



July 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 40 Over Lucas Drain Bridge Replacement
Site No. 13-234
Highway 401/Highway 40 Interchange Reconfiguration
Chatham-Kent, Ontario
GWP 3093-09-00
Ministry of Transportation, Ontario - West Region

Submitted to:

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REPORT



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PART A

FOUNDATION INVESTIGATION REPORT

**HIGHWAY 40 OVER LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234
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 40 bridge over Lucas Drain, Site 13-234.

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.

Historically, Lucas Drain was identified as the McGregor Creek Diversion. The original McGregor Creek was filled in and rerouted beneath Highway 40 at the current Lucas Drain alignment. Lucas Drain flows from west to east beneath Highway 40 which is a two-lane non-divided highway oriented in a generally northwest-southeast direction in the area of the site. The Highway 40 pavement surface at the site is at about elevation 184 metres and the invert of the drain is at about elevation 179 metres. The existing 9.3 metre long single span rigid frame bridge over Lucas Drain, south of Highway 401, was constructed in 1961 and rehabilitated in 1989. The area immediately surrounding the bridge generally consists of flat-lying agricultural lands with a commercial property in the southeast quadrant. Site photographs are provided in Appendix D.

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 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

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.



or no weight of the overlying ice. East of Windsor, toward the Chatham area, some areas of the ice sheet may have been grounded and produced hard cohesive glacial till, while in other areas the ice may have been floating or partially floating which has resulted in complex conditions in some areas. Near moraines, geologic conditions can be especially complex because of highly localized outwash (sand and gravel) deposits, silt and clay deposited in local ice-proximal lakes and ponds, and comparatively short duration re-advances and retreats of the former ice sheets.

2.3 Previous Site Construction History

This site was extensively altered in the early 1960s when the Highway 401 and Highway 40 interchange was constructed. Prior to construction of the interchange, McGregor Creek and Lucas Drain both existed along previous channel alignments. Aerial photographs from the mid-1950s also indicate that McGregor Creek meandered in the project area producing at least two former and subsequently buried meander channels with one on each of the south and north sides of the present channel. During construction of the interchange, both McGregor Creek and Lucas Drain were realigned to their present positions. The available evidence also indicates that the area between the existing highway interchange ramps in the northeast quadrant of the site and the current McGregor Creek channel was in-filled over former low-lying wet areas. In some areas, it also appears that the organic matter may not have been fully removed prior to the in-filling. The degree to which localized soft and wet areas and organic matter was or was not removed beneath the existing highway and ramp embankments remains unknown. Figure 2 illustrates the approximate locations of the former McGregor Creek and Lucas Drain channels that existed within the 5 to 7 years preceding interchange construction as well as the former meander channel. Also shown are estimated pre-interchange ground surface contours. The former meander channel was likely cut-off from flow in McGregor Creek by flood flows at some point in its history and the former channel was filled in with sediments and agricultural reworking of the area.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between May 6 and 29, 2014 during which time five 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		
201	4 693 563	338 575	183.16	8.08
202	4 693 570	338 580	182.88	25.76
203	4 693 527	338 600	184.20	8.08
204	4 693 554	338 610	183.22	30.45
205	4 693 523	338 592	183.71	26.88

The investigation was carried out using 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. 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 bedrock in borehole 204 was cored using NQ-sized rock coring equipment.

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 in excess of 150 millimetres 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 expected to be present in the fill materials and glacial till deposits as discussed in the text of this report.

The boreholes were terminated between 8.1 and 30.5 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 202 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).



FOUNDATION INVESTIGATION AND DESIGN REPORT HIGHWAY 40 OVER LUCAS DRAIN BRIDGE, SITE 13-234

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. Unconfined compressive strength (UCS) testing was carried out on a select rock core 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 Appendices A and B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil and rock types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

In general, the boreholes drilled at the highway interchange site generally encountered surficial topsoil or pavement structure over fill materials underlain by a deposit of clayey silt and silty clay glacial till with layers of sandy silt till, clayey silt, silty clay, sand and silt at depth, over shale bedrock.

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

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawings 1 and 2. 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

Borehole 203 was advanced through the traveled portion of Highway 40 and encountered 270 millimetres of asphaltic concrete underlain by 190 and 300 millimetres of granular road base and subbase materials, respectively.

4.1.2 Topsoil

Between 120 and 460 millimetres of surficial topsoil was encountered in boreholes 201, 202, 204 and 205. 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

Between 1.2 and 2.3 metres of fill materials were encountered beneath the topsoil in boreholes 201, 202, 204 and 205 and beneath the pavement structure in borehole 203 at elevations ranging from 181.9 to 183.6 metres.



The fill materials were generally cohesive, consisting of silty clay and clayey silt, with the exception of a layer of sandy silt fill in borehole 204. Measured N values from standard penetration testing carried out in the firm to very stiff fill materials ranged from 5 to 15 blows per 0.3 metres. Select samples of the fill materials had measured water contents ranging from 16 to 25 per cent. Atterberg limits determinations carried out on samples of the silty clay and clayey silt fill materials yielded liquid and plastic limits ranging from 33 to 46 per cent, and 18 to 21 per cent, respectively, and plasticity indices of 15 to 25 per cent, indicating low to intermediate plasticity. The results of the Atterberg limits determinations are shown on Figure A-6. Grain size distribution curves for samples of the fill materials are provided on Figure A-1.

4.1.4 Upper Clayey Silt

A 0.8 metre thick layer of firm to stiff clayey silt was encountered beneath the fill material in borehole 202 at elevation 181.5 metres. A single measured N value in the clayey silt was 8 blows per 0.3 metres.

4.1.5 Clayey Silt Glacial Till

Clayey silt till was encountered underlying the fill and pavement structure in boreholes 201, 203 and 204, beneath the upper layer of clayey silt in borehole 202, and beneath a layer of sandy silt till in borehole 205, at elevations between 180.1 and 181.8 metres, or about 1.4 to 3.7 metres below the ground surface. The clayey silt till deposit was between 13.4 and 14.0 metres in total thickness in boreholes 202, 204 and 205. Boreholes 201 and 203 were terminated in clayey silt till after penetrating the layer 6.7 and 5.0 metres, respectively.

Measured N values in the clayey silt till ranged from 8 to 24 blows per 0.3 metres. The clayey silt till was too stiff to permit field vane shear testing except at borehole 204. Field vane shear strength testing indicated undrained shear strengths ranging from 132 to greater than 144 kilopascals (kPa) indicating a very stiff consistency. Unconfined compressive tests were carried out on seven samples retrieved from between elevations 171 and 176 metres for Geocres Report No. 40J8-14, the Foundation Investigation and Design Report prepared in 1960 for the existing structure. The undrained shear strength of the samples ranged from 77 to 174 kPa indicating a stiff to very stiff material. In general, the clayey silt till deposit exhibited low sensitivity with values of 1.4 and 1.5.

Samples of the clayey silt till deposit had water contents ranging from 13 to 26 per cent and generally less than 20 per cent. Fourteen Atterberg limits determinations were carried out on the clayey silt till as part of the 2014 investigation, the results of which are shown on Figure A-6. Samples of the clayey silt till had liquid limits ranging from 26 to 33 per cent, plastic limits ranging from 16 to 18 per cent, and plasticity indices ranging from 10 to 16 per cent, indicating low plasticity.

Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the glacial till deposit due to its depositional history. Grain size distribution curves for samples of the clayey silt till are provided on Figure A-4.



4.1.6 Silty Clay to Clayey Silt

Cohesive soils consisting of silty clay and clayey silt were encountered underlying the clayey silt till in boreholes 202, 204 and 205. This deposit was found to be structurally different than the overlying deposit of glacial till and exhibited evidence of layering in some of the samples based on visual inspection. Standard penetration test results, Atterberg limits determinations, undrained shear strength measurements and grain size distributions also indicated that the material below elevation 167 metres is a geologically different deposit than the glacial till.

Measured N values in the silty clay to clayey silt deposit were generally less than 9 blows per 0.3 metres.

Field vane shear strength testing in the silty clay indicated undrained shear strengths ranging from 25 to 105 kPa indicating a firm to stiff consistency; however, the low values are judged to be unrepresentative of the shear strength of the soil and likely represent disturbance from both drilling and relief of porewater pressures. Based on interpretation of field vane shear tests, SPT N values, water content values and comparisons to the existing overburden stress, the undrained shear strength of the silty clay to clayey silt deposit is estimated to range from about 55 to 70 kPa.

Samples of the silty clay and clayey silt had water contents of 30 and 26 per cent, respectively. Based on one Atterberg limits determination, a sample of the silty clay had liquid and plastic limits of 41 and 20 per cent, respectively, and a plasticity index of 21 per cent, indicating intermediate plasticity. A sample of the lower clayey silt obtained in borehole 204 had a liquid limit of 30 per cent, a plastic limit of 18 per cent and a plasticity index of 12 per cent, indicating low plasticity. The results of the Atterberg limits determinations are provided on Figure A-6. The grain size distribution curves for samples of the silty clay and clayey silt are provided on Figures A-5 and A-2, respectively.

4.1.7 Sandy Silt Glacial Till

A 0.8 metre thick layer of compact sandy silt glacial till was encountered beneath the fill material in borehole 205 at about elevation 180.8 metres. A 1.7 metre thick layer of dense sandy silt till was encountered beneath the silty clay in borehole 202 at elevation 159.7 metres, and a 0.2 metre thick layer of sandy silt till was encountered beneath a layer of sand in borehole 204 at elevation 157.7 metres.

Measured N values in the sandy silt till ranged from 20 to 39 blows per 0.3 metres. Samples of the sandy silt till had water contents of 6 and 13 per cent. The gradation of the sandy silt till varied from sand and silt to sandy silt. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the sandy silt glacial till. Grain size distribution curves for samples of the sandy silt till are provided on Figure A-3.

4.1.8 Silt

Beneath the sandy silt till in borehole 202, a 0.6 metre thick layer of very dense silt was encountered at elevation 158.0 metres. A measured N value in the silt of 75 blows per 125 millimetres was obtained at the inferred soil-bedrock interface.



4.1.9 Sand

A 0.6 metre thick layer of fine to medium sand was encountered in borehole 204 beneath the clayey silt at elevation 158.2 metres. The sand was inferred to be compact based on a measured N value of 20 blows per 0.3 metres for the sample.

4.1.10 Bedrock

The bedrock surface was proven by coring or inferred, based on drilling behavior, in boreholes 202, 204 and 205. The bedrock surface was encountered at elevation 157.4 metres at the north abutment and elevation 157.5 metres at the south abutment. It should be noted that it was possible to auger through the upper 1.0 to 4.0 metres of bedrock in boreholes drilled by others for the 2011 preliminary foundation investigation of the remaining structures within the Highway 401/Highway 40 interchange. The upper 0.2 to 1.5 metres of the shale was penetrated at boreholes 202, 204 and 205 using a tricone roller drilling bit.

Borehole 204 penetrated the bedrock 1.1 metres by tricone drilling prior to coring. The bedrock in borehole 204 was cored and samples were obtained with an NQ-size core barrel for 4.7 metres before terminating the borehole. The Rock Quality Designation (RQD), Total Core Recovery (TCR) and Solid Core Recovery (SCR) for each rock core run are summarized in the following table.

Borehole	Elevation (m)		RQD (%)	TCR (%)	SCR (%)
	From	To			
204	156.34	155.57	17	33	18
	155.57	154.11	76	89	80
	154.11	152.77	100	100	100

The RQD varied from 17 to 100 per cent, increasing with depth, indicating very poor to excellent quality. The rock core was described as alternating dark grey to black and olive green, very thinly bedded mudstone and shale. The initial run of core between elevations 155.6 and 156.3 metres was rubbly. The strength of the bedrock as estimated by a geologist after examination of the core varied between R2 to R4 according to the Canadian Foundation Engineering Manual 2006 (CFEM) rock strength classification system. The strength of a select bedrock sample from borehole 204 was determined by carrying out an unconfined compression test on selected rock core, the results of which are shown in Appendix B. The intact rock is medium strong based on a measured UCS of 56 megapascals (MPa). It should be noted that the shale rock core was found to be relatively brittle once dry, and exhibited the tendency to fracture perpendicular to the core axis. Consequently, once returned to our London laboratory, only a single piece of rock core was found to meet the minimum length to diameter ratio required for compression testing (2.0) and, upon delivery to our Mississauga laboratory, had further fractured. The length to diameter ratio of the core tested was 1.95. A photograph of the rock core recovered is shown in Appendix C. Although the intact rock is classified based on testing as strong, the behaviour of the rock mass will also be influenced by fissures, joints, and other discontinuities.



The properties of the shale as evidenced by our observations and test results are typical of the Kettle Point Formation of southwestern Ontario. The shale is a relatively strong material possessing extremely weak and usually horizontal bedding planes. Vertical core is susceptible to splitting horizontally.

4.2 Methane Gas

Suspected methane gas was encountered at elevation 172.1 metres during drilling at borehole 205 as evidenced by an initial high pressure return of drilling fluids and bubbling of the drilling fluids at the ground surface. Methane gas has been reported in exploratory borings within or near the Kettle Point Formation which underlies the overburden at this site.⁵

4.3 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. Each of the boreholes advanced for the current investigation remained dry during drilling prior to switching to mud-rotary drilling techniques which introduce water and drilling fluids into the boreholes. A piezometer was installed in borehole 202 as shown on the Record of Borehole sheets. The measured groundwater levels are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Measured Groundwater Elevation (m)					
		May 8, 2014	May 16, 2014	June 4, 2014	July 10, 2014	August 12, 2014	September 24, 2014
202	182.88	174.27	175.21	175.20	175.14	175.23	174.99

The preliminary general arrangement drawing, Sheet S7, dated February 2013, indicates that the water level in Lucas Drain was at elevation 179.68 metres on March 2012 and the 50 year high water level in Lucas Drain is at elevation 182.04 metres. The water level in Lucas Drain was measured at elevation 179.37 metres on August 12, 2014. Based on the measured and encountered groundwater levels and the soil colour change from brown to grey, the inferred groundwater level in the glacial till is at about elevation 181 metres. A groundwater pressure within the underlying bedrock is equivalent to a water surface at about elevation 175.5 metres indicating that the bedrock, and possibly hydraulically connected granular layers above the 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 Dusseault, M.B. and Loftsson M., 1985: The Mechanical Properties of the Kettle Point Oil Shale, Ontario Geological Survey Open File Report 5560, 93p. 36 figures, 8 tables.




5.0 MISCELLANEOUS

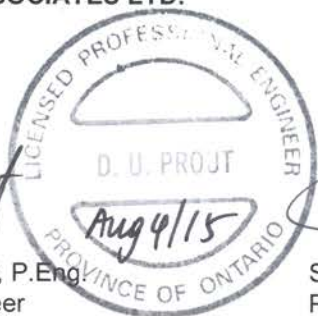
This investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc. and Henderson Drilling Inc., Ontario Ministry of Environment and Climate Change licensed well contractors. The field operations were supervised by Mr. Simon Lutz, P.Eng. and Mr. Brett Thorner, E.I.T. under the direction of the Field Investigation Manager, Mr. David J. Mitchell. The rock core was logged in detail by geologist Mr. Derek Mulligan, B.Sc.

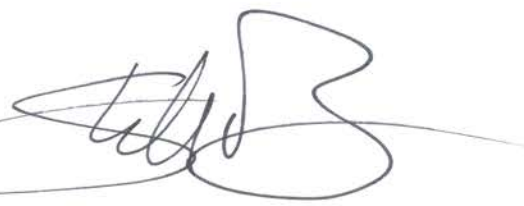
Routine laboratory tests were carried out at Golder's London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. The unconfined compression 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.

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PART B

FOUNDATION DESIGN REPORT

**HIGHWAY 40 OVER LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234
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 replacement bridge on Highway 40 over Lucas Drain, Site No. 13-234. 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 40 bridge over Lucas Drain is a single span rigid frame bridge built in 1961. Based on the original General Layout drawings dated May 1960, the bridge was designed to have a span of 9.3 metres, a width of 12.8 metres, and the abutments at about a 12 degree skew angle. The bridge abutments were designed to be supported by conventional shallow spread footings bearing in “dense grey fissured silty clay till” at about elevation 177.4 metres. The design safe allowable bearing pressure (working stress design) was 239 kPa (5,000 pounds per square foot) considering a minimum foundation width of 2.1 to 3.0 metres. Concrete retaining walls are located in each quadrant, which were designed to be founded at the same elevation as the bridge abutments. The bridge currently carries one lane each of Highway 40 north and southbound traffic over Lucas Drain. The bridge is located south of the Highway 401/Highway 40 underpass and immediately south of the existing entrance/exit of the S-E and W-N/S ramps.

The structure was inspected by MTO’s Bridge Office in 2008, 2010 and 2012 which included limited visual inspections of the foundations. No performance deficiencies associated with the foundations were noted. However, a Detailed Deck Condition Survey Report carried out by Stantec Consulting Ltd. in 2007 noted that there was settlement of the approach slabs and curbs. The magnitude of settlement was not defined. Photographs taken at the time of the inspection and since that time indicate that the differential settlement between the curb and gutter and the bridge structure is on the order of 25 millimetres.

Based on information provided in the MTO’s RFP and the preliminary design information provided by Dillon, it is understood that the replacement structure will be a single 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-E ramp traffic. The bridge will have a total width of 23.9 metres, a span of 15 metres, and will be constructed at a skew angle of approximately 12 degrees. Wingwalls and cast-in-place retaining walls are proposed for each quadrant of the structure. According to the preliminary general arrangement drawing provided by Dillon, the proposed bridge abutments are to be supported on driven steel H-piles with abutment pile caps at about elevation 177.4 metres. The profile grade of the Highway 40 pavements will be increased by about 2.6 and 2.3 metres at the north and south abutments, respectively, which will result in a downward slope of the Highway 40 grade of approximately 3.5 per cent to the south. The proposed Highway 40 pavement elevations at the north and south abutments are 187.3 and 186.7 metres, respectively. The existing embankments will be widened approximately 12 metres to the east with a marginal widening of 1.0 to 1.5 metres to the west.



Construction staging will require that the Highway 40/Lucas Drain overpass structure is built within one construction season. At the time this report was prepared, the structure construction was planned for the second construction season within the overall interchange construction contract.

6.2 Bridge Foundations

The subsurface soil conditions at the site typically consist of surficial topsoil or pavement structure underlain by fill materials over a deposit of clayey silt and silty clay glacial till with layers of sandy silt till, clayey silt, silty clay, silt and sand, over bedrock. The inferred groundwater level within the cohesive till is at about elevation 181 metres.

Semi-integral abutments may be founded on driven steel H-piles, concrete filled steel tube piles, or drilled shafts (caissons). Shallow foundations may be utilized for support of abutments if semi-integral or conventional abutments are designed; however, shallow foundations may not provide sufficient resistance to lateral loads. 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. Alternatively, integral abutments may be founded on concrete filled steel tube piles provided the additional resistance to lateral loads is adequately considered during structural design. Recommendations for each of these foundation systems are provided in subsequent report sections below.

Foundation and embankment settlement considerations at the Highway 40/Lucas Drain overpass site are somewhat different than at other structure and embankment locations on this project site. Most of the settlement issues are, like at the other sites, controlled largely by the new loads imposed by changing the embankment widths and elevations. The Highway 40/Lucas Drain site, however, is different in that a softer layer of clayey silt to silty clay was found underlying the site below approximately elevation 167 metres whereas at the other locations granular deposits dominated the stratigraphy below this elevation.

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. The preferred technical alternative from a foundation engineering perspective is to found the structure on driven steel H or tube piles with use of lightweight fill materials in the approach embankments with the amount of lightweight materials depending on settlement tolerance, economic and staging considerations.



6.2.1 Shallow Foundations

Geotechnical Axial Resistance

The abutments for the replacement bridge may be founded on spread footings on native soils at or below the following elevations.

Location	Maximum Founding Elevation (m)	Founding Material
North Abutment	179.8	Stiff to very stiff clayey silt till
South Abutment	180.8	Compact sandy silt till/ stiff to very stiff clayey silt till

A factored geotechnical resistance at Ultimate Limit States (ULS) of 425 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 275 kPa may be used for design of the footings. The SLS value corresponds to an estimated total foundation settlement of 25 millimetres or less based solely on the structural loads and assuming that the locations of the new foundations are not in the same location as the existing foundations. These values do not take into account eccentricity or inclined loads and are applicable for footing widths on the order of 3 to 9 metres. It has been assumed that the footings will be placed at or close to the same elevation as the existing footings.

In addition to settlement caused by structural loads, the planned change in grade and widening of the approach embankments may increase the total settlement of the structure by about 30 to 50 millimetres as discussed in a subsequent section of this report. Settlement caused by new embankment loads is largely associated with the soil layers below about elevation 167 metres. Differential settlement associated with the embankment should be relatively gradual across the site because of increased redistribution of stresses at progressively greater depths below the road. Total and differential settlement associated with the bridge structure loads, however, will be largely associated with soil layers above elevation 167 metres. These conditions are the underlying reason that the existing rigid frame structure appears to have performed relatively well being supported by spread footings in spite of the load and settlement likely caused by the existing embankments.

Provided that the structure can withstand differential settlements on the order of about 25 millimetres (between north and south abutments) and a similar magnitude east to west where the eastern side settles more than the western side, rigid and shallow foundations may be a relatively economical and suitable solution for this site. Where the new bridge foundations are located coincident with the footprint of the existing bridge foundations, settlement associated with the structural dead load should be relatively minor. Where the bridge is widened, the ground beyond the existing foundation perimeters will not have been stressed within the zone from the existing foundation bearing elevation to a depth approximately equal to the existing foundation width (as measured parallel to the road centerline). Therefore, if shallow foundations are to be used, the abutments should be designed to be sufficiently rigid to allow for some redistribution of local stresses beneath the foundations.



If shallow foundations are to be considered for this site, use of lightweight fill may assist with improving foundation performance and mitigating the magnitudes and time-rate of differential settlement. In particular, widening of the embankment eastward using a combination of lightweight and conventional earth fill would be most beneficial for limiting differential settlement. Recommendations regarding use of lightweight fill are provided in a subsequent report section.

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 Canadian Highway Bridge Design Code (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' (degrees)	Unfactored Coefficient of Friction, $\tan \phi'$
Compact sandy silt till	32	0.62
Stiff to very stiff clayey silt till	30	0.58

Frost and Scour Protection

All footings should be provided with a minimum of 1.2 metres of earth cover or thermal equivalent for frost protection purposes. Footings should be adequately protected against scour as noted in Section 1.9.5.2 of the CHBDC. Noting that the design elevation for the bed of Lucas Drain (former McGregor Creek Diversion) is elevation 179.93 metres, it may be necessary to found footings below this elevation to provide protection against scour.

6.2.2 Deep Foundations

The preliminary bridge design indicates semi-integral abutments for the replacement structure to be founded on driven steel HP 310x110 piles. Consideration may also be given to founding the structure on 324 millimetre outside diameter (OD) by 9.5 millimetre wall thickness, concrete-filled steel tube piles. These pile types are considered appropriate for conventional or integral abutment design. If a conventional abutment design is selected consideration may also be given to founding the abutments 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

For design, the factored axial geotechnical resistances at ULS and geotechnical reaction at SLS for HP 310 x 110 piles and concrete filled steel tube piles as described above driven closed ended to bear on competent shale bedrock at or below the anticipated elevations shown are provided in the following table. SLS values are not provided since the bedrock is considered to be effectively unyielding.

Pile Type	Pile Location	Assumed Cut-off Elevation (m)	Approximate Tip Elevation (m)	Factored Geotechnical Resistance at ULS (kN)
HP 310 x 110	North Abutment	177.7	157.5	2,000
	South Abutment		157.5	
324 mm OD x 9.5 mm concrete filled steel tube	North Abutment	177.7	157.5	2,500
	South Abutment		157.5	

The above cut-off elevations have been assumed based on the proposed pile cap elevations shown on the preliminary general arrangement drawing and an assumed 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)

Drilled shaft foundations (caissons) could be constructed for support of the planned bridge. At this site, however, the caissons would need to be drilled to rock to limit total and differential settlements to magnitudes less than the spread foundation option. In this case, for end-bearing caissons socketed at least 1 metre into sound rock, a factored geotechnical resistance at ULS of 4 MPa may be used in design provided that:

- the rock along the shaft and at the base of the socket is free of cavities/voids and cleaned of loose materials, and
- the caissons would have to be fully lined to allow thorough cleaning and inspection prior to pouring concrete.



The bedrock is considered to be a relatively unyielding medium and as such, the geotechnical resistance at SLS would not be considered applicable.

It is considered essential that a steel liner be used during construction of the caisson. This is to retain the sides of the augered holes and to effectively limit water flow into the open hole emanating from the more pervious and water bearing zones known to exist. It should be noted that pockets and/or layers of water-bearing granular soils are also known to exist. In these areas, groundwater inflow into the caissons should be anticipated. It is expected that the use of steel liners will likely be ineffective in controlling the upward water flow at the base of the caissons emanating from the granular soils and bedrock. The contract should have a provision that tremie concrete be used to construct the caissons/bored piles. The presence of cobbles and/or boulders could potentially be problematic during drilling and/or advancement of the liner for the caisson. Provision should be made in the contract to deal with these issues should they arise.

Local experience has further indicated the presence of methane gas in granular deposits overlying the bedrock and/or in the bedrock. Air monitoring and adequate ventilation will be required during construction should gases be detected. Also, the design of the site development should account for the presence of methane and include for effective gas management and control.

As noted below, downdrag loads will be induced by settlement of the native soils under the loads of the new embankments unless lightweight fill options are used to limit new embankment loading.

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 proposed grade raise of between 2.3 and 2.6 metres and embankment widening will result in consolidation settlement of the native soils and, therefore, the piles could experience downdrag loads if the piles are installed prior to the grades being changed. It is anticipated that downdrag loads on driven piles may be on the order of 160 kN per pile and on drilled caissons may be on the order of 475 kN.

Consolidation settlement is time-dependent and will not completely occur during the construction period, unless the embankments are placed well in advance of bridge construction. As discussed in Section 6.7.1, it is anticipated that 90 per cent of the long-term settlement is expected to occur about 7 years following embankment construction with 50 per cent of the settlement occurring within 2 years of construction of the embankment widenings.

Reduction of the downdrag loads on the piles could be achieved by pre-loading the native soils by constructing the widened embankments well in advance of pile driving. If construction staging allows, downdrag loads on abutment piles may be effectively reduced by placing embankment fills a minimum of 1 year prior to driving the piles. Consideration could also be given to the use of lightweight fill materials for some portion of the new embankment fill. Given the small size of this site and project and the stiffness of soils above the most compressible of the deposits, the use of wick drains to accelerate consolidation is not considered practical. Sand drains, however, may be a practical method to accelerate consolidation and the choice of settlement



mitigation methods may be driven largely by economic, constructability and staging considerations as discussed in a subsequent section of this report.

Resistance to Lateral Loads

It is understood that inclined piles may be used for support of the semi-integral abutments. Inclined piles are also considered appropriate for abutments subject to lateral loads. In the case of integral abutments, vertical piles or caissons must provide the resistance to lateral loading.

The horizontal reaction to the pile can be estimated using the following equations and soil properties.

$$k_h = \begin{array}{ll} \text{coefficient of horizontal subgrade} & = n_h (z/d) \quad \text{for cohesionless soils} \\ \text{reaction in megapascals per metre} & = \frac{67 s_u}{d} \quad \text{for cohesive soils} \\ \text{(MPa/m)} & \end{array}$$

where:

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

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

Location	Soil Type	Elevation (m)	n_h (MPa/m)	S_u (MPa)
CSPs for integral abutments	Granular backfill	Where applicable	5 - 10	-
Abutments	Firm to stiff clayey silt fill	Surface to 182.0	-	0.03 – 0.10
North Abutment	Stiff to hard clayey silt till	182 to 167	-	0.10 – 0.23
	Firm to stiff clayey silt to silty clay till	167 to 160	-	0.03 – 0.08
South Abutment	Stiff to hard clayey silt till	181 to 167	-	0.08 – 0.22
	Firm to very stiff silty clay till	167 to 161	-	0.03 – 0.11
	Soft to stiff silty clay/clayey silt	161 to 157	-	0.06 – 0.10



The lateral resistances for the various deep 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)	35	*
HP 310 x 110, strong axis bending (for semi-integral or conventional abutments)	200	110
324 mm OD x 9.5 mm tube (for integral abutments)	45	*
324 mm OD x 9.5 mm tube (for semi-integral or conventional abutments)	210	120
Concrete Caissons	1,500	500

*Load to mobilize 10 mm 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". 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 concrete caissons. A 1.2 metre diameter caisson socked into the bedrock as described above was assumed. For semi-integral or conventional abutments, the load was assumed to be applied at the proposed pile cap elevation. For integral abutments, the horizontal load was assumed to be applied at an assumed height of 3 metres above the proposed pile-cap elevation. An ultimate compressive strength of 32 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.

If the design is found to be sensitive to the horizontal modulus of subgrade reaction, additional analyses should be carried out to refine these parameters given specific foundation sizes, displacement tolerance, and static and dynamic load characteristics along with sufficient information to ascertain whether the piles are most representative of fixed or free end conditions at the pile cap elevation.

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

Based on the preliminary general arrangement drawing, it is understood that wingwalls and cast-in-place retaining walls will be required in each quadrant of the structure to retain the approach fills behind each abutment. The walls will be between 9 and 14 metres in total length and up to 9.2 metres in total height. The preliminary design indicates that the walls are to be parallel to the highway and founded on driven HP 310x110 piles. Supporting the retaining walls on deep foundations is not necessary at this site and conventional shallow footings or RSS leveling pads may also be used subject to the discussion related to settlement of shallow spread footings for the abutments. The preliminary design indicates that the pile caps for the north walls are to be stepped up from about elevation 179.0 to 181.0 metres and the pile caps for the south walls are to be stepped up from about elevation 179.0 to 180.0 metres. Consideration may be given to designing the retaining walls as concrete cantilever or gravity walls, or Retained Soil System (RSS) walls.

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. Concrete gravity walls could consist of pre-cast or cast-in-place elements provided that the joints between the walls and bridge structure permit differential movement.

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 RSS walls are selected, the geotechnical aspects of the global stability of the detailed retaining wall design should be reviewed prior to construction. 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.

6.3.1 Retaining Wall Foundations

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.



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 and be protected from scour. 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.

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

Wall Location	Elevation (m)
Northwest	179.8
Northeast	180.8
Southeast	181.1
Southwest	180.8

Concrete gravity and cantilever walls and RSS walls founded on the stiff to very stiff clayey silt till or compact sandy silt till may be designed using a factored geotechnical resistance at ULS of 475 kPa and a geotechnical reaction at SLS of 300 kPa. The SLS value corresponds to an estimated total settlement of 25 millimetres. All retaining wall foundations must be protected against scour as noted in the CHBDC Section 1.9.5.

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 walls and the founding soil.



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Wall Type	Interaction	Effective Internal Angle of Friction, ϕ' (degrees)	Coefficient of Internal Friction, $\tan \phi'$	Angle of Interface Friction, δ (degrees)	Coefficient of Interface Friction, $\tan \delta$
Concrete Gravity or Cantilever Wall	Cast-in-place concrete strip footing on:				
	– Compact sandy silt till	32	0.62	-	-
	– Stiff to very stiff clayey silt till	30	0.58	-	-
	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.⁶ 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 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.



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 structures, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments and retaining walls, 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 Special Provision (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).
- 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

Coefficient 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 may be present in the glacial tills 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 understood that the abutments for the new structure will be offset from the existing abutment locations and, therefore, it is anticipated that the existing abutment foundations will not impact pile driving/caisson drilling for the replacement structure. 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. 5 indicating piles are to be driven to bedrock should be included in the project drawings.

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.



In order to reduce the potential for buildup of excess pore water pressure in the new embankment fills, 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 and compact sandy 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. It is anticipated that the final embankments will be up to approximately 6 metres in height as measured from the surrounding natural elevations within about 75 metres north, and up to 2.5 metres in height south of the bridge and the crests will be approximately 24 metres wide.

Data Interpretation

Estimates of engineering parameters used in the settlement analysis were based on SPT N values obtained in boreholes 201 to 205, and an oedometer test carried out on a sample from borehole 616 advanced for the high fill embankments portion of this project (Geocres Report No.40J8-61). 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 approximated using the SPT N field values using the following relationship:

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

This correlation between the SPT values is an approximation due to the inherent variability of the energy delivered during the SPT procedure; however, this 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.



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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.

$$\sigma'_p = s_u / 0.22 \quad (\text{after Mesri 1975}^7)$$

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, all of the soils at the site above elevation 167 metres would exhibit recompression behaviour and that the preconsolidation pressure would have little if any effect on the magnitude of settlement. However, below elevation 167 metres, the weight of the new embankment fill materials combined with the existing in situ stresses is close to exceeding the estimated preconsolidation pressure.

Settlement Analysis

Settlement of the founding soils due to the proposed embankment loading was analysed using Settle^{3D} Version 2.0, a 3-dimensional program by Rocscience for the analysis of vertical consolidation and settlement under foundations, embankments and surface loads. The existing and future embankment loads were inferred from the drawings provided by Dillon. The table below summarizes the geotechnical engineering parameters and simplified stratigraphy used in the analysis.

Material and Elevation (m)	γ (kN/m ³)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Clayey Silt Till 179.0 to 181.0	21	-	-	0.3	0.147	0.016	682	0.47
Clayey Silt Till 175.0 to 179.0	21	-	-	0.3	0.135	0.015	636	0.47
Clayey Silt Till 167.0 to 175.0	21	-	-	0.3	0.135	0.015	500	0.47

⁷ Mesri, G. (1975). New Design Procedure for Stability of Soft Clays: Discussion. Journal of the Geotechnical Engineering Division, ASCE 101(4), 409 – 411.



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Material and Elevation (m)	γ (kN/m ³)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Silty Clay to Clayey Silt 158.0 to 167.0	20	-	-	0.3	0.232	0.026	290	0.69

Assuming that the new embankment widening and grade raise consists of conventional engineered fill, with a unit weight between 21 and 22 kilonewtons per cubic metre (kN/m³), the combination of the new and existing embankment loads and in situ vertical effective stresses are expected to be near the preconsolidation pressure of the silty clay and clayey silt materials below approximately elevation 167 metres. There is some uncertainty with respect to the magnitude of settlement that may be caused by the planned construction. Therefore, settlement associated with the previous construction of the existing embankments was also estimated in an effort to provide an approximate calibration to settlement estimates. In this case, construction of the existing embankments likely produced maximum road centreline settlement on the order of 10 to 30 millimetres within the immediate vicinity of the bridge structure.

Based on these assumptions, 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 80 millimetres at the south abutment decreasing southward to about 30 millimetres at a point 75 metres south of the abutment; and
- from about 70 millimetres at the north abutment increasing northward to about 120 millimetres at a point 75 metres north of the abutment, depending on the northward extent of the compressible silty clay layer found below elevation 167 metres.

These values exceed the MTO's post-construction settlement criteria for longitudinal transition zones and embankment widening on non-freeways. Ninety per cent of the long-term settlement is expected to occur about within 5 to 10 years following embankment construction with 50 per cent of this settlement occurring within 2 years.

6.7.2 Settlement Mitigation

In order to reduce post-construction settlement, 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 consisting of:
 - water-cooled blast-furnace slag;
 - cellular concrete; or
 - expanded polystyrene blocks.

Each of these settlement mitigation options is described below.



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 (downdrag) 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 bridge. In order to limit, 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 silty clay layer. However, given the planned staging and embankment heights, this approach will not be possible. Therefore, in the vicinity of the future bridge the embankment should be widened to its future full width and filled as high as possible in the construction year preceding bridge construction if at all possible. At the time the bridge is constructed, the embankment materials will then require removal and reinstatement along with lightweight fill, 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. 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. Pre-drilling may be required to elevation 167 metres. 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 160 metres and be constructed on a uniform triangular grid spacing of 1.5 to 2 metres. Care will be needed to minimize the effects of puncturing into the underlying granular layers near the bedrock surface if the sand drain drill holes will not be fully filled with water during drilling. 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. 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 this bridge site is to minimize the new embankment net loads on the underlying soil to the degree practicable. Four 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;
- Tire Derived Aggregate (TDA): TDA consists of scrap tires shredded to strips 50 to 300 millimetres in length. This material has a unit weight in the range of 8 kN/m³. The use of TDA may create the potential for leaching of environmentally adverse chemicals and, therefore, require permitting, monitoring and/or capping. Also, in order to prevent self-heating and potential combustion, the thickness of TDA used as embankment fill is restricted to no more than 3 metres. Regulatory approvals and related processes to control environmental concerns are not yet resolved in Ontario.
- 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. Blast furnace slag is susceptible to crushing if over compacted. A non-standard special provision (NSSP) for lightweight fill material has been included in Appendix E which discusses construction methods and means of preventing overcrushing.

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

For this project, design of the approach embankments should be based on the following sequence during the construction season prior to bridge construction:

1. within a distance of about 20 metres north of the north abutment and 20 metres south of the south abutment, strip the ground surface in all areas that will be within the footprint of the final embankments to the native soil interface, excluding the areas covered by the existing embankments;
2. place geotextile filter fabric to separate the native soils from conventional earth fill and define the general boundaries for subsequent removal; and



3. place conventional earth fill in the areas of embankment widening to match or exceed the existing roadway grade elevations to preload the underlying ground to assist with limiting differential settlement between the new and old construction.

The conventional earth fill placed as described above need only be controlled to the degree that an in situ unit weight of 21 kN/m^3 is achieved since this material will be subsequently removed. The following construction season's activities should then include:

1. construct new bridge foundations and abutments;
2. remove the earth fill previously placed as described above to native materials;
3. remove existing embankment materials to match the native soil interface and produce a uniform subgrade elevation for subsequent construction of the new embankment;
4. removing any disturbed native soils;
5. placement of lightweight fill to achieve planned dimensions; and
6. final construction of the pavement section.

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 and rutting 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 (north east and south east 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 until the fill is removed 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, topsoil and/or fill and extend into the underlying clayey silt till and sandy 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 are expected to 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 footing 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. Any fill materials and the native firm clayey silt would be classified as Type 3 soils. The native cohesive till and sandy silt till would be classified as Type 2 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 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:



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- H = the height of the excavation at any point (m)
K_a = active coefficient of earth pressure
γ = soil unit weight (kN/m³)
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 (degrees)	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
Sandy silt till	0.27	0.43	3.7	35	21.0
Clayey silt to silty clay till	0.31	0.47	3.3	32	21.0
Clayey silt	0.36	0.53	2.8	28	20.0
Silty clay	0.38	0.55	2.7	27	20.0


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.

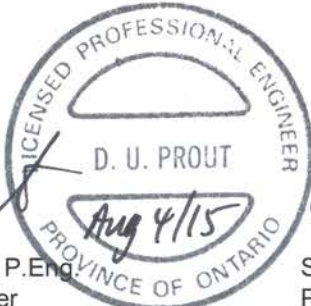



7.0 MISCELLANEOUS

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. An independent quality review of this report was carried out by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES – BRIDGE REPLACEMENT

Highway 40 over Lucas Drain Bridge Replacement, Site 13-234
 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 till and compact sandy silt till	<ul style="list-style-type: none"> • Feasible 	<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. 	<ul style="list-style-type: none"> • Low 	<ul style="list-style-type: none"> • Relatively low construction risk. • Deeper excavations required if soil at founding elevation is unsuitable. • Higher risk for long-term settlement related performance problems.
End bearing steel H-piles or steel tube piles driven to refusal on shale bedrock	<ul style="list-style-type: none"> • Feasible • Preferred technical alternative 	<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. • More construction noise and vibration compared to shallow foundations. • Cannot be visually inspected at depth. • Integrity inspection requires specialty dynamic testing. • Settlement mitigation required for approaches. 	<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.

COMPARISON OF FOUNDATION ALTERNATIVES

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS/ CONSEQUENCES
Concrete caissons drilled into bedrock	<ul style="list-style-type: none"> • Feasible 	<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. • Potential for encountering methane gas and the need for a methane monitoring plan. 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Cleaning of base could be problematic or overlooked during construction. • Excessive settlement can occur depending on selected tip elevations and loads.

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

Prepared By: NG
 Checked By: DUP/SJB



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
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

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

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



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

RECORD OF BOREHOLE No 201

1 OF 1

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693563.2 , E 338575.2 ORIGINATED BY BT
DIST HWY 40 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	W _P	W	W _L		
183.16	GROUND SURFACE															
0.00	TOPSOIL, clayey silt															
0.15	Brown															
	FILL, clayey silt, some sand, trace gravel															
	Firm															
	Brown		1	SS	5											
181.79																
1.37	CLAYEY SILT TILL, some sand, trace gravel		2	SS	12											
	Stiff to very stiff															
	Brown to grey below about elev. 180.4m		3	SS	19											
			4	SS	16											
			5	SS	12											
			6	SS	14											
			7	SS	14											
			8	SS	15											
175.08	END OF BOREHOLE															
8.08	Groundwater not established during drilling on May 6, 2014.															

RECORD OF BOREHOLE No 202

1 OF 2

METRIC

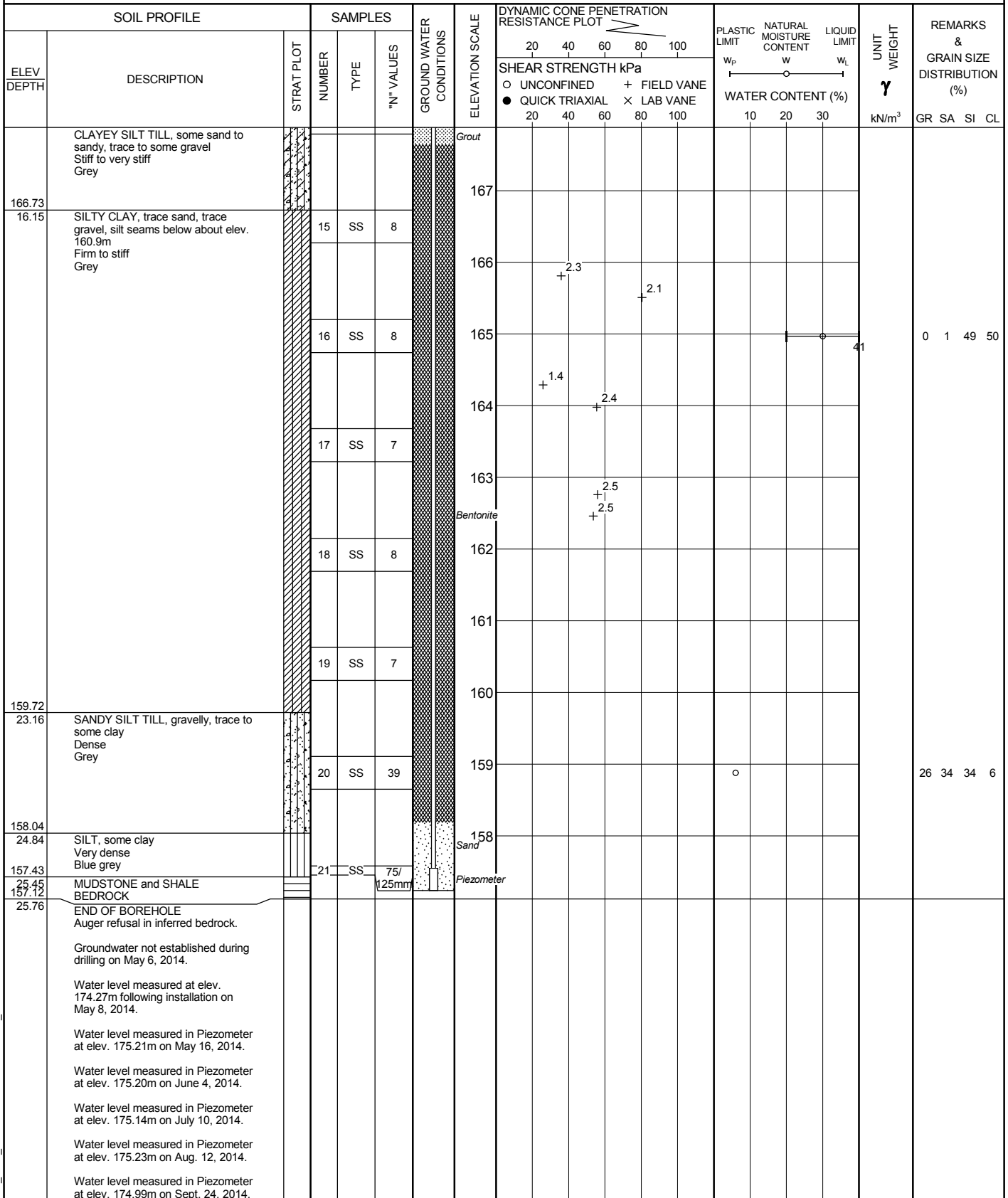
PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693570.1 , E 338579.9 ORIGINATED BY SL
DIST HWY 40 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE May 7 - 8, 2014 CHECKED BY

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						
182.88	GROUND SURFACE							20 40 60 80 100						
0.00	TOPSOIL, silty clay, some sand, roots Black							○ UNCONFINED + FIELD VANE						
0.12	FILL, silty clay, trace to some sand Firm to stiff Brown							● QUICK TRIAXIAL × LAB VANE						
181.51			1	SS	8		182							0 5 55 40
1.37	CLAYEY SILT, some sand Firm to stiff Brown		2	SS	8		181							
180.75														
2.13	CLAYEY SILT TILL, some sand to sandy, trace to some gravel Stiff to very stiff Grey		3	SS	17		180							
			4	SS	12		179							6 25 48 21
			5	SS	14		178							
			6	SS	12		177							
			7	SS	15		176							
			8	SS	14		175							1 16 50 33
			9	SS	16		174							
							173							
			10	SS	14		172							
							171							4 27 45 24
			11	SS	12		170							
							169							
			12	SS	13		168							
			13	SS	19									
			14	SS	24									

Continued Next Page

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

PROJECT 13-1132-0111		RECORD OF BOREHOLE No 202		2 OF 2		METRIC	
W.P. 3093-09-00		LOCATION N 4693570.1, E 338579.9		ORIGINATED BY SL			
DIST _____ HWY 40		BOREHOLE TYPE POWER AUGER, HOLLOW STEM		COMPILED BY WDF			
DATUM GEODETIC		DATE May 7 - 8, 2014		CHECKED BY _____			



LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 30/07/15

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

RECORD OF BOREHOLE No 203

1 OF 1

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693527.2 , E 338599.6 ORIGINATED BY BT
DIST HWY 40 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WDF
DATUM GEODETIC DATE May 12, 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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
184.20	GROUND SURFACE						20	40	60	80	100	WATER CONTENT (%)						
0.00	ASPHALTIC CONCRETE																	
0.27	FILL, granular base, sand and gravel, crushed Brown																	
0.46																		
0.76																		
	FILL, granular subbase, sand and gravel, some silt Brown		1	SS	15													
	FILL, clayey silt, sandy, trace to some gravel Stiff Brown		2	SS	11													
			3	SS	12													
181.15																		
3.05	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Brown and grey to grey		4	SS	14													
			5	SS	23													
			6	SS	13													
			7	SS	11													
			8	SS	11													
			9	SS	8													
176.12																		
8.08	END OF BOREHOLE																	
	Groundwater not established during drilling on May 12, 2014.																	

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

PROJECT 13-1132-0111

W.P. 3093-09-00

LOCATION N 4693553.9 , E 338610.2

ORIGINATED BY SL

DIST HWY 40

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

COMPILED BY WDF

DATUM GEODETIC

DATE May 27 - 28, 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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	×						
								● QUICK TRIAXIAL	×	LAB VANE						
							20 40 60 80 100		10 20 30							
167.68							168									
15.54	SILTY CLAY, some sand, trace gravel Firm to very stiff Grey		15	SS	9		167									
							166									
			16	SS	6		165									
							164									
			17	SS	3		163									
							162									
			18	SS	6		161									
161.27							160									
21.95	CLAYEY SILT, trace sand, silty sand and silt seams Soft to firm Grey		19	SS	2		159									
							158									
			20	SS	4		157									
158.23							156									
24.99	SAND, fine to medium, some silt Compact Grey		21	SS	20		155									
157.68	SANDY SILT TILL, clayey, trace gravel Compact Grey						154									
25.54	MUDSTONE and SHALE BEDROCK, very thinly bedded, localized sulphide mineralization, initial run is rubbly, weathered bedrock surface into solid bedrock with rare natural fractures along bedding planes Alternating dark grey to black and olive green bedding is horizontal 90 degrees to core axis and defined by strong (R4) black shale and olive green (R2/R3) limey mudstone		22	SS	75/ 50mm		153									
25.76			23	NQ RC			152									
			24	NQ RC			151									
			25	NQ RC			150									

Continued Next Page

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

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 204		3 OF 3	METRIC
W.P. <u>3093-09-00</u>	LOCATION <u>N 4693553.9 , E 338610.2</u>	ORIGINATED BY <u>SL</u>			
DIST <u> </u> HWY <u>40</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE</u>	COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>	DATE <u>May 27 - 28, 2014</u>	CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			W _p	W	W _L		GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)						
152.77 30.45	END OF BOREHOLE Groundwater not established during drilling on May 27, 2014.																	

RECORD OF BOREHOLE No 205

1 OF 2

METRIC

PROJECT 13-1132-0111
W.P. 3093-09-00 LOCATION N 4693522.8 , E 338592.0 ORIGINATED BY SL
DIST HWY 40 BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE COMPILED BY WDF
DATUM GEODETIC DATE May 29, 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	GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE						WATER CONTENT (%)						
						● QUICK TRIAXIAL × LAB VANE	20	40	60	80	100	10	20	30						
183.71	GROUND SURFACE																			
0.00	TOPSOIL, clayey silt, some sand, roots																			
0.15	Black																			
	FILL, silty clay, some sand, trace gravel																			
	Stiff		1	SS	8		183													
	Brown		2	SS	13		182										0	18	48	34
			3	SS	14		181													
180.81																				
2.90	SANDY SILT TILL, some clay, trace gravel		4	SS	23		180										4	33	46	17
	Compact																			
	Brown																			
180.05	CLAYEY SILT TILL, some sand to sandy, trace gravel		5	SS	19		179													
	Stiff to very stiff		6	SS	17		178													
	Grey		7	SS	15		177													
			8	SS	16		176													
			9	SS	18		175													
			10	SS	14		174													
			11	SS	11		173													
			12	SS	21		172													
			13	SS	13		171													
			14	SS	15		170													
							169													
	Methane gas encountered while driving spoon from about elev. 172.13m																			

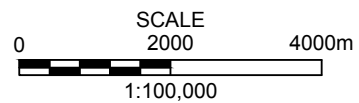
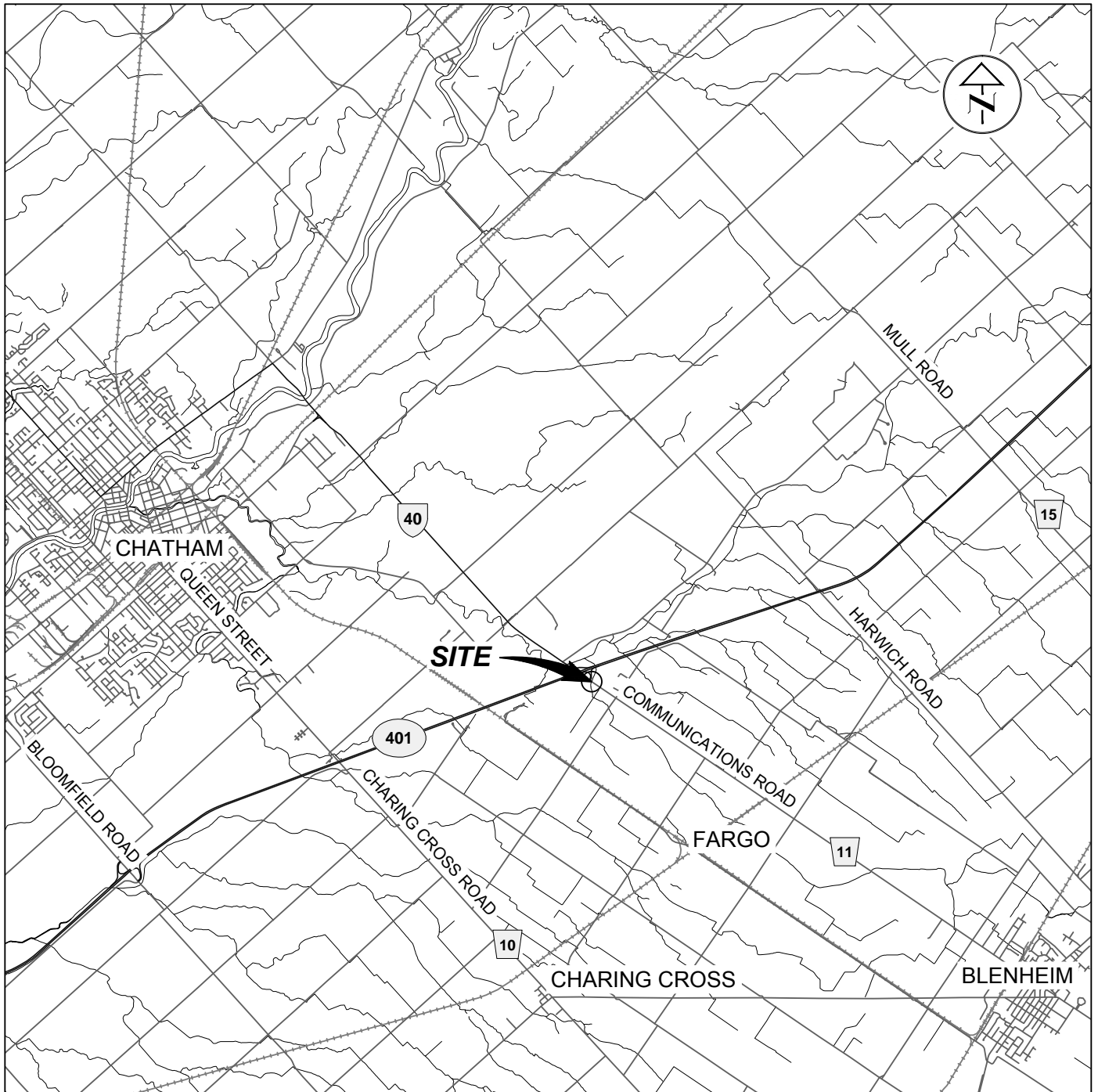
Methane gas encountered while driving spoon from about elev. 172.13m

Continued Next Page

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100						10 20 30			
								SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL × LAB VANE										
								20 40 60 80 100										
166.64	CLAYEY SILT TILL, some sand to sandy, trace gravel Stiff to very stiff Grey		15	SS	11									0 10 46 44				
17.07	SILTY CLAY, trace sand Stiff Grey		16	SS	3													
			17	SS	3													
			18	SS	5									0 2 48 50				
			19	SS	5													
			20	SS	5													
			21	SS	6													
157.01	BEDROCK		22	SS	70/50mm													
26.70	END OF BOREHOLE																	
26.88	Split-spoon refusal on inferred bedrock. Groundwater not established during drilling on May 29, 2014.																	

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



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

NOTE

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

PROJECT

LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234
HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

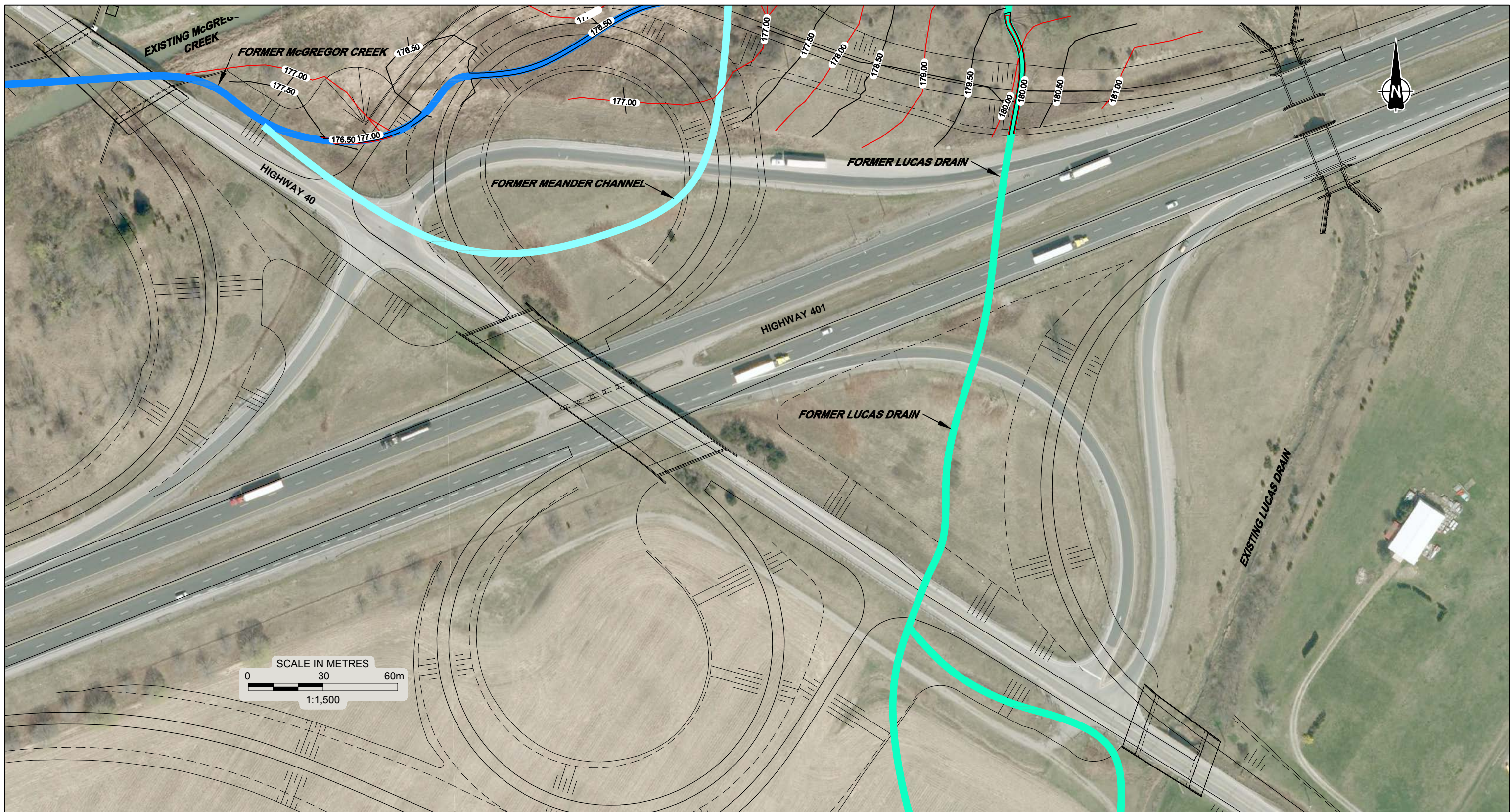
KEY PLAN



PROJECT No. 13-1132-0111		FILE No. 1311320111-1000-F02001	
CADD	LMK	July 17/14	SCALE AS SHOWN
CHECK			REV. 0

FIGURE 1

Drawing file: 1311320111-1000-F02002.DWG Jul 30, 2015 4:50pm




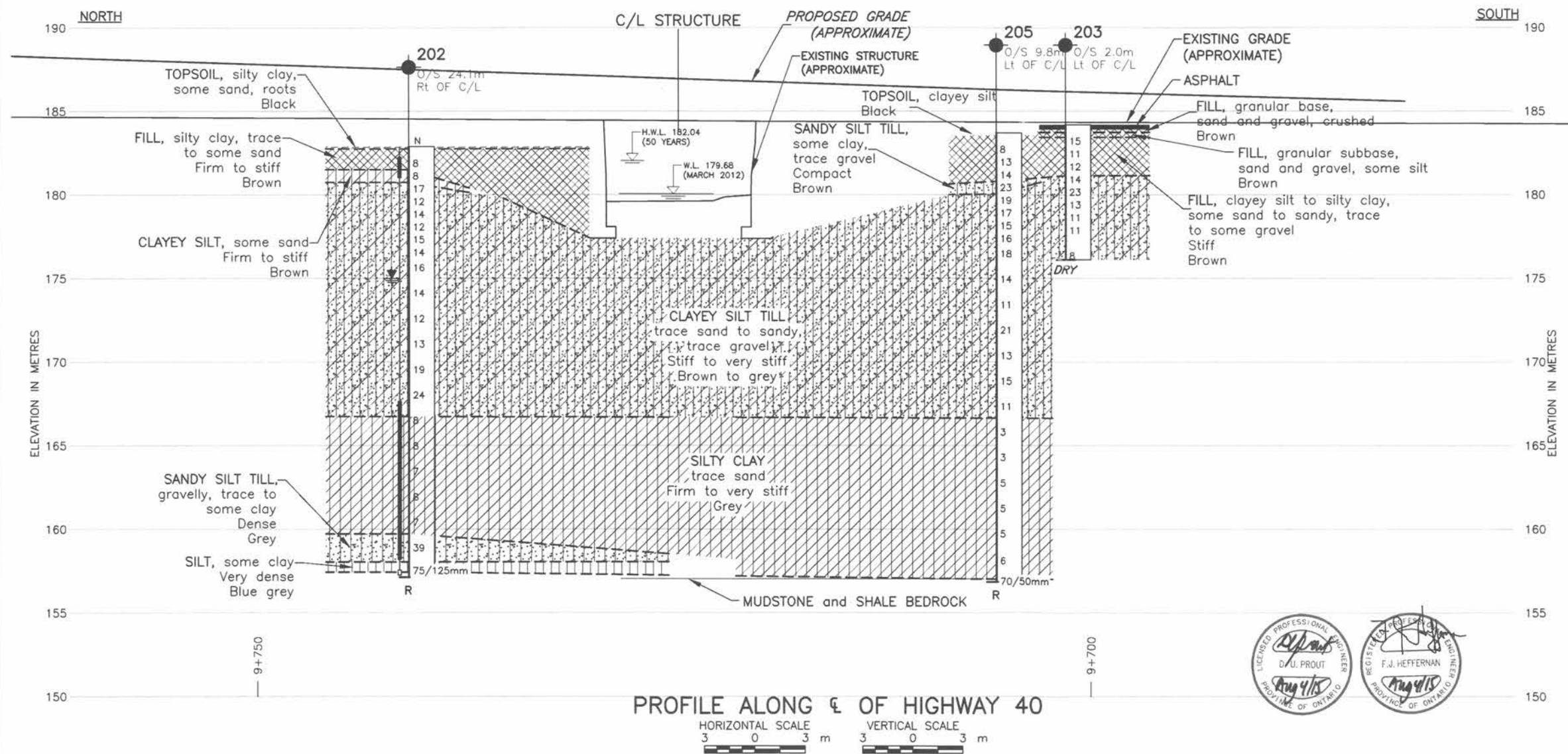
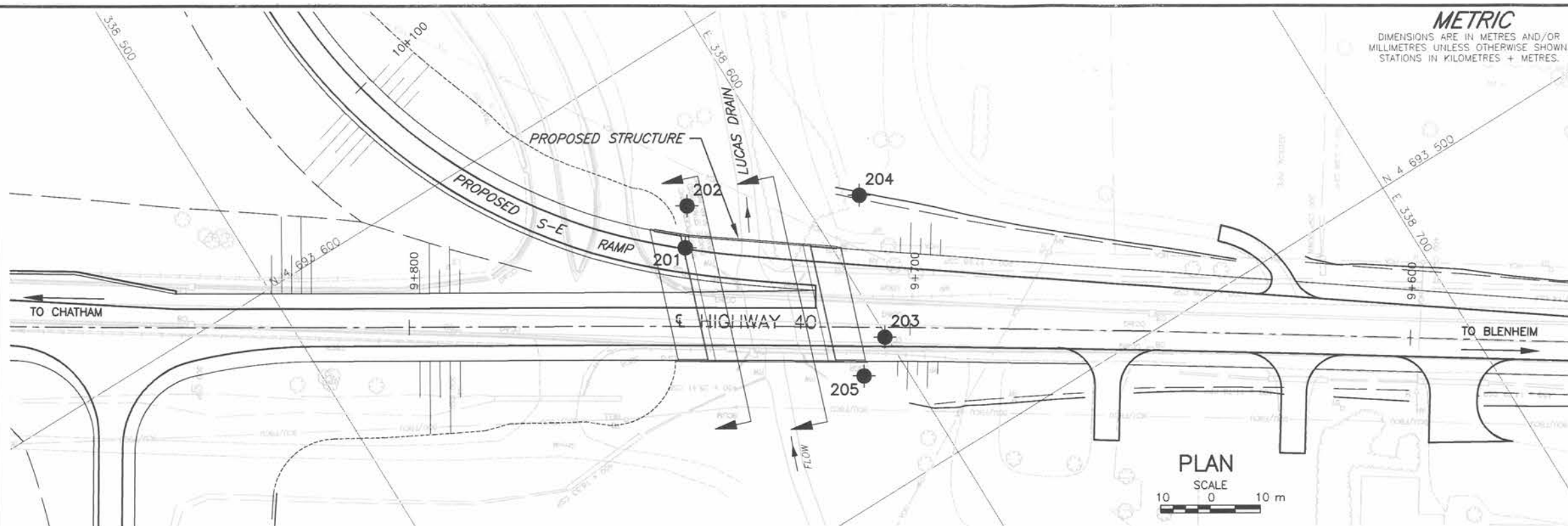
REFERENCE

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

NOTES

THIS DRAWING IS SCHEMATIC ONLY AND IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.
ALL LOCATIONS ARE APPROXIMATE.

PROJECT		LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234 HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00	
TITLE		FORMER LOCATION OF MCGREGOR CREEK, LUCAS DRAIN, AND MEANDER CHANNEL	
 Golder Associates LONDON, ONTARIO	PROJECT No.	13-1132-0111	FILE No. 1311320111-1000-F02002
	CADD	DCH/WDF	Feb. 13/15
	CHECK		
		SCALE	AS SHOWN
		REV.	0
FIGURE 2			

CONT No.
WP No. 3093-09-00HIGHWAY 40 OVER LUCAS DRAIN
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE
RECONFIGURATION
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

LEGEND

- Borehole - Current Investigation
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, on October 18, 2011.
- DRY WL not established during drilling.
- R Auger or Split-spoon refusal on inferred bedrock

No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
201	183.16	4 693 563.2	338 575.2
202	182.88	4 693 570.1	338 579.9
203	184.20	4 693 527.2	338 599.6
204	183.22	4 693 553.9	338 610.2
205	183.71	4 693 522.8	338 592.0

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 Dillon
Received Feb. 5, 2015.

NO.	DATE	BY	REVISION
1	Feb 26/15	WDF	UPDATED BASE PLANS - SEE REFERENCE

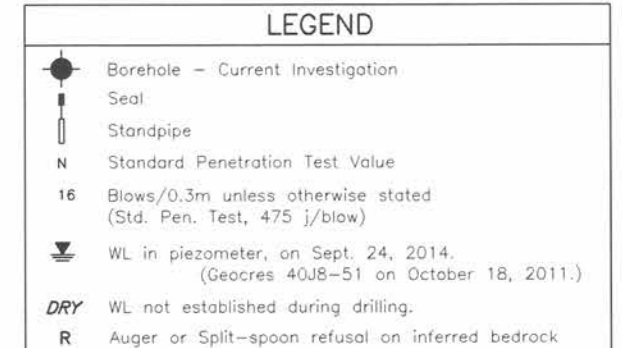
Geocres No. 40J8-64

HWY.	401	PROJECT NO.	13-1132-0111	DIST.
SUBM'D.	NG	CHKD.	NG	DATE: Nov. 20/14
DRAWN:	LMK\WDF	CHKD.	DUP	APPD. FJH
DWG.	1			



SHEET

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

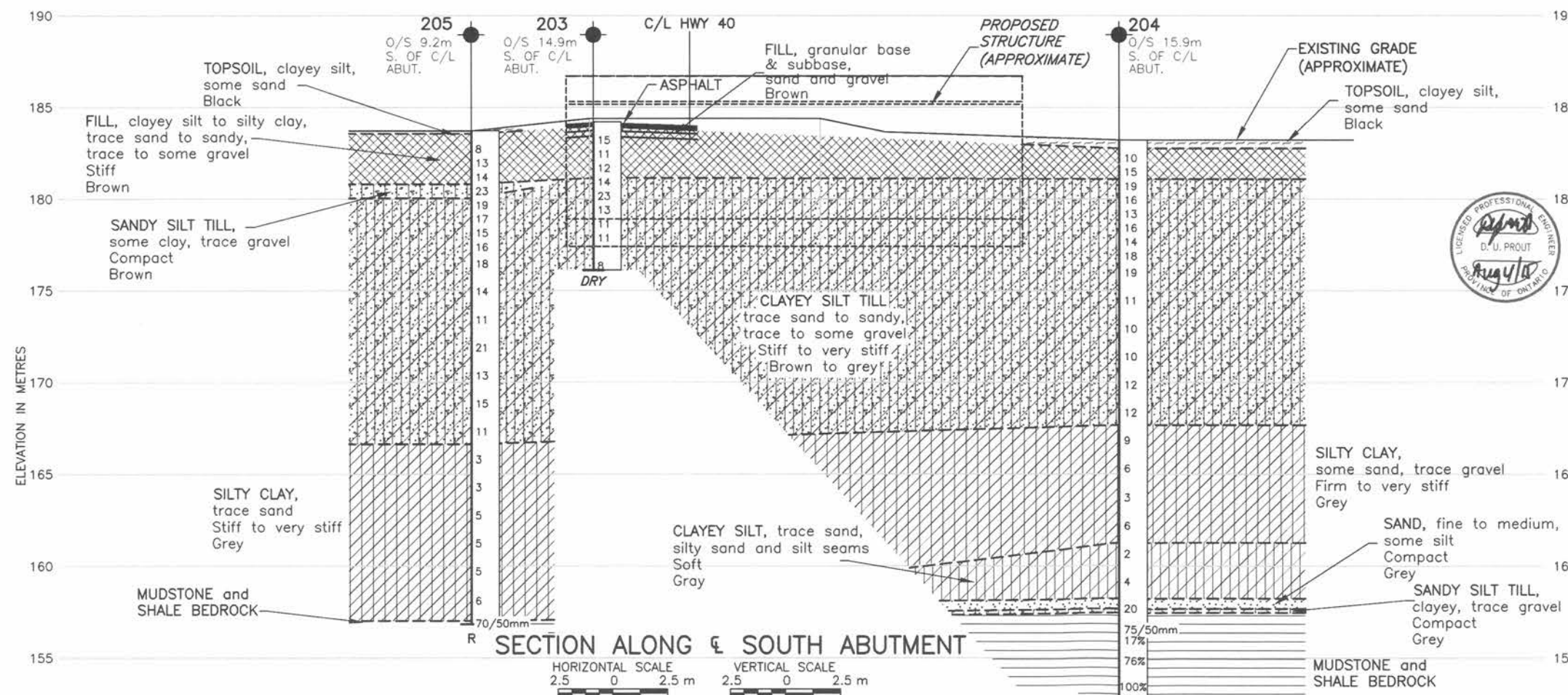


No.	ELEVATION	CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
201	183.16	4 693 563.2	338 575.2
202	182.88	4 693 570.1	338 579.9
203	184.20	4 693 527.2	338 599.6
204	183.22	4 693 553.9	338 610.2
205	183.71	4 693 522.8	338 592.0

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

Base plans provided in digital format by McCORMICK RANKIN

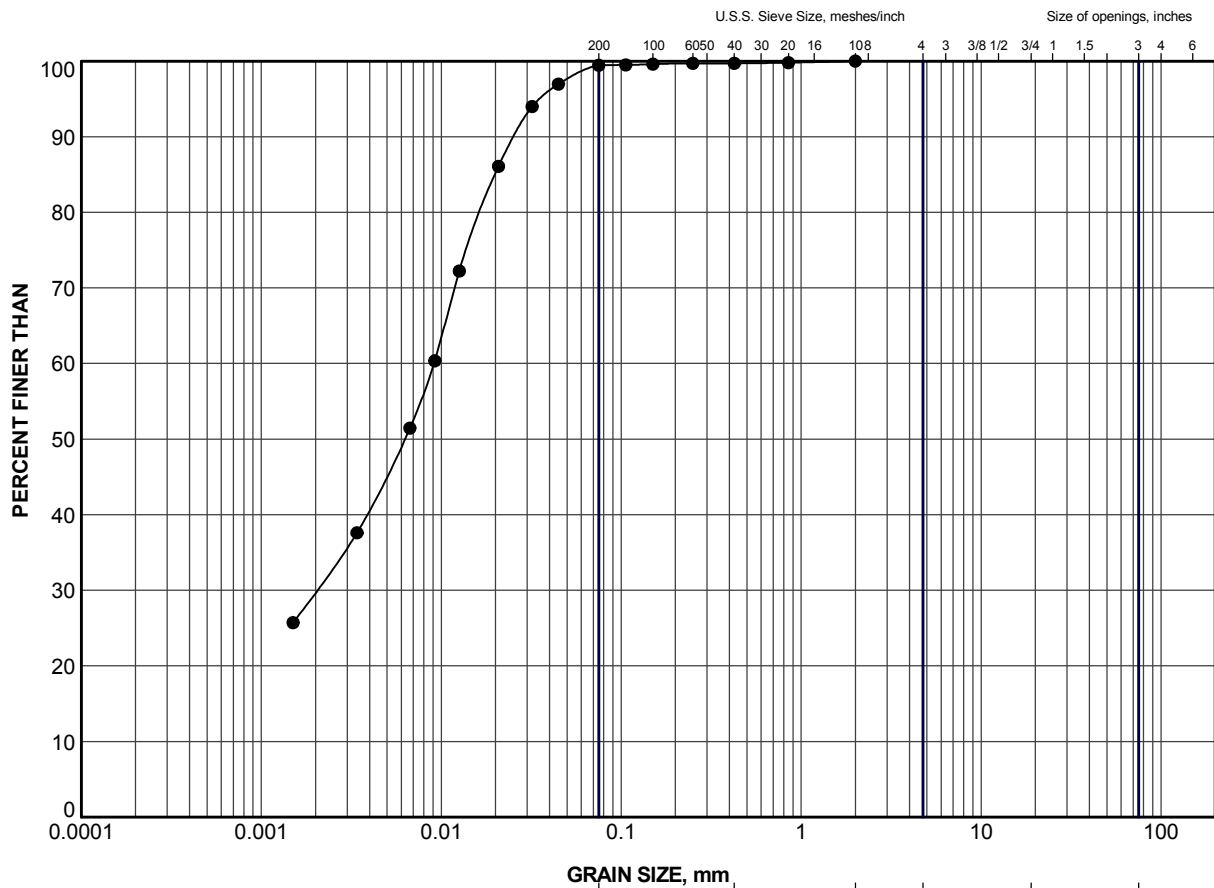
NO.	DATE	BY	REVISION	
Geocres No. 40J8-64				
HWY.	401	PROJECT NO.	13-1132-0111	DIST.
SUBM'D.	NG	CHKD.	NG	DATE:
DRAWN:	WDF	CHKD.	DUP	Nov. 20/14
			APPD.	FUH
			DWG.	2





APPENDIX A

Laboratory Test Data – Soils



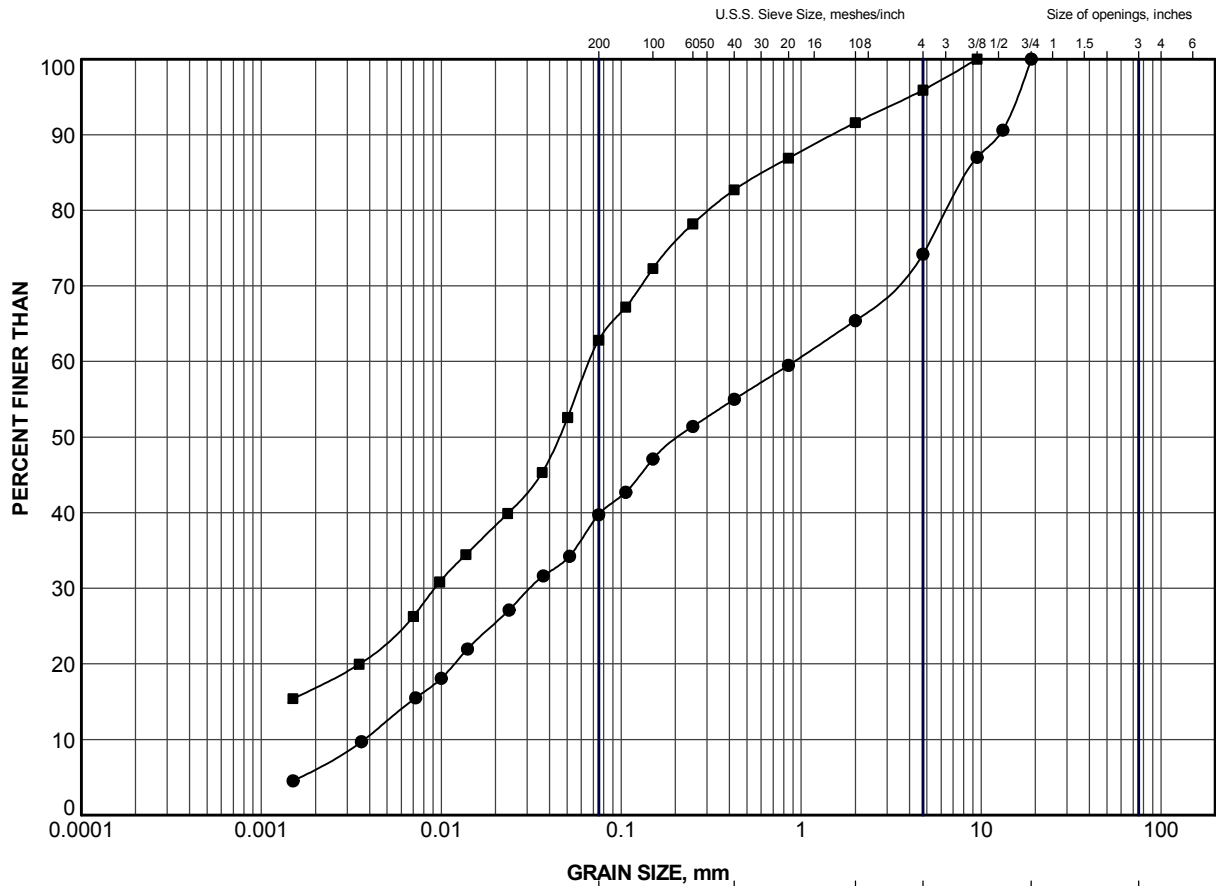
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	204	19	160.7

PROJECT				LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234 HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		13-1132-0111		FILE No.1311320111-1000-R020A2			
DRAWN		WDF		Aug 06/14		SCALE N/A REV.	
CHECK						FIGURE A-2	





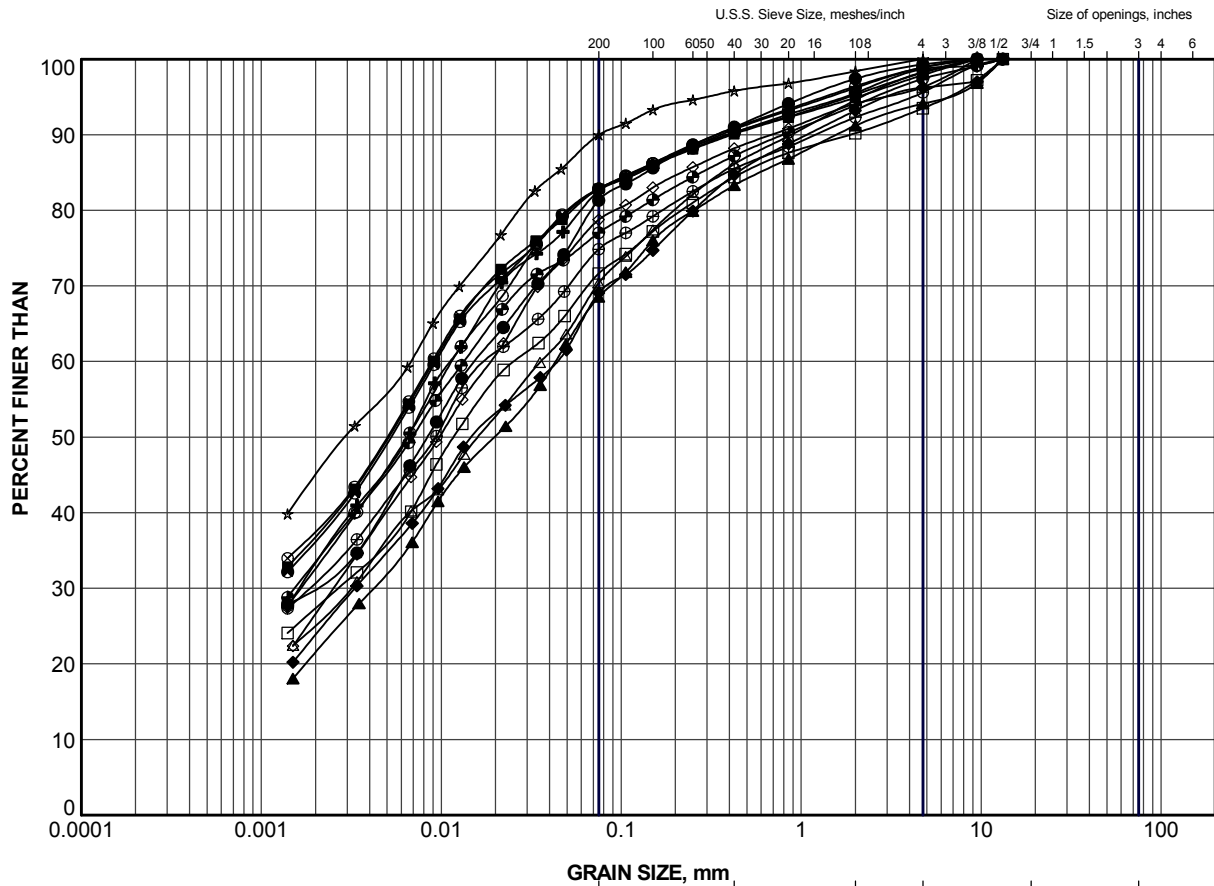
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	20	158.9
■	205	4	180.4

PROJECT				LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234 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-R020A3			
DRAWN		WDF		Aug 06/14		SCALE N/A REV.	
CHECK						FIGURE A-3	




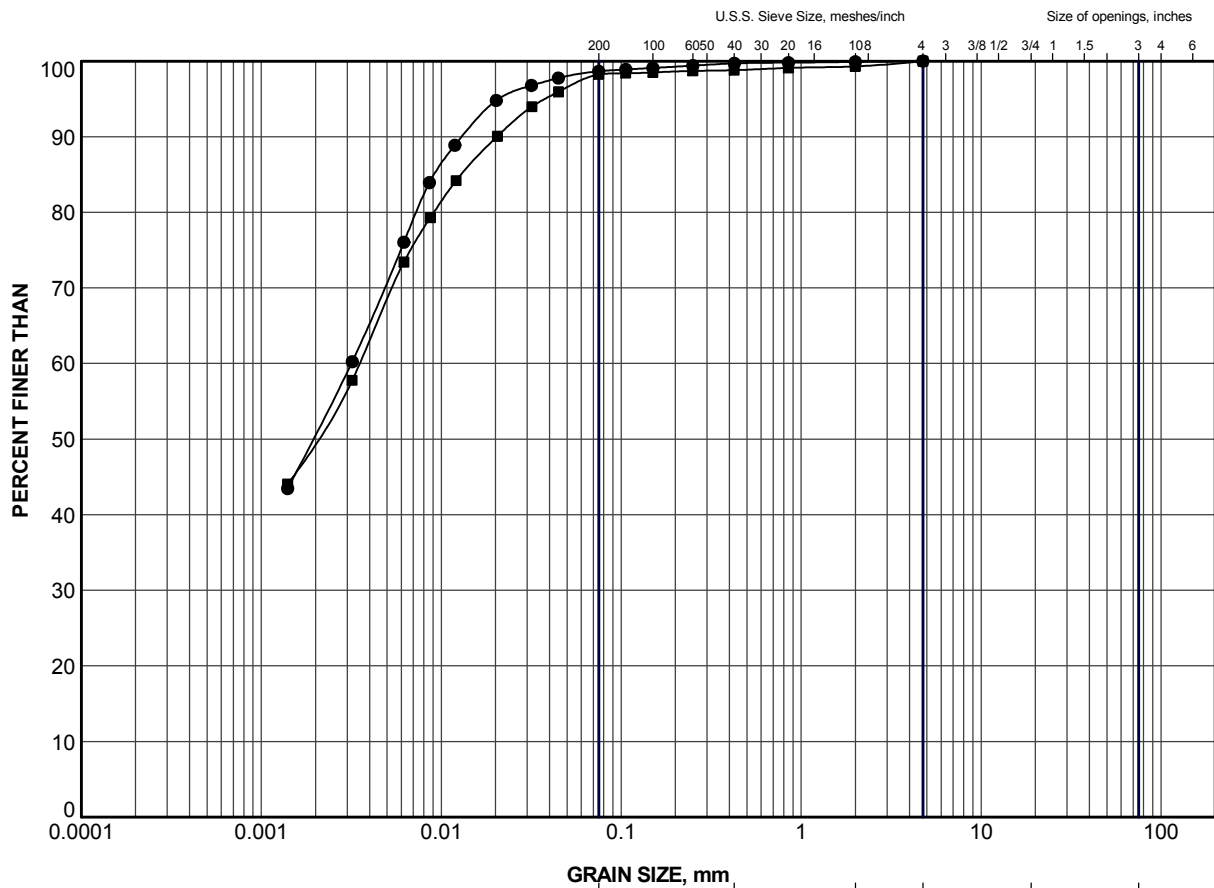


CLAY AND SILT	GRAVEL SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	201	4	179.9
■	201	7	176.8
▲	202	4	179.6
+	202	8	176.6
◆	202	12	171.1
◇	203	6	179.4
○	203	8	177.9
△	204	4	179.9
⊗	204	6	178.4
⊕	204	11	172.9
□	204	14	168.4
⊙	205	9	176.5
⊗	205	11	173.4
★	205	15	167.3

PROJECT			
LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234 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-R020A4	
DRAWN	WDF	Aug 06/14	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-4



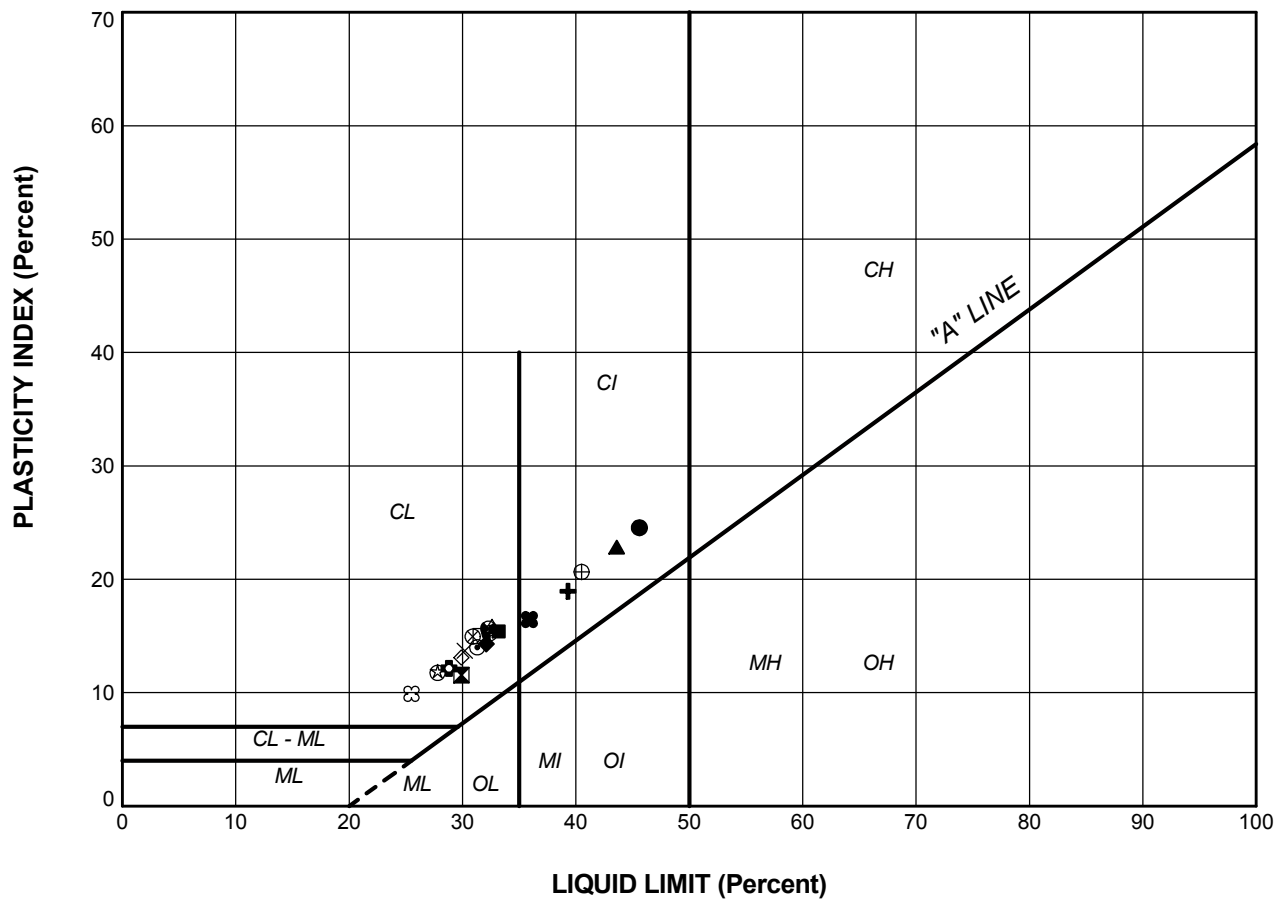
CLAY AND SILT	GRAVEL SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	202	16	165.0
■	205	18	162.8


PROJECT				LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234 HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.		13-1132-0111		FILE No.1311320111-1000-R020A5			
DRAWN		WDF		Aug 06/14		SCALE N/A REV.	
CHECK						FIGURE A-5	





LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI	
●	202	1	45.6	21.1	24.6	(FILL)
■	203	3	33.2	17.8	15.4	(FILL)
▲	204	1	43.6	20.8	22.9	(FILL)
+	205	2	39.3	20.4	19.0	(FILL)
◆	201	4	32.1	17.8	14.3	
◇	201	7	29.9	16.8	13.1	
○	202	4	27.8	16.1	11.8	
△	202	8	32.6	16.7	15.9	
⊗	202	12	30.9	16.0	15.0	
⊕	202	16	40.5	19.9	20.7	
□	203	6	31.6	16.6	15.0	
⊙	203	8	32.3	16.7	15.7	
⊛	204	6	32.4	17.2	15.3	
☆	204	11	27.8	15.9	11.9	
⊗	204	14	25.5	15.6	9.9	
⊗	204	19	29.9	18.4	11.6	
⊗	205	9	31.3	17.3	14.0	
⊗	205	11	28.8	16.7	12.2	
×	205	15	30.2	16.5	13.7	
■	205	18	35.9	19.5	16.5	

PROJECT			
LUCAS DRAIN BRIDGE REPLACEMENT, SITE 13-234 HIGHWAYS 401 & 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE			
PLASTICITY CHART			
PROJECT No.		13-1132-0111	FILE No.1311320111-1000-R020A6
DRAWN	WDF	Aug 06/14	SCALE N/A REV.
CHECK			
 Golder Associates LONDON, ONTARIO			FIGURE A-6



APPENDIX B

Laboratory Test Data – Rock

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS **ASTM D7012**

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1132-0111	SAMPLE NUMBER	25
BOREHOLE NUMBER	204	SAMPLE DEPTH, m	28.42-28.52

TEST CONDITIONS

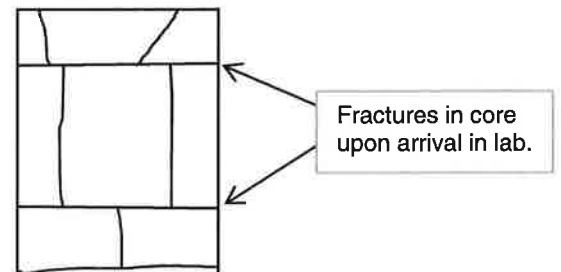
MACHINE SPEED, mm/min	0.00	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	1.95

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.20	WATER CONTENT, (specimen) %	0.07
SAMPLE DIAMETER, cm	4.71	UNIT WEIGHT, kN/m ³	24.02
SAMPLE AREA, cm ²	17.42	DRY UNIT WT., kN/m ³	24.01
SAMPLE VOLUME, cm ³	160.30	SPECIFIC GRAVITY	-
WET WEIGHT, g	392.82	VOID RATIO	-
DRY WEIGHT, g	367.36		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRENGTH, MPa	56.2
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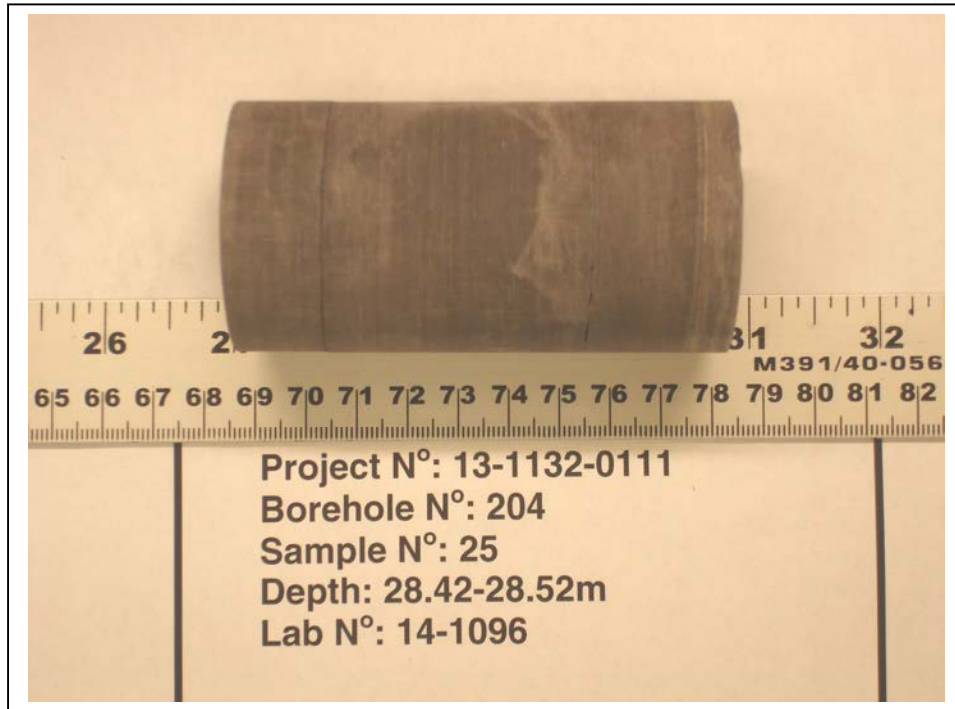
REMARKS:	L/D Ratio not in accordance with ASTM Standard	DATE:	6/10/2014
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Checked By: *fo*

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE B-1B



BEFORE COMPRESSION



AFTER COMPRESSION

Date June 11, 2014
Project 13-1132-0111

Golder Associates

Drawn Frank
Chkd. *lv*



APPENDIX C

Rock Core Photograph



APPENDIX C
ROCK CORE PHOTOGRAPH



Photograph 1: BH204 Elevation 156.34 to 152.77 metres.



APPENDIX D

Site Photographs



APPENDIX D SITE PHOTOGRAPHS



Photograph 1: East elevation of existing bridge, facing south.



Photograph 2: West side of existing bridge, Borehole 205 location, facing north.



APPENDIX D SITE PHOTOGRAPHS



Photograph 3: Highway 40, facing north.

n:\active\2013\1132-geo\1132-0100\13-1132-0111 dillon-gwp 3093-09-00-hwy 401-40\ph 1000-fdns\rpts\r02 hwy 40 & lucas dr bridge\1311320111-1000-r02 jul 31 15 (final) app d - site photos.docx



APPENDIX E

Special Provisions - Lightweight Fill Materials

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.

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:

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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

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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.

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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.

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- 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.

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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.

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