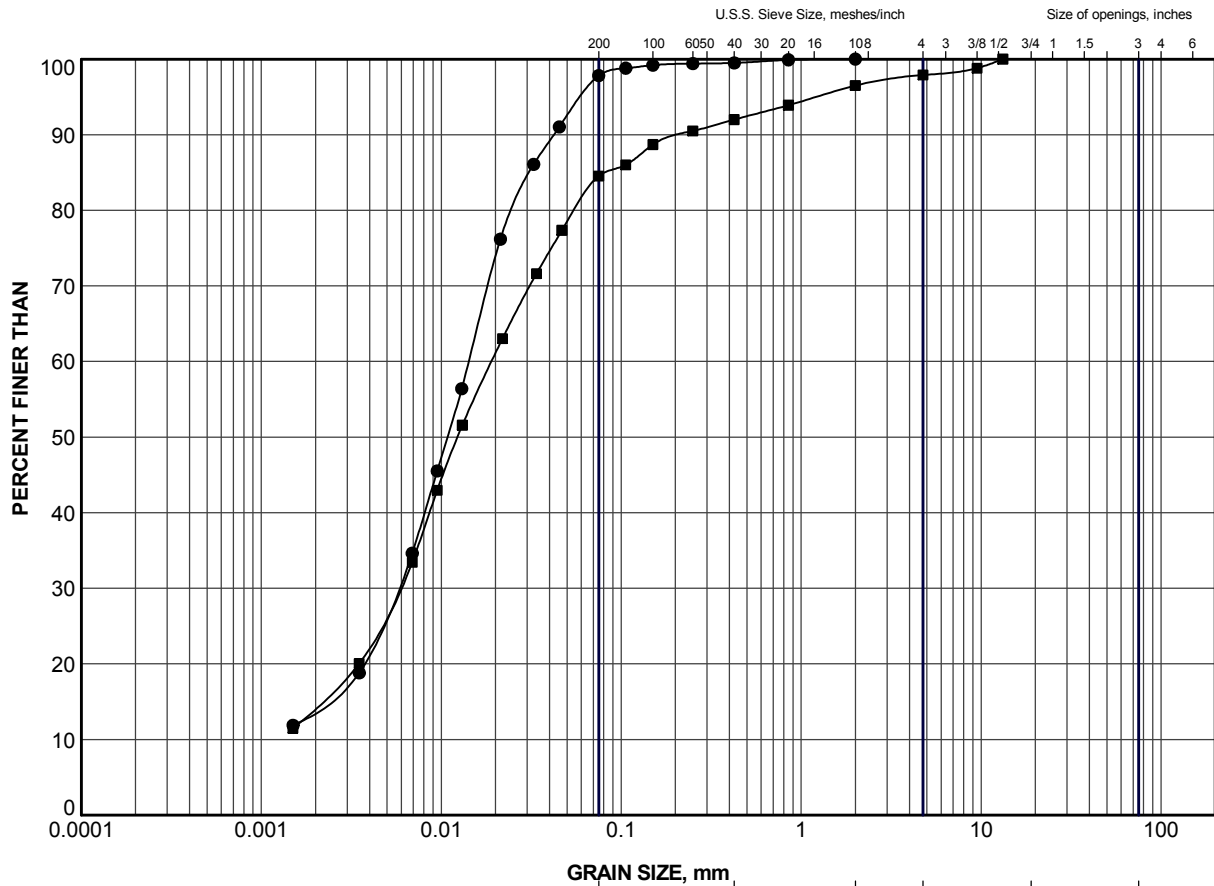


CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	13	168.1
■	106	13	169.1


PROJECT			
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
PROJECT No.		13-1132-0111	
FILE No.		1311320111-1000-R010A7	
DRAWN	WDF	Aug 07/14	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-7

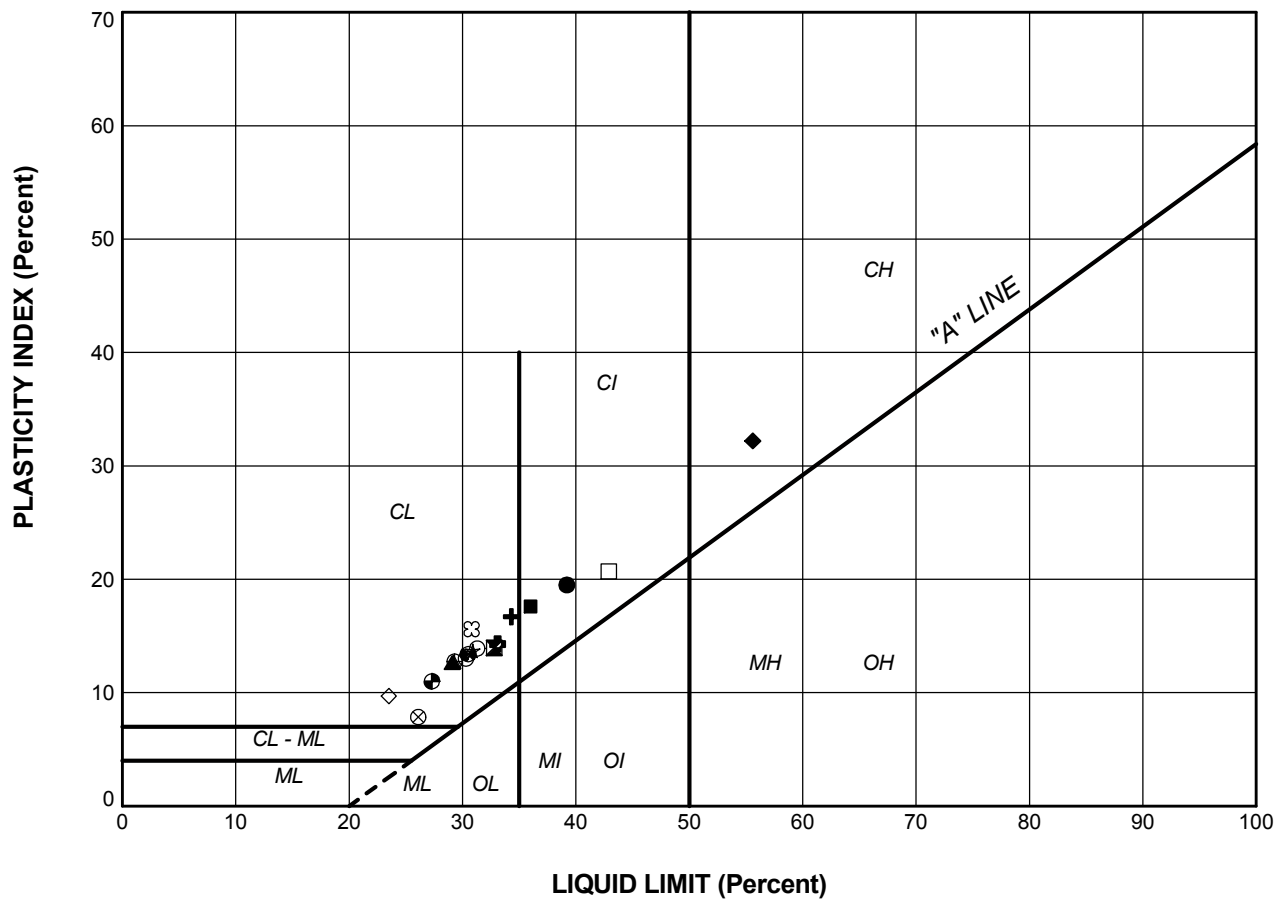


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	105	18	160.6
■	106	16	164.6

PROJECT			
HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
GRAIN SIZE DISTRIBUTION SILT			
PROJECT No.		13-1132-0111	
FILE No.		1311320111-1000-R010A8	
DRAWN	WDF	Aug 07/14	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-8



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI	
●	102	3	39.2	19.7	19.5	(FILL)
■	102	7	36.0	18.4	17.6	(FILL)
▲	103	6	29.1	16.5	12.7	(FILL)
+	103	9	34.3	17.6	16.7	(FILL)
◆	104	2	55.6	23.4	32.2	(FILL)
◇	101	3	23.5	13.8	9.7	
○	101	7	31.3	17.4	13.9	
△	101	9	29.2	16.5	12.7	
⊗	102	12	26.1	18.3	7.9	
⊕	102	15	29.3	16.6	12.8	
□	103	11	42.9	22.2	20.7	
⊙	103	14	30.5	17.1	13.4	
⊛	104	4	27.3	16.3	11.0	
☆	104	8	30.9	17.2	13.7	
⊗	104	9	30.8	15.2	15.6	
⊠	106	3	32.8	18.9	14.0	
⊡	106	7	30.3	17.3	13.0	
⊞	107	7	33.1	18.8	14.3	

PROJECT
 HIGHWAY 401 / HIGHWAY 40 UNDERPASS REPLACEMENT
 HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION
 GWP 3093-09-00

TITLE

PLASTICITY CHART



PROJECT No.	13-1132-0111	FILE No.	1311320111-1000-R010A9
DRAWN	WDF	Aug 07/14	SCALE N/A REV.
CHECK			

FIGURE A-9



APPENDIX B

Laboratory Test Data – Consolidation Testing

CONSOLIDATION TEST SUMMARY**FIGURE B-1A****SAMPLE IDENTIFICATION**

Project Number	13-1132-0111	Sample Number	9
Borehole Number	104	Sample Depth, m	7.0-7.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	5/28/2014		
Date Completed	6/11/2014		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	20.62
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	17.67
Area, cm ²	31.60	Specific Gravity, measured	2.71
Volume, cm ³	80.17	Solids Height, cm	1.687
Water Content, %	16.70	Volume of Solids, cm ³	53.30
Wet Mass, g	168.56	Volume of Voids, cm ³	26.87
Dry Mass, g	144.44	Degree of Saturation, %	89.8

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	2.537	0.504	2.537				
5.89	2.524	0.496	2.531	1	1.36E+00	8.63E-04	1.15E-04
10.74	2.522	0.495	2.523	24	5.62E-02	1.38E-04	7.61E-07
20.51	2.518	0.493	2.520	60	2.24E-02	1.78E-04	3.90E-07
39.96	2.508	0.487	2.513	667	2.01E-03	2.05E-04	4.03E-08
78.75	2.479	0.469	2.493	1185	1.11E-03	2.98E-04	3.24E-08
156.11	2.443	0.448	2.461	1162	1.10E-03	1.83E-04	1.98E-08
311.07	2.404	0.425	2.423	1270	9.80E-04	9.87E-05	9.48E-09
620.93	2.363	0.401	2.384	694	1.74E-03	5.19E-05	8.83E-09
1241.46	2.305	0.366	2.334	290	3.98E-03	3.71E-05	1.45E-08
2480.62	2.242	0.329	2.273	342	3.20E-03	2.01E-05	6.30E-09
1241.46	2.245	0.331	2.243				
311.07	2.278	0.350	2.262				
78.75	2.322	0.377	2.300				
20.51	2.366	0.403	2.344				
5.89	2.409	0.428	2.388				

Note:


Specimen swelled under 10.7kPa.

Specimen taken 0-8 cm from bottom of the tube

k calculated using cv based on t₉₀ values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.41	Unit Weight, kN/m ³	21.68
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	18.61
Area, cm ²	31.60	Specific Gravity, measured	2.71
Volume, cm ³	76.13	Solids Height, cm	1.687
Water Content, %	16.54	Volume of Solids, cm ³	53.30
Wet Mass, g	168.33	Volume of Voids, cm ³	22.83
Dry Mass, g	144.44		

Prepared By: LG

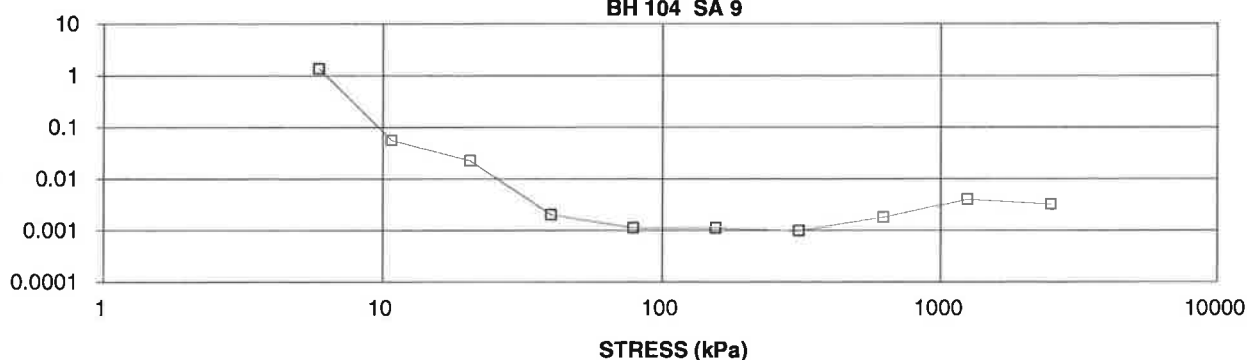
Golder AssociatesChecked By: 

CONSOLIDATION TEST SUMMARY

FIGURE B-1B

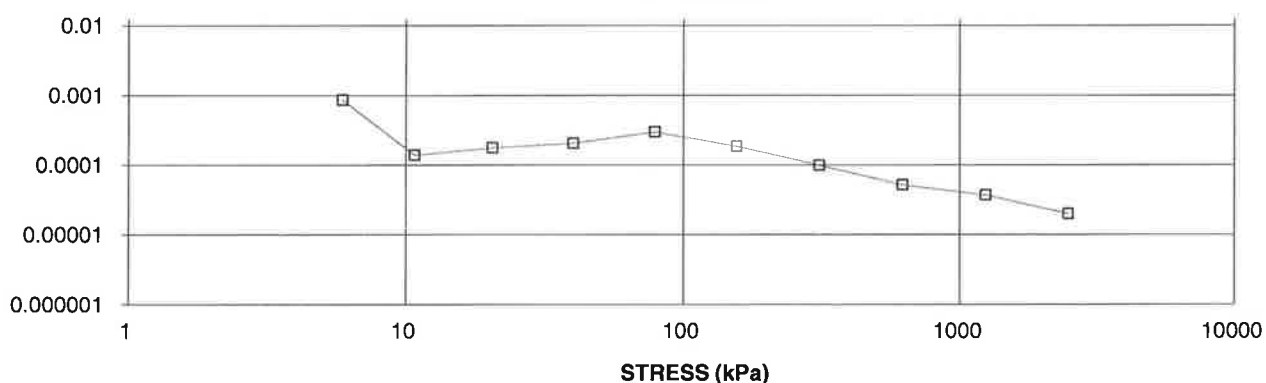
COEFFICIENT OF CONSOLIDATION,
cm²/s

CONSOLIDATION TEST
CV cm²/s VS STRESS (kPa)
BH 104 SA 9



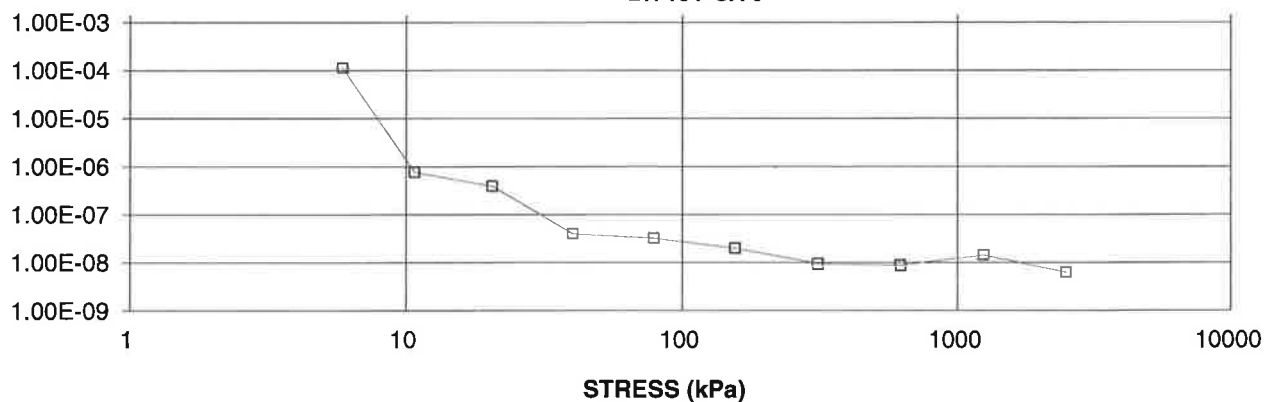
VOLUME COMPRESSIBILITY, m²/kN

CONSOLIDATION TEST
MV m²/kN vs STRESS (kPa)
BH 104 SA 9



HYDRAULIC CONDUCTIVITY,
cm/s

CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 104 SA 9



Project No. 13-1132-0111

Prepared By: LG

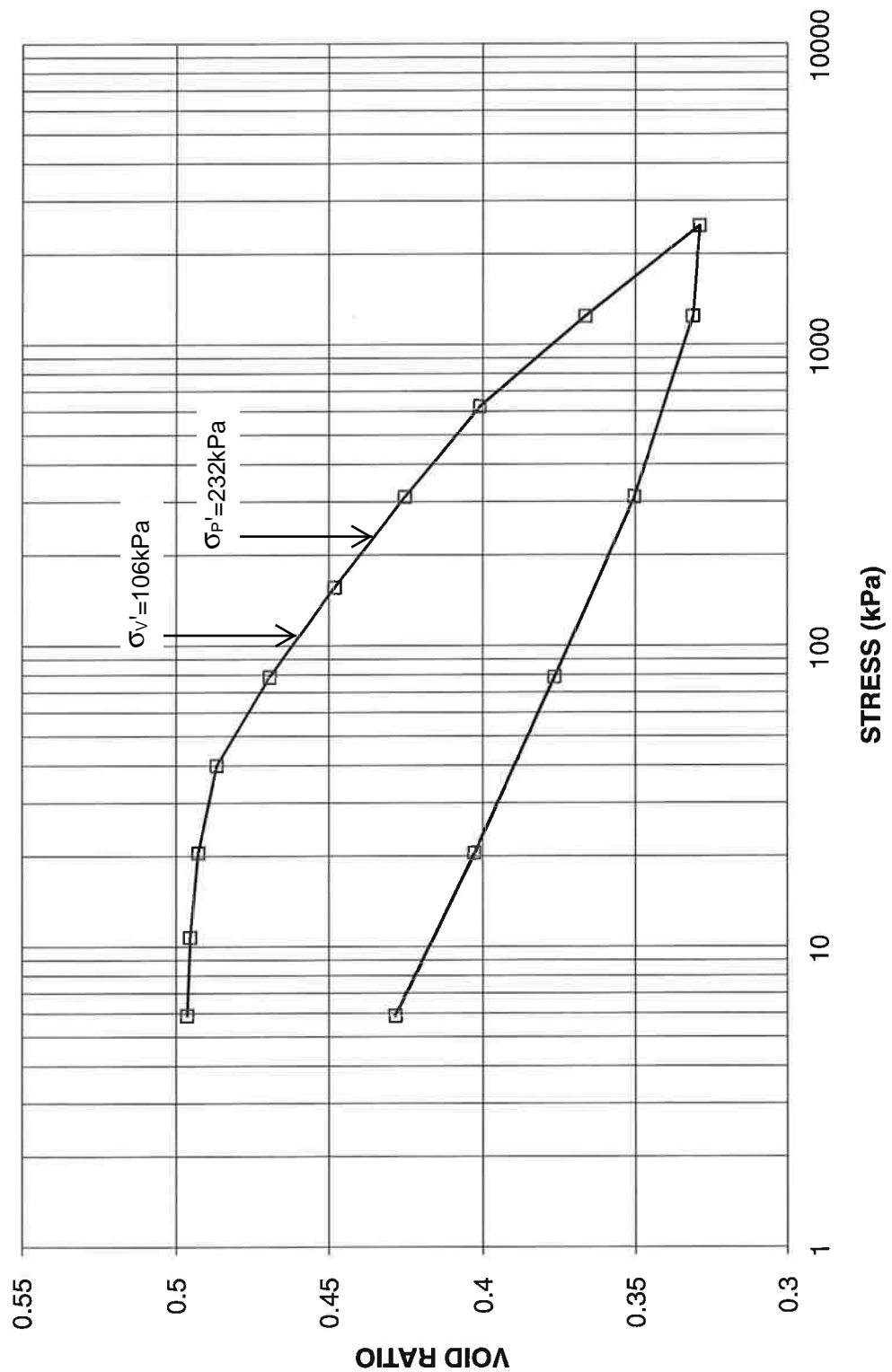
Golder Associates

Checked By: *[Signature]*

CONSOLIDATION TEST VOID RATIO VS LOG STRESS

FIGURE B-1C

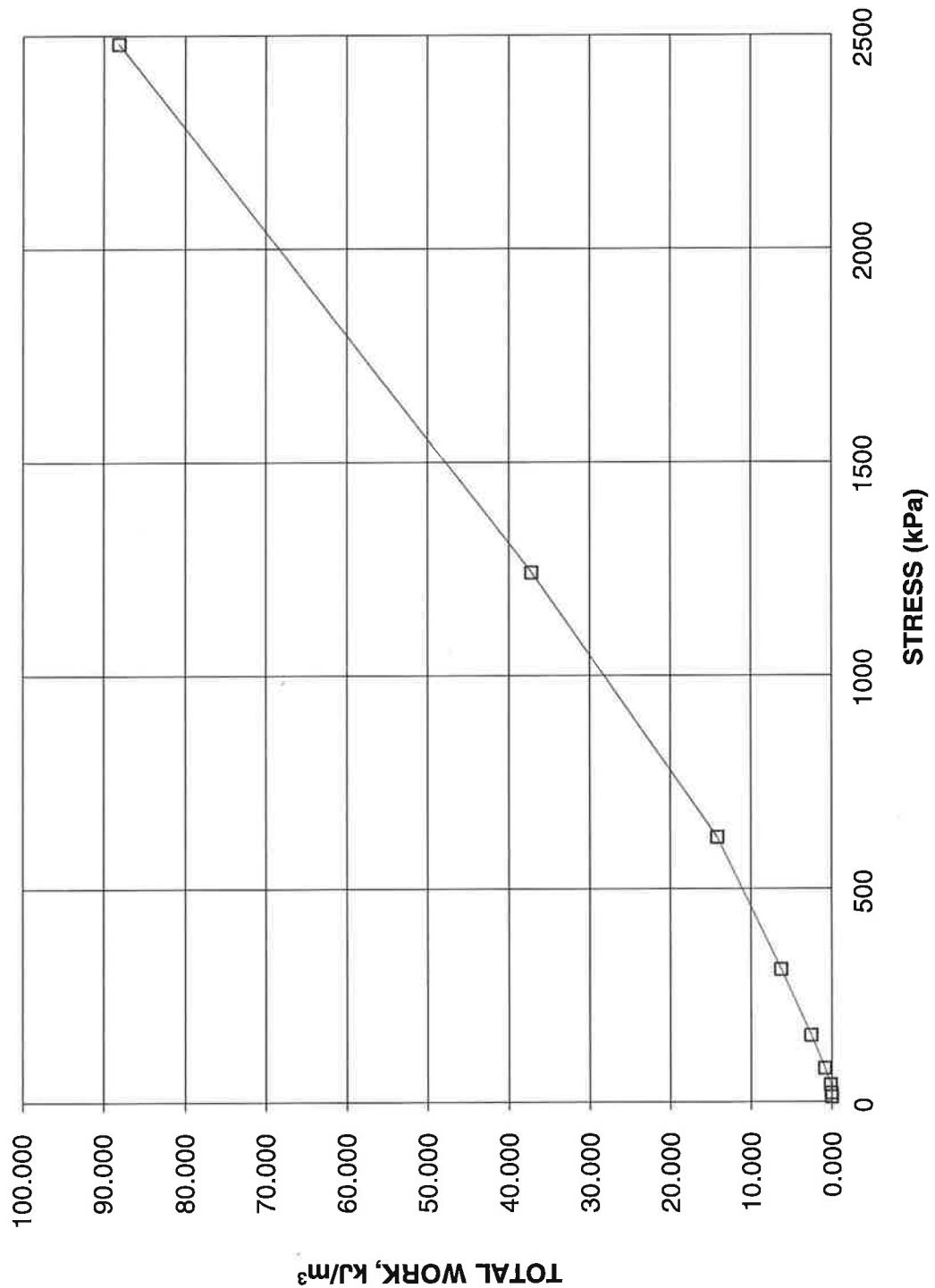
CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 104 SA 9



**CONSOLIDATION TEST
TOTAL WORK VS STRESS**

FIGURE B-1D

**CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH 104 SA 9**



Project No. 13-1132-0111

Prepared By: LG

Golder Associates

Checked By:

[Signature]



APPENDIX C

Site Photographs



APPENDIX C SITE PHOTOGRAPHS



Photograph 1: East elevation of existing underpass, facing west.



Photograph 2: West side of existing underpass, facing south.



APPENDIX C SITE PHOTOGRAPHS



Photograph 3: Highway 40, bridge deck, facing north.

n:\active\2013\1132-geo\1132-0100\13-1132-0111 dillon-gwp 3093-09-00-hwy 401-40\ph 1000-fdns\rpts\r01 hwy 401 & 40 up\1311320111-1000-r01 jul 28 15 (final) app c - site photos.docx



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.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

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.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

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:

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

TABLE 1 – MATERIAL PROPERTIES

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	Mm	1200 x 600 x 200 $\pm 0.5\%$	
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 $\pm 0.5\%$.

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 than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

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.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

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 90° 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.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

- 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.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

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.

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 m³, 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.

LIGHTWEIGHT MATERIAL - Item No.

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/m³.

LIGHTWEIGHT MATERIAL - Item No.

Prior to incorporating any of the material into the work the Contractor shall build a minimum trial area of 400 m² 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.

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

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

