



February 2015

FOUNDATION INVESTIGATION AND DESIGN REPORT

Retaining Wall at E-N/S Ramp, Site 13-525/W
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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Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	2
2.1 General.....	2
2.2 Site Geology.....	2
2.3 Previous Site Construction History.....	3
3.0 INVESTIGATION PROCEDURES.....	4
4.0 SUBSURFACE CONDITIONS.....	6
4.1 Site Stratigraphy.....	6
4.1.1 Topsoil.....	6
4.1.2 Fill.....	6
4.1.3 Clayey Silt Glacial Till.....	7
4.1.4 Silty Clay Glacial Till.....	7
4.1.5 Clayey Silt.....	7
4.1.6 Sandy Silt Glacial Till.....	8
4.1.7 Sand and Silty Fine Sand.....	8
4.1.8 Sand and Gravel.....	8
4.1.9 Silt.....	8
4.2 Methane Gas.....	9
4.3 Piezo-Cone Penetration Testing.....	9
4.4 Groundwater Conditions.....	9
5.0 MISCELLANEOUS.....	11

PART B - FOUNDATION DESIGN REPORT

6.0 ENGINEERING RECOMMENDATIONS.....	12
6.1 Proposed Design Options.....	12
6.2 Backfill and Settlement Considerations.....	16
6.3 Foundations.....	18
6.3.1 Shallow Foundations.....	18
6.3.2 Deep Foundations.....	20



FOUNDATION INVESTIGATION AND DESIGN REPORT RETAINING WALL AT E-N/S RAMP, SITE13-525/W

6.4	Stability	21
6.5	Lateral Earth Pressures	22
6.6	Settlement	23
6.7	Additional Design Considerations	25
6.7.1	RSS Walls	25
6.7.2	Drainage	25
6.7.3	Permanent Deadmen or Soil Anchors	25
6.7.4	Other Design Considerations	26
6.8	Construction Considerations	26
6.8.1	Excavations and Groundwater Control	26
6.8.2	Driven or Drilled Soldier Piles and Sheet Piles	27
7.0	MISCELLANEOUS	28

TABLE I - Comparison of Retaining Wall Alternatives

LIST OF SYMBOLS

LIST OF ABBREVIATIONS

RECORD OF BOREHOLE SHEETS

RECORD OF CONE PENETRATION TEST

FIGURE 1 - Key Plan

FIGURE 2 - Summary of Subsurface Test Data Borehole 603/CPT 603

FIGURE 3 - Former Location of McGregor Creek, Lucas Drain and Meander Channel

DRAWING 1 - Borehole Locations and Soil Strata

APPENDICES

APPENDIX A

Laboratory Test Data – Routine Soils

APPENDIX B

Laboratory Test Data – Consolidation Testing

APPENDIX C

Slope Stability Analyses

APPENDIX D

Special Provisions - Lightweight Fill Materials



PART A

FOUNDATION INVESTIGATION REPORT

RETAINING WALL AT E-N/S RAMP, SITE 13-525/W
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
CHATHAM-KENT, ONTARIO
GWP 3093-09-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 3093-09-00. The project involves the detail design for the reconfiguration of the Highway 401 and Highway 40 (Communication Road) interchange as well as the realignment of Pinehurst Line and reconstruction of the Highway 401 eastbound lanes. This report addresses the construction of the retaining wall along the E-N/S Ramp, Site No. 13-525/W.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed 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 was provided by the MTO.



2.0 SITE DESCRIPTION

2.1 General

The Highway 401/Highway 40 interchange is located in the Municipality of Chatham-Kent, Ontario. The location of the project is shown on the Key Plan, Figure 1.

This section of Highway 401 is currently a four lane divided highway oriented generally east-west. Highway 40 is oriented in a generally northwest-southeast direction in the area of the site. For the purposes of this report, Highway 40 is assumed to be oriented in a north-south direction. It is proposed to construct new E-N/S and S-W ramps in the northeast quadrant of the interchange. This area of the site comprises the McGregor Creek floodplain. In order to restrict the embankment width and prevent the embankment footprint from encroaching on McGregor Creek, a retaining wall is to be constructed along the north side of the E-N/S Ramp to retain the embankment fills. The ground surface in the area of the proposed retaining wall varies from about elevation 180.7 to 181.5 metres.

2.2 Site Geology

This project lies within the physiographic region of southwestern Ontario known as the Bothwell Sand Plain which formed a delta geologic precursor of the Thames River where it discharged into the former glacial Lake Warren. The Bothwell Sand Plain primarily consists of a thin layer of sand, approximately 1 metre thick, over the clay floor.¹

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

The project area is also about 2 kilometres north and 6 kilometres west of an area mapped as "till moraine". Although the mapping provides a general indicator of the geologic conditions of the site, these maps only address the most recent phase of the region's glacial geology based on near-surface materials and may not characterize the geologic complexity of the site at greater depths. In southwestern Ontario, the most significant prehistoric glacial features are associated with the last advance and retreat of ice through the area. As the ice receded from the region, a number of moraines and lakes were formed near the retreating ice front. In some areas, such as Windsor and Wallaceburg, Ontario, the clayey silt or silty clay deposits have a grain size

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.



distribution consistent with that of a cohesive glacial till although the density and strength of the materials are not consistent with deposition below a grounded ice sheet as commonly assumed for materials described as glacial till. In the Windsor area, much of the soils described as glacial till were likely deposited from the underside of floating ice through a shallow water depth as a diamict (broadly graded mud) and, therefore, the soil carried little or no weight of the overlying ice. East of Windsor, toward the Chatham area, some areas of the ice sheet may have been grounded and produced hard cohesive glacial till, while in other areas the ice may have been floating or partially floating which has resulted in complex conditions in some areas. Near moraines, geologic conditions can be especially complex because of highly localized outwash (sand and gravel) deposits, silt and clay deposited in local ice-proximal lakes and ponds, and comparatively short duration re-advances and retreats of the former ice sheets.

2.3 Previous Site Construction History

This site was extensively altered in the early 1960s when the Highway 401 and Highway 40 interchange was constructed. Prior to construction of the interchange, McGregor Creek and Lucas Drain both existed along previous channel alignments. Aerial photographs from the mid-1950s also indicate that McGregor Creek meandered in the project area producing at least two former and subsequently buried meander channels with one on each of the south and north sides of the present channel. During construction of the interchange, both McGregor Creek and Lucas Drain were realigned to their present positions. The available evidence also 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 3 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 June 4 and 13, 2014 during which time four boreholes, numbered 601 to 604, were drilled at the location of the proposed retaining wall. The boreholes were supplemented with two boreholes, numbered 605 and 606, advanced for the E-N/S Ramp high fill embankment (Geocres Report No. 40J8-61). The borehole locations are shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations, and borehole depths.

Borehole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
601	4 693 861	338 356	180.44	14.17
602	4 693 846	338 407	180.40	11.13
603	4 693 847	338 294	180.57	16.92
604	4 693 800	338 254	180.71	19.96
605	4 693 783	338 260	181.54	5.79
606	4 693 848	338 326	180.49	8.08

The investigation was carried out using track mounted drilling equipment (CME 75) supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.76 and 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of ASTM D1586. Thin-walled Shelby tube samples were obtained in the boreholes at selected depths in accordance with ASTM D1587. Piezo-Cone Penetration Testing (CPT) was carried out adjacent to borehole 603 in accordance with ASTM D5778.

The recorded SPT N values are noted on the Record of Borehole sheets. According to ASTM D1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres, after an initial 150 millimetres of penetration. In cases where it was not possible to achieve a full 450 millimetres of drive, a penetration resistance representing the number of blows to drive the sampler is recorded on the Record of Borehole. The penetration resistance obtained in the first 150 millimetres is normally neglected unless the sampler could only be driven 150 millimetres or less, in which case SPT testing was terminated after 100 blows. The results of the SPT testing as presented on the Record of Borehole sheets and in Section 4 are unmodified (not standardized for hammer efficiency, borehole diameter, rod length, etc.). The samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore, particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes including cobbles and boulders are known to be present in the glacial till deposits as discussed in the text of this report.



FOUNDATION INVESTIGATION AND DESIGN REPORT RETAINING WALL AT E-N/S RAMP, SITE13-525/W

The boreholes were terminated between 5.8 and 20.0 metres below the existing ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations. Groundwater observation piezometers were installed in boreholes 602, 604 and 606 as indicated on the corresponding Record of Borehole sheets. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder staff who also located the boreholes in the field, monitored the drilling, sampling, and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in labelled containers, and transported to our London and Mississauga laboratories for further examination and testing. Index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations were carried out on selected soil samples. Consolidation testing was carried out on a selected Shelby tube sample. The results of the testing are shown on the Record of Borehole sheets and in Appendices A and B.

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

The CPT consisted of pushing a 35 millimetre outside diameter cone from a known depth to refusal at an approximate rate of 20 millimetres per second by the drill rig in accordance with ASTM D5778. Electronic sensing elements connected to the CPT probe continuously measure tip resistance, local side friction and pore water pressures with depth. The measurements are shown on the Record of Cone Penetration Test sheet and Figure 2.



4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendices A and B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The boreholes drilled at the site generally encountered surficial topsoil over fill materials underlain by clayey silt to silty clay glacial till which was, in turn, underlain by sandy silt till interbedded with sand, sand and gravel and silt layers.

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

4.1.1 Topsoil

Between 30 and 460 millimetres of topsoil was encountered at the ground surface in each of the boreholes. Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

4.1.2 Fill

Fill materials were encountered beneath the topsoil in each of the boreholes. The fill materials were between 2.4 and 4.0 metres thick and extended to between elevations 176.7 and 177.7 metres. The fill was variable in composition and comprised layers of sand, sandy silt, silty sand, sand and gravel, silty clay, clayey silt and silt. The lower 0.6 to 1.5 metres of fill materials in boreholes 602, 603, 605 and 606 consisted of dark grey to black sand and gravel to sand with pockets of clayey silt and varying amounts of wood, organic materials and other debris. Based on the site history, this uncontrolled fill material may have been placed during historic rerouting of McGregor Creek and highway interchange construction.

Measured N values in the fill materials ranged from 2 to 16 blows per 0.3 metres. Samples of the fill materials had water contents between 16 and 33 per cent. Atterberg limits determinations were carried out on three samples of the cohesive fill material, the results of which are provided on Figure A-7. The samples of cohesive fill tested had liquid and plastic limits ranging from 27 to 40 per cent and 18 to 19 per cent, respectively, and plasticity indices of 8 to 21, indicating low to intermediate plasticity. Grain size distribution curves for samples of the fill materials are shown on Figure A-1.



4.1.3 Clayey Silt Glacial Till

A deposit of firm to very stiff clayey silt glacial till was encountered beneath the fill materials in each of the boreholes between elevations 176.7 and 177.7 metres. Where fully penetrated in boreholes 601 to 604 and 606, the clayey silt till was between 3.2 and 6.1 metres thick. Boreholes 605 and 606 were terminated in the clayey silt till after penetrating the layer for 1.1 and 1.8 metres, respectively. Layers of sand and silty clay till were encountered within the clayey silt till in boreholes 601 and 606, respectively, as described below. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the clayey silt glacial till.

The clayey silt till had measured N values ranging from 6 to 30 blows per 0.3 metres. Samples of the clayey silt till had water contents ranging from 14 to 19 per cent. Five Atterberg limits determinations carried out on samples of the clayey silt till, the results of which are provided on Figure A-7, yielded liquid limits ranging from 29 to 32 per cent, plastic limits ranging from 16 to 17 per cent, and plasticity indices of 12 to 15 per cent, indicating low plasticity. Grain size distribution curves for samples of the clayey silt till are shown on Figure A-2.

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

Borehole	Sample	Depth (m)	Effective Overburden Pressure (kPa)	Initial Void Ratio ¹	Compression Index, C _c	Recompression Index, C _r	Preconsolidation Pressure (kPa)
604	7	5.64	69	0.53	0.136	0.014	249

¹ Initial void ratio is taken at the in situ vertical effective stress.

4.1.4 Silty Clay Glacial Till

As noted above, a 1.8 metre thick layer of stiff to very stiff silty clay glacial till was encountered within the clayey silt till in borehole 606 at elevation 175.3 metres. Measured N values in the silty clay till were 13 and 20 blows per 0.3 metres. Although not specifically encountered in the boreholes, cobbles and boulders should be anticipated within the silty clay glacial till.

4.1.5 Clayey Silt

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



4.1.6 Sandy Silt Glacial Till

Underlying the clayey silt glacial till in boreholes 601 and 602 and the clayey silt in boreholes 603 and 604, a deposit of compact to very dense sandy silt glacial till was encountered. The sandy silt till was between 3.8 and 4.3 metres thick and was encountered between elevations 171.6 and 173.4 metres. The sandy silt till was interbedded with layers of sand and sand and gravel, as described below. The sandy silt till varied in gradation from silty sand to sand and silt, with varying amounts of gravel and clay. Cobbles and boulders should be anticipated within the glacial till deposits.

Measured N values from the sandy silt till ranged from 18 to greater than 100 blows per 0.3 metres. Samples of the sandy silt till had water contents between 8 and 11 per cent. Grain size distribution curves for samples of the sandy silt till are shown on Figure A-3.

4.1.7 Sand and Silty Fine Sand

Layers of sand were encountered within the clayey silt till in borehole 601, interbedded within the sandy silt till in boreholes 603 and 604, and below the sandy silt till in boreholes 602 and 604, between elevations 164.3 and 173.1 metres. Where fully penetrated, the sand layers were between 0.4 and 1.2 metres thick. Borehole 602 was terminated after penetrating a sand layer for 0.3 metres. The sand layer in borehole 603 was noted to be silty. Measured N values in the sand ranged from 14 to 92 blows per 0.3 metres and samples of the sand had water contents between 11 and 12 per cent. Grain size distribution curves for samples of the sand are shown on Figure A-4.

4.1.8 Sand and Gravel

Sand and gravel layers were encountered within the sandy silt till in borehole 603 at elevation 170.5 metres and beneath the lower sand layer in borehole 604 at elevation 163.3 metres. The sand and gravel layer in borehole 603 was 0.9 metres thick. Borehole 604 was terminated in the sand and gravel after penetrating the layer for 2.6 metres. The sand and gravel in borehole 603 had a single measured N value of 23 blows per 0.3 metres, and the sand and gravel in borehole 604 had measured N values of 100 blows per 0.3 metres and greater. Samples of the sand and gravel had measured water contents of 7 and 9 per cent. Grain size distribution curves for samples of the sand and gravel are shown on Figure A-5.

4.1.9 Silt

Very dense silt layers were encountered beneath the sandy silt till in boreholes 601 and 603 at elevations 167.3 and 163.8 metres, respectively. The silt layers were explored for 1.1 and 0.1 metres prior to terminating boreholes 601 and 603, respectively. Measured N values in the silt layers were 84 and 101 blows per 0.3 metres. A select sample of the silt had a water content of 22 per cent. A grain size distribution curve for a sample of the silt is shown on Figure A-6.



4.2 Methane Gas

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

4.3 Piezo-Cone Penetration Testing

During the CPT carried out adjacent to borehole 603, the cone was pushed from the ground surface through the existing fill materials, clayey silt till and clayey silt, and further penetration was refused at a depth of 8.1 metres, or elevation 172.5 metres. The tip resistance, local side friction and pore water pressures were measured over the depth penetrated. The stratigraphy within this depth range is presented on the Record of Borehole sheet 603 following the text of this report. This data was correlated to the preconsolidation pressure and undrained shear strength, water content and oedometer data from the borehole samples as shown in Figure 2 and discussed in further report sections.

4.4 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling and, in borehole 605, the water level in the open borehole was measured immediately following drilling. Piezometers were installed in boreholes 602, 604 and 606 as shown on the Record of Borehole sheets. Encountered and measured groundwater levels are summarized in the following tables.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level		Measured Groundwater Level Immediately Following Drilling	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
601	180.4	2.1	178.3	-	-
602	180.4	2.1	178.3	-	-
603	180.6	5.8	174.8	-	-
604	180.7	*	*	-	-
605	181.5	*	*	4.0	177.5
606	180.5	*	*	-	-

* Groundwater level not established during drilling.

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.



FOUNDATION INVESTIGATION AND DESIGN REPORT RETAINING WALL AT E-N/S RAMP, SITE13-525/W

Borehole	Ground Surface Elevation (m)	Measured Groundwater Elevation (m)			
		June 9/14	June 11/14	June 16/14	July 9/14
602	180.40	178.27	179.70	179.58	179.76
604	180.71	June 16/14 175.44	June 20/14 173.64	July 9/14 175.53	-
606	180.49	June 6/14 172.96	June 9/14 173.02	June 16/14 173.30	July 9/14 173.91

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. Groundwater seepage was observed during drilling of each borehole from the fill materials between 1.7 and 3.8 metres below the ground surface, or between elevations 177.7 and 178.9 metres, and from the deeper granular layers interbedded within the tills. Based on the soil colour change from brown to grey and the measured ground water elevations, the inferred groundwater level in this area is at about elevation 180 metres. There appears to be a confined sand and gravel aquifer at depth which was encountered at other locations within the Highway 401/Highway 40 interchange. The inferred groundwater level of this aquifer is elevation 175.5 metres. Groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.



5.0 MISCELLANEOUS

This investigation was carried out using equipment supplied and operated by Aardvark Drilling Inc., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Simon Lutz, under the direction of the Field Investigation Manager, Mr. David J. Mitchell. The CPT was supervised by Mr. Mrinmoy Kanungo, P.Eng.

Routine laboratory tests were carried out at Golder's London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. The consolidation testing was conducted in Golder's Mississauga laboratory under the supervision of Dr. J. Paul Dittrich, P.Eng. The Mississauga laboratory is a MTO registered laboratory in the specialty of soil and rock including testing for Foundation Engineering Low and High complexity.

This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by the Team Leader, Dr. Storer J. Boone, P.Eng., a senior geotechnical engineer and Principal with Golder. 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

RETAINING WALL AT E-N/S RAMP, SITE 13-525/W
HIGHWAY 401/HIGHWAY 40 INTERCHANGE RECONFIGURATION
CHATHAM-KENT, ONTARIO
GWP 3093-09-00
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION



6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides our recommendations on the foundation aspects of the design of the retaining wall at the E-N/S Ramp, Site No. 13-525/W, to be constructed in conjunction with the Highway 401 / Highway 40 interchange reconfiguration. The recommendations are based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

6.1 Proposed Design Options

The proposed E-N/S Ramp is to be constructed in the McGregor Creek floodplain, south of the creek. In order to restrict the embankment width and prevent the embankment footprint from encroaching on McGregor Creek, a retaining wall is to be constructed along the north side of the ramp to retain the embankment fills. The wall is to extend from approximate Station 10+240 to 10+410. The existing ground surface elevation along the length of the wall varies from about 180.7 to 182.9 metres. The proposed top of wall elevation ranges from about 184.6 to 190.4 metres and, therefore, the preliminary proposed wall heights may vary from about 6 to 10 metres.

It has been indicated by Dillon that four primary design options have been considered:

- a Retained Soil System (RSS) type wall;
- a soldier pile and lagging type wall with concrete facing using tiebacks where cantilever walls are not feasible;
- a steel sheet pile wall using tiebacks where cantilever walls are not feasible; and
- a cast-in-place concrete cantilever wall.

Combinations of these have also been considered. The choice of retaining wall type will be dependent upon four primary concerns:

- 1) removal of existing uncontrolled fill materials;
- 2) settlement tolerance;
- 3) backfill (embankment fill) choices;
- 4) total height; and
- 5) long-term performance considerations established by MTO.

This proposed retaining wall is located in a flood plain and the base of the wall will also require protection from scour. In this particular case, there is typically about 2.5 to 4 metres of uncompacted and uncontrolled (debris) fill along the retaining wall alignment that is unsuitable to support the retaining wall and its backfill. Therefore, while scour protection will be needed, wall foundations will be embedded at least 2.5 to 4 metres below the



existing ground levels to the interface of the underlying clayey silt till or on new engineered fill built on the native soils. If the existing ground surface elevations are to be re-established, this embedment depth may partially or fully address scour protection issues pending a more detailed evaluation to be completed by others.

As an alternative to full removal of the existing fill materials, consideration may be given to constructing rammed aggregate piers (RAP) through the fill materials beneath the retaining wall footings and, in the case of an RSS wall, beneath the mass of reinforcing backfill. Benefits of a RAP system could include time and cost savings as compared to full removal of existing fills, a reduction in the volume of spoil to be handled/removed from the site, potential savings to the project schedule and accelerated consolidation settlement associated with the effects of grade raising. A RAP system should be designed and constructed by a geotechnical contractor specializing in earth improvement works.

A comparative summary of the advantages/disadvantages, relative costs and risks/consequences associated with each type of retaining wall is presented in Table I, attached. The various retaining wall options are briefly discussed in the following report sections.

Retained Soil System Wall

A RSS wall is considered to be feasible from a geotechnical perspective. Suitable facing options include a modular block wall (MBW), articulated concrete panels and concrete cast-in-place panels. Wet-cast MBW facing panels are preferred to dry cast facing due to their significantly lower water absorption capacity and higher freeze-thaw durability. Full-height pre-cast panels should be avoided since they are least accommodating of settlement. Reinforcement can be in the form of galvanized or epoxy coated steel strips or grids or synthetic geogrids. Depending on the RSS system chosen, the full range of wall height from 6 to 10 metres can be erected using the same design. This wall type is the most accommodating of settlement. RSS walls are proprietary systems which are to be designed by the supplier and constructed in accordance with their specifications. The internal stability of these mechanically-reinforced soil walls should be verified by the RSS supplier/designer. If a RSS wall is selected, the geotechnical aspects of the global stability of the detailed retaining wall design should be reviewed prior to construction. RSS walls can generally be constructed rapidly using small equipment and are typically cost-effective compared to other walls systems. Drainage of the granular backfill behind RSS walls should be provided to ensure stability. The performance and appearance of the RSS wall should be specified in the Contract Documents.

A RSS wall may be designed such that the facing elements are constructed either on a granular levelling pad or a concrete leveling pad. Depending on the design selected by the RSS supplier and if suitable native soils exist at appropriate elevations, it may not be necessary to provide 1.2 metres of frost cover or thermal equivalent; however, the foundations must have adequate embedment to provide a stable structure and protection against scour.

The MTO's terms of reference for this project indicates that a RSS type structure is not permitted below the maximum flood elevation of a 1 in 250 year event plus 0.5 metres, or about elevation 183.0 metres. Other North American transportation agencies permit use of RSS adjacent to shorelines and within floodplains provided the following items are accounted for in the design.



- Scour protection – The reinforced soil mass must be founded at a depth sufficient to prevent damage by scour or measures to protect against scour (and wave action, if applicable) must be provided in front of the wall. Given that the existing fill materials are unsuitable for support of retaining walls, any RSS wall should have its leveling pad and reinforced backfill supported by the native clayey silt glacial till found at elevations ranging from about 177 to 178 metres. If the existing ground surface elevations of about 180 to 181 metres in front of the wall are to be re-established after wall construction, the resulting embedment depth may be sufficient for scour protection. If additional scour protection is required, backfilling in front of the RSS wall foundation with suitable rip-rap and crushed stone products could provide additional protection.
- Hydrostatic pressure and seepage forces – During flood conditions, the groundwater level may rise and floodwaters may seep behind the wall. During or after the floodwaters have subsided, the groundwater level behind the wall may remain temporarily elevated compared to the receding flood surface depending on backfill materials, overall site topography and wall geometry. The resulting hydrostatic pressures and seepage forces should be assessed and accounted for in the design. In lieu of a detailed assessment, a drainage system should be incorporated into the wall design to collect groundwater and seepage flows resulting from flood conditions. The global slope stability of the RSS wall should be assessed using the most severe combination of groundwater flow, creek elevation, rapid drawdown and applicable surcharge loading.
- Rapid drainage – The wall design should allow free flow of water through the wall. The wall should be backfilled with free-draining materials with less than 5 per cent fines.
- Durability of facing units – The facing units must resist degradation by freeze-thaw events and ice action during flood conditions.
- Buoyancy – The design should be based on effective stress geotechnical engineering parameters and the buoyant unit weight of the facing elements, foundation soil, retained fill and drainage fill, as applicable for a flood design case.
- Corrosion and environmental degradation – The reinforcing elements could be subject to corrosion or environmental degradation due to periodic submersion during flood events. The reinforcing elements should be selected to provide effective long-term performance in such an environment.

If the use of a RSS wall is not permitted, the wall might be constructed using a traditional cast-in-place concrete or other wall systems below the maximum flood elevation, with a RSS-type structure above the maximum flood elevations noted above. Use of cast-in-place walls is, however, dependent upon implementing measures to control differential settlement.

Removal of the existing fill materials may require use of temporary excavation support systems to minimize the effects of construction on the adjacent floodplain. In this case, it may be advantageous to utilize a steel sheet pile system for the temporary excavation support and leave the sheet piles in place permanently to address scour and flood protection for the retaining system. If the sheet pile remains in place, a horizontal off-set distance of about 1 metre may be required between the walls to allow for construction access if needed. Further, the RSS wall design should not rely upon the sheet piles for lateral support. If the sheet piles are left in place, the base of RSS wall (reinforced mass and leveling pad) should be no higher than about 2 metres above the elevation of the native soil and engineered fill interface. Any space between the face of the RSS wall and the sheet piling should be backfilled with a free-draining granular fill consistent with the materials recommended for



retaining wall back fill as described below in Section 6.5. Depending on the distance between the two walls, compaction may not be possible. In such case, the fill between the walls could consist of clear, crushed stone products with less than 5 per cent passing the 0.075 millimetre sieve where this material is placed without compaction. Some settlement of the fill material should be expected.

Soldier Pile and Lagging Wall

Soldier pile and lagging walls consist of H-piles placed in pre-drilled holes or driven at regular intervals along the wall line with lagging inserted behind the front pile flanges as fill placement proceeds. The lagging for permanent walls typically consist of pre-cast concrete panels to resist the load of the retained backfill and transfers it to the piles. At this particular site, a soldier pile and lagging wall can be designed as a cantilever wall to an effective height of about 5 metres depending on the planned fill materials. Above this height, additional lateral support from soil anchors or deadmen anchors will be required. Although the wall itself may not be adversely affected by settlement, the sequence of fill placement and ground anchor/deadman construction must be carefully considered to avoid damage to the restraining elements by fill settlement or construction activities. Differential settlement of deadmen or anchors as compared to the wall face could result in either overstressing or relaxation of the ties depending on the numbers of ties and their elevations. Soldier pile and lagging walls are generally less expensive compared to RSS and cantilever walls but can typically be constructed in a shorter time period compared to cast-in-place cantilever walls. While soldier pile walls are generally used for temporary excavation protection, the design may be adapted for permanent construction.

At this location, the soldier piles will derive their lateral toe resistance from embedment into the native clayey silt glacial till. The existing fill (top 2.5 to 3 metres) cannot be considered reliable for lateral support. As discussed in a subsequent section of this report, the vertical weight of the existing fill may be used in calculations for passive resistance to lateral loads from underlying native soils but should not itself be considered as providing any passive support. The existing fill materials are also not suitable for support of permanent lagging systems and the lagging should be structurally connected such that all vertical lagging loads are carried by the soldier piles. A soldier pile and lagging wall at this location will also experience periodic inundation at the base during flood conditions. As such, the wall should be designed to accommodate the resulting seepage and scour.

Reinforced Concrete Cantilever Wall

Construction of a cast-in-place concrete cantilever wall is considered to be marginally feasible from a geotechnical perspective and is also likely the most costly of the options. A concrete cantilever wall may be constructed with shallow or deep foundations. However, shallow foundations are preferred if settlement tolerances permit. This wall type is the most sensitive to damage if total and differential settlements along the alignment exceed design values. A concrete cantilever wall may require increased foundation construction duration compared to RSS and soldier pile walls. A concrete retaining wall will provide a barrier to drainage. It is imperative that drainage be provided through the wall to eliminate water build up in the granular backfill. It is anticipated that cast-in-place concrete footings for a concrete cantilever wall may be founded on the native soils as described in the following sections.



6.2 Backfill and Settlement Considerations

The retaining wall will be subject to settlement induced by the retaining wall backfill and the abutting ramp embankment fill. A summary of settlement estimates is provided in a subsequent section of this report. In general, settlement at this site will be largely driven by the placement of ramp embankment fill. If there is potential to place the embankment fill during an early construction phase, it may reduce post-filling settlement to the degree that cast-in-place concrete retaining walls are appropriate; however, an early fill placement stage will also require subsequent excavation through the new fill for retaining wall construction and such sequencing may not be practical or cost-effective.

Of the retaining wall options discussed above, the RSS system is the most practical and settlement tolerant of all walls and would be the best solution from a foundation engineering perspective. In some cases, a two-stage RSS wall system can be used whereby the pre-cast concrete panels are installed a period of time after the full reinforced soil height is constructed to result in greater tolerance for construction-phase and post-construction settlement. The two-stage RSS system would be the best alternative for this site to limit the influence of settlement on the finished work.

If a permanent soldier pile and lagging wall is selected, resistance to lateral loads for wall heights greater than about 4 to 5 metres will require construction of “deadmen” or other anchors within the new embankment fill. Provided that the existing uncompacted and uncontrolled fill is removed, settlement of the embankment associated with the underlying native soil conditions could have adverse effects on tie rods or other anchor systems. Similar to the RSS systems, there may be advantages to placing embankment fill early to induce settlement prior to retaining wall construction. However, retaining wall construction may require removing some portion of the new embankment fill. If the existing uncontrolled fill is not removed from beneath the new embankment, unpredictable settlement could exacerbate potential settlement-related problems with any wall anchors as well as result in vertical shear forces at the boundary between the back side of the retaining wall and backfill. To eliminate use of anchors, lightweight fill materials, discussed below, could be used to reduce lateral loads.

Based on settlement estimates, as discussed in a subsequent report section, it is unlikely that a cast-in-place concrete retaining wall would be selected as the preferred option unless a RSS wall is prohibited for other reasons. At the time this report was prepared, several options were under consideration with the lower half of the wall constructed as a conventional reinforced concrete or soldier pile and lagging wall and a RSS system constructed above the design flood levels. If such a composite system is to be constructed, it may be necessary to either preload the embankment and retaining wall area with the embankment fill (as discussed above) or utilize lightweight materials for backfill to minimize the effects of embankment-induced settlement.

A variety of lightweight fill materials could be used to address the issues described above:

- 1) **Blast Furnace Slag:** Water-cooled blast furnace slag has been used for MTO highway projects in situations where lightweight fill is required. This material has a design unit weight typically ranging between about 12 to 14 kilonewtons per cubic metre (kN/m^3) and relatively high angles of internal friction comparable to conventional granular backfill materials. Blast furnace slag may not be acceptable based on either environmental or other chemistry considerations (e.g., interaction with steel reinforcing for RSS walls). Chemical and physical properties of readily available slag fill should be examined prior to concluding designs. Blast furnace slag is susceptible to crushing if over compacted. A non-standard special provision



(NSSP) for lightweight fill material has been included in Appendix D which discusses construction methods and means of preventing overcrushing. If lightweight blast furnace slag is used fully within the backfill zone behind retaining wall defined by the “active” earth pressure zone, the lateral earth pressures on the wall may be reduced to about half the values ordinarily produced when conventional granular backfill is used.

- 2) **Expanded Polystyrene Blocks:** Expanded polystyrene (EPS) is commonly used to form lightweight embankments for MTO projects in Ontario. The EPS materials typically exhibit design unit weights of less than 1 kN/m^3 . Because the unit weight of EPS is less than that of water and due to the proximity of the retaining wall to the watercourse, the buoyancy of the EPS blocks must be considered during design. The effects of fluctuating groundwater and flood water levels due to the 100-year flood event, or other extreme event as recommended by a hydraulic or river engineer should be evaluated. These effects would include flotation, long-term water absorption and horizontal sliding due to unbalanced water levels. The EPS blocks must be protected against buoyancy effects both during and after construction. If EPS is used fully or partially within the backfill zone behind retaining wall defined by the “active” earth pressure zone, the lateral earth pressures on the wall may be greatly reduced.
- 3) **Cellular Concrete:** Use of cellular concrete as a lightweight backfill material has become increasingly cost-effective for construction projects in Ontario. This material has a design unit weight typically ranging between about 4 and 7 kN/m^3 and unconfined compressive strengths ranging from 0.5 to 3 megapascals (MPa). Cellular concrete is initially placed in a near-liquid state in lifts of about 400 to 600 millimetres thick. Cellular concrete has been accepted by MTO for general void-filling or mass-fill applications but has not yet been accepted for applications in which the cellular concrete forms a structural component of a retaining wall system (e.g., for lateral load or shear resistance). Until such time that the MTO has determined that cellular concrete and its associated quality control methods are suitable for RSS systems, cellular concrete should not be used as part of the RSS structural backfill.
- 4) **Tire Derived Aggregate (TDA):** Relatively new to the MTO, TDA consists of scrap tires shredded to strips 50 to 300 millimetres in length. TDA has been successfully used as engineered fill in the United States since the early 1990s and, more recently, in eastern provinces. In 2012, the MTO completed construction of widened embankments for the Highway 401/Boundary Road East underpass near Cornwall, Ontario. This material has a unit weight in the range of 8 kN/m^3 . In addition to its light weight, advantages related to TDA include environmental benefits associated with diversion of scrap tires from landfills, thermal insulation properties limiting frost penetration, and it is free-draining. However, scrap tires are considered designated waste in Ontario and a certificate of approval from the Ministry of Environment and Climate Change is required for its use. Due to the potential for chemical pollution leaching out of the tires, an impervious seal is required on all sides of the TDA and pre- and post-construction monitoring of ground water for chemical pollutants is required. Due to this risk, TDA is not considered suitable for use on sites with environmental sensitivities. Further, 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.

For this project, extensive use of lightweight fill may not be practical or necessary. However, for design and comparison purposes, the volume (thickness) of lightweight fill may be estimated based on the settlement magnitudes being directly and linearly proportional to the total vertical load generated by a combination of lightweight and conventional granular fill materials as compared to the total vertical load generated by embankments fully constructed using conventional granular fill materials and the settlement estimates provided



in this report. Non-standard special provisions for use of the lightweight fill materials are provided in Appendix D if needed. If EPS is to be used, a concrete pad should be constructed on top of the EPS followed by construction of the pavement section such that a final cover thickness of about 1 metre is provided over the EPS. The concrete pad is recommended to minimize the potential for reflective cracking of the pavement associated with joints between EPS blocks. Further, all lightweight fill materials should be covered by a minimum of 1 metre of new controlled fill materials along embankment slopes. Unless the entire embankment for both new ramps in the immediate vicinity of the retaining wall are constructed of lightweight fill, differential settlement could be induced by the presence of and transitions between embankment areas built using lightweight materials as compared to areas built with conventional engineered earth fill materials. In this case, potential optimized designs should be developed based on detailed three-dimensional settlement models, permissible settlement criteria, and alternative fill geometry schemes. If lightweight fill is utilized, the effect on sliding and overturning resistance and wall stability should be considered.

6.3 Foundations

6.3.1 Shallow Foundations

A concrete cantilever wall may be founded on strip footings constructed on the native soils subject to the settlement considerations discussed above. Footings for a RSS wall may be founded on a compacted Granular A leveling pad at least 300 millimetres thick. The fill materials present at the site are not suitable for support of a cantilever or RSS wall. It is anticipated that the wall may be founded either on the native clayey silt till which was encountered beneath the fill between elevations 176.7 and 177.7 metres or on newly placed engineered fill. A factored geotechnical resistance at Ultimate Limit States (ULS) of 500 kilopascals (kPa) and a geotechnical reaction at Serviceability Limit States (SLS) of 300 kPa may be used for design of shallow footings for a concrete cantilever wall. Load inclination factors for a resultant load inclination of 15 degrees were applied in determining the above values. This inclination angle is based on the cantilever wall option (Option 1) provided by Dillon's structural engineering department and results of associated calculations. An RSS retaining wall may be designed using the same geotechnical resistance/reaction.

Alternatively, a concrete cantilever or RSS wall also may be constructed on properly placed and compacted embankment backfill materials such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type III constructed on the native clayey silt till. A cantilever or RSS wall founded on a minimum 1.5 metre thickness of compacted granular fill may be designed using a factored geotechnical resistance at ULS of 600 kPa and a geotechnical reaction at SLS of 400 kPa. For concrete cantilever walls, these recommendations are based on footings having a minimum width of 3 to 5 metres.

Frost and Scour Protection and Embedment

All concrete cantilever wall footings should be provided with at least 1.2 metres of soil cover or thermal equivalent for frost protection purposes. An RSS wall founded on a granular levelling pad should have sufficient embedment to provide frost protection and overall stability. The embedment depth, defined as the distance from the top of the levelling pad to the top of the finished grade at the toe of the wall should be the maximum of:



- 0.5 metres;
- the minimum depth required for overall stability or to achieve bearing on native soils;
- $H/20$ – if the area in front of the wall is horizontal, where H is the total wall height;
- $H/10$ – if the area in front of the wall slopes down and away from the wall at a ratio of at 3 horizontal to 1 vertical or flatter⁶; or
- the minimum depth to provide adequate protection against scour.

Resistance to Lateral Forces

The resistance to lateral forces/sliding resistance between the retaining wall and the subgrade soils should be calculated in accordance with Section 6.7.5 of the Canadian Highway Bridge Design Code (CHBDC). In the case of cast-in-place concrete footings, shear failure occurs within the soil just below the surface of the footing and the effective internal angle of friction, ϕ' , of the founding soil should be used to determine the factored horizontal resistance. In the case of pre-cast block facing units for RSS systems, the interface angle of friction, δ , between the pre-cast elements and the bedding/leveling pad should be used. A factor of 0.8 is applied in the equation to calculate the factored horizontal geotechnical resistance, H_{ri} or H_{rs} , as follows:

$$H_{ri} = 0.8A'c' + 0.8V\tan\delta > H_f \quad (\text{for pre-cast elements})$$

$$H_{rs} = 0.8A'c' + 0.8V\tan\phi' > H_f \quad (\text{for cast-in-place elements})$$

where: A' - effective contact area, square meters

$$c' = \text{Nil}$$

$\tan \delta$ - coefficient of friction for interface between pre-cast founding elements and bedding/leveling pad

$\tan \phi'$ - coefficient of internal friction for soil close to the underside of cast-in-place footings

V - unfactored vertical force, kilonewtons

H_f - unfactored horizontal load, kilonewtons

Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the parameters provided in the table below may be used to determine the factored horizontal resistances.

⁶ FHWA (2001). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines. FHWA-NHI-00-043. Federal Highway Administration, Washington, D.C., USA.



FOUNDATION INVESTIGATION AND DESIGN REPORT RETAINING WALL AT E-N/S RAMP, SITE13-525/W

Wall Type	Interaction	Internal Angle of Friction, ϕ' (°)	$\tan \phi'$	Interface Angle of Friction, δ (°)	$\tan \delta$
Concrete Cantilever Wall	Cast-in-place concrete strip footing on native clayey silt till	32	0.62	-	-
	Cast-in-place concrete strip footing on granular fill	35	0.70	-	-
RSS Wall	Pre-cast concrete block facing units on Granular A levelling pad	-	-	30	0.58
	Pre-cast concrete block facing units on cast-in-place concrete levelling pad	-	-	25	0.47

6.3.2 Deep Foundations

For design of a soldier pile and lagging wall, the passive resistance is provided by the lateral resistance of the piles. Anchors embedded in the new embankment fill may be used to provide additional lateral support. It has been indicated that the soldier piles may consist of driven H-piles or H-piles embedded in pre-drilled holes. The fill materials encountered in the boreholes are not considered suitable to provide lateral support to the piles. The following unfactored parameters may be used to determine the lateral resistance of the soldier piles.

Soil Type and Location	Elevation (m)	Unit Weight, γ (kN/m ³)	Buoyant Unit Weight, γ' (kN/m ³)	Effective Friction Angle, ϕ'^1 (°)	Undrained Shear Strength, s_u (kPa)	Passive Earth Resistance Coefficient, K_p
Existing Fill	As applicable	19.0	9.2	-	-	-
Clayey Silt Till						
10+200 to 10+300	173.0 to 176.9					
10+300 to 10+330	175.3 to 176.9 & 171.6 to 173.5	21.0	11.2	32	150	3.3
10+330 to 10+400	171.6 to 176.9					
Clayey Silt						
10+330 to 10+400	171.6 to 173.0	20.0	10.2	30	125	3.0
Silty Clay Till						
10+330 to 10+310	173.5 to 175.3	21.0	11.2	30	200	3.0
Sandy Silt Till						
10+200 to 10+300	Below 173.0	21.0	11.2	32	-	3.3
10+300 to 10+400	Below 171.6					

Note 1: Use of the effective angle of internal friction and undrained shear strength are mutually exclusive, i.e., the undrained shear strength value is not to be interpreted or used as a "cohesion" value for analysis based on effective stress principles.



Where an undrained shear strength, s_u , is provided, the undrained capacity of the pile should be checked to determine whether the drained or undrained case will govern. The lateral resistance for the length of the pile within cohesive soil should be calculated assuming an unfactored passive lateral pressure distribution varying linearly from $2 s_u$ at the surface to $9 s_u$ at a depth of three times the pile width and below.

Where the lateral loading exerted by the piles will be resisted by granular soils, the unfactored passive lateral earth pressure, P_p , distributed along the length of the pile foundation may be calculated using the following equations:

$$P_p = K_p \gamma d \quad \text{above the groundwater table}$$

$$P_p = K_p \gamma d_w + K_p \gamma' (d - d_w) \quad \text{below the groundwater table}$$

where: K_p = passive earth pressure coefficient

γ = bulk unit weight, kN/m^3

γ' = effective unit weight below the groundwater level ($\gamma' = \gamma - \gamma_w$), kN/m^3

d = depth below the ground surface, m

d_w = depth to the groundwater level, m

A resistance factor of 0.5 should be applied to the calculated lateral resistance in order to obtain the factored ULS lateral geotechnical resistance. The upper 1.2 metres below the ground surface should be neglected in the calculation of the passive resistance in front of the pile to account for frost action. Further, the existing fill should not be considered reliable for providing passive resistance; however, the vertical dead load (weight) of the upper 1.2 metres of existing fill may be considered equivalent to a surcharge pressure when calculating passive resistances offered by the native soil.

6.4 Stability

The internal stability of mechanically-reinforced soil walls should be verified by the RSS supplier/designer. If a RSS wall is to be constructed below the maximum flood elevation specified by the MTO, the design considerations outlined in Section 6.3 for walls subjected to submergence must be taken into account. The global stability under static loading for a properly designed and constructed wall at the critical section was analysed using SLOPE/W Version 7.19, a commercially available software package by Geo-Slope International for limit equilibrium stability analyses. A preliminary examination of the global and external stability of concrete cantilever and RSS retaining wall options was conducted for the critical section associated with the anticipated maximum wall height. A tiered RSS wall with dimensions equivalent to the single RSS wall was also analysed. Effective stress (long-term) and undrained (short-term) conditions were considered. At about Station 10+400 near the west end of the wall, based on an anticipated founding elevation of 176.9 metres and top of wall elevation of 189.9 metres, a total wall height of 13.0 metres is anticipated with the lower 3 metres being buried below the final grade. The analyses suggested that each wall option was stable and met or exceeded the minimum required factors of safety of 2.0 for overturning and 1.5 for sliding (neglecting passive resistance). The results of the stability analyses are provided in Appendix C. Granular backfill materials were assumed for the



analyses. If lightweight fill is used, the effect on the wall stability should be analysed. Upon completion of the detail design, the RSS wall designer/supplier should provide the results of the analyses of the internal, external, global and compound stability (in the case of tiered walls), confirming that the RSS retaining wall design meets the above-noted requirements.

6.5 Lateral Earth Pressures

The lateral pressures acting on the proposed retaining wall will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure and on the drainage conditions behind the wall. Backfill behind the wall should be in accordance with the MTO Structural Manual, OPSS 501 and Ontario Provincial Standard Drawing (OPSD) 3121.150. The following recommendations are made concerning the design of the wall.

- Select, free-draining granular fill meeting the specifications of OPSS Granular A or Granular B Type III but with less than 5 per cent passing the 0.075 millimetre sieve should be used as backfill behind the wall. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with OPSS 501.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3121.150 and 3190.100, as appropriate.
- Cantilever, soldier pile and RSS retaining walls are considered to be unrestrained and allow lateral yielding of the stems; active earth pressures may be used in the geotechnical design of the structures. For unrestrained walls, 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 III</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
Passive, K_p	3.7	3.3

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 Settlement

Data Interpretation

Estimates of engineering parameters used in the settlement analysis were based on SPT N values obtained in boreholes 601 to 606 as well as borehole 611 to 614 advanced for the adjacent S-W ramp, and an oedometer test carried out on a sample from borehole 604. 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 (Subsurface Conditions Interpretation Report for the Windsor Essex Parkway) since the soils at the Highway 401/Highway 40 interchange site are of similar geologic origin and composition.

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

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

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

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

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

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

Also, the preconsolidation pressure was established using the undrained shear strength values based on CPT data. The following well-known relationship was used to estimate the preconsolidation pressure based on undrained shear strength measurements.

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

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 at this 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 would exhibit recompression behaviour and that the preconsolidation pressure would have little if any effect on the magnitude of settlement.

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



Settlement Analysis

Settlement of the founding soils due to the proposed wall construction and 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 table below summarizes the engineering parameters and simplified stratigraphy used in the analysis. It should be noted that the selected preconsolidation pressure is based on the correlations between the SPT, CPT, field shear vane data and oedometer testing as described above. This methodology for deriving the preconsolidation pressure was considered to be more representative than using the oedometer data only.

Material and Elevation (m)	γ (kN/m ³)	E_s (kPa)	E_{ur} (kPa)	ν	C_c	C_r	σ'_p (kPa)	e_o
Clayey Silt to Silty Clay Till 172.5 to 177.0	22	-	-	0.49	0.142	0.016	773 to 545	0.51
Sandy Silt Till 164.0 to 172.5	22	110,000	110,000	0.3	-	-	-	-
Sand and Gravel 160.0 to 164.0	22	400,000	400,000	0.3	-	-	-	-

The embankment loads and retaining wall heights were inferred from the provided drawings and it was assumed that all existing fill materials are removed and the ground surface elevation at the time of construction will be consistent with the elevation of the fill material/native soil interface. The calculated total settlement along the face of the retaining wall is 60 to 75 millimetres near the maximum retaining wall height. In general, settlement magnitudes will diminish approximately proportionally as the retaining wall and embankment heights diminish. The majority of the settlement is expected to occur during construction in proportion to the rate and magnitude of applied loads.

Settlement Performance Requirements

According to the MTO Embankment Settlement Criteria for Design (July 2010) the total post-construction settlement of the paved portion of non-freeways for new embankments on compressible soils shall not exceed 200 millimetres over a 20 year period with maximum differential settlement of 100:1.

The sections of the E-N/S and S-W ramp embankments adjacent to the proposed wall are considered to be high fill areas. Design and settlement of these new embankments have been discussed under separate cover for this assignment.



6.7 Additional Design Considerations

6.7.1 RSS Walls

The RSS retaining wall is to be designed in accordance with the MTO RSS Guidelines (2008) and special provision (SP) 599S22. If metallic reinforcement will be utilized for the RSS wall, it is recommended that an impervious barrier be placed between the pavement structure and the reinforced fill to protect it from damage by de-icing salts.

It is preferred that utilities with alignments parallel to the wall face not be placed within the reinforced backfill zone of a RSS wall. The design of a RSS retaining wall must consider any proposed highway infrastructure such as culverts, barriers, guide rails, catch basins, signs and light poles. This may require construction of a structural frame around the culvert, splaying, or full or partial omission of RSS wall reinforcement in the area of the obstruction. The adjacent reinforcement must be designed to accommodate the additional loading resulting from removal of reinforcing elements in the area of the infrastructure.

Stepped footings for a RSS or cantilever wall, if required, should not exceed a height of 0.5 metres and should be provided with a minimum 1 metre length of horizontal footing on each side (i.e. an overall slope of 1 vertical to 2 horizontal).

6.7.2 Drainage

The design of the retaining wall must incorporate subsurface and surface drainage elements. The design of drainage facilities must consider the extreme water levels associated with flood conditions and rapid drawdown. It is expected that the existing topography generally directs surface flows towards McGregor Creek. These surface flows should be adequately redirected when the site is regraded.

6.7.3 Permanent Deadmen or Soil Anchors

An anchored soldier pile and lagging wall or sheet pile wall may be more compatible than an RSS system with respect to placement of buried utilities or highway infrastructure. For this project, however, it is anticipated that there will be no significant utilities in the vicinity of the retaining wall.

Given the expected geometry of this retaining wall, soil anchors are likely not a practical option whereas driven sheet pile, driven pile, or cast-in-place concrete deadmen constructed within the new ramp fill are far more practical. If, for other reasons, soil anchors (tiebacks) will be considered for this project, this office should be contacted for additional design recommendations. If deadmen anchors will be used the length of the tie-rods and positions of the anchors should be selected such that the active earth pressure zone of the wall and the passive earth pressure zone of the anchors does not overlap. Otherwise, the passive resistance of the anchor may be reduced. For multiple levels of deadmen anchors, this office should be contacted for design assistance.



6.7.4 Other Design Considerations

The durability of backfill materials, facing elements, reinforcing or restraining elements located in areas subject to submergence must be considered in the wall design. Factors that must be considered include corrosion, freeze-thaw, scour and ice action. The level of scour protection required must be assessed with input from a hydraulic or river engineer. Ground anchors may be required for higher sections of a concrete cantilever wall or a soldier pile and lagging wall. The surficial clayey silt till deposit is considered suitable for placement of soil anchors. This office should be contact to provide design recommendations if ground anchors or deadmen are to be included in the design.

6.8 Construction Considerations

6.8.1 Excavations and Groundwater Control

Excavation and backfilling operations for the proposed retaining wall should be conducted in accordance with OPSS 902. All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The uncontrolled fill materials at this site would be classified as Type 3 soils as would any cohesionless materials below the groundwater level. The native clayey soils and properly dewatered granular materials may be classified as Type 2 soils. Temporary open cut slopes within the fill materials should be maintained no steeper than 1 horizontal to 1 vertical.

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

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



soon as practicable following exposure of the native soils. This working subgrade of granular fill should limit the influence of water and construction traffic on the native subgrade and, when supplemented with properly filtered sumps and pumps, could assist with control that enters the excavations.

6.8.2 Driven or Drilled Soldier Piles and Sheet Piles

It should be noted that the presence of cobbles and boulders in the till soils and the underlying granular layers as well as the potential for debris within the existing fill materials should be anticipated which may affect pile driving/drilling operations. Also, suspected methane gas was encountered during drilling at borehole 603 and has been reported in exploratory borings within or near the Kettle Point bedrock formation. An NSSP should be added to the Contract Documents to alert the Contractor to the need for special procedures to deal with groundwater flow, cobbles, boulders and other obstructions, and potential methane gas that may be encountered during pile installation. Driven piles should be equipped with reinforced flanges or Type I driving shoes as shown in OPSD 3000.100. Driven soldier piles should also be installed and monitored in accordance with OPSD 3000.150 and OPSS 903. In this particular case, the depth of pile penetration will be determined by lateral resistance considerations and, therefore, determination of vertical resistance and bearing stratum will not be necessary. However, the drawings should specify the maximum (highest) tip elevation permissible for achieving lateral load resistance requirements.

Temporary or permanent liners may be required during any pile drilling due to the presence of saturated and granular fill materials. Groundwater flow is expected from the fill materials. In such instances, tremie concrete placement techniques may also be required.



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.

GOLDER ASSOCIATES LTD.

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TABLE I

COMPARISON OF RETAINING WALL ALTERNATIVES

Retaining Wall at E-N/S Ramp, Site 13-525/W
 Highway 401 and Highway 40 Interchange Reconfiguration
GWP 3093-09-00

WALL OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Retained Soil System Wall	<ul style="list-style-type: none"> Feasible (preferred technical alternative). 	<ul style="list-style-type: none"> Formwork not required. Least expensive wall option. More rapid construction compared to cantilever walls. More tolerant of differential settlement compared to cantilever walls. Intermediate in cost between soldier pile and lagging and cantilever walls. Does not require specialized equipment. 	<ul style="list-style-type: none"> Select granular backfill required within reinforced zone. Reinforcing elements vulnerable to degradation from exposure to UV light, de-icing salts, aggressive backfill geochemistry and wet-dry cycles if located in submergence zone. Wall option with greater restriction of highway infrastructure within reinforcement zone. More space required behind wall for construction than some other wall types. 	<ul style="list-style-type: none"> 1.0 	<ul style="list-style-type: none"> Low to moderate risk. Risk dependent on care and control taken during reinforcement installation and backfill compaction. Risk dependent on anticipated and actual water levels and dewatering.

COMPARISON OF RETAINING WALL ALTERNATIVES

WALL OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Soldier Pile and Lagging Wall	<ul style="list-style-type: none"> • Feasible. 	<ul style="list-style-type: none"> • Formwork not required. • Wall option requiring least effort for site preparation and excavation. • Less expensive than cantilever walls. • Relatively fast construction compared to RSS and cantilever walls. 	<ul style="list-style-type: none"> • Where wall heights exceed 4 to 6 metres, ground anchors or deadmen may be required to provide lateral resistance. • Restrictions on locations of highway infrastructure will be required if wall is anchored. 	<ul style="list-style-type: none"> • 1.2 to 1.5 higher cost if wall is anchored 	<ul style="list-style-type: none"> • Moderate risk. • Risk dependent on anticipated and actual water levels and dewatering. • Settlement related damage may occur if sequence of fill placement and anchor installation is not well coordinated.
Concrete Cantilever Wall	<ul style="list-style-type: none"> • Marginally feasible depending on embankment fill sequencing. 	<ul style="list-style-type: none"> • Less strict backfill requirements compared to RSS walls. • No restriction on placement of highway infrastructure behind wall. 	<ul style="list-style-type: none"> • Most expensive wall option. • Wall option with longest construction time. • Less accommodating to differential settlements than RSS and Soldier Pile walls. • Greatest effort for site preparation and excavation compared to RSS and Soldier Pile walls. 	<ul style="list-style-type: none"> • 1.7 to 2.5 higher cost if wall is anchored 	<ul style="list-style-type: none"> • Low to moderate construction risk. • Settlement related damage may occur if sequence of fill placement and anchor installation is not well coordinated. • Risk dependent on anticipated and actual water levels and dewatering.

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

Prepared By: NG
 Checked By: 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 601

1 OF 1

METRIC

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

SOIL PROFILE			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 ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE	20						40	60
180.44	GROUND SURFACE																
0.00	TOPSOIL, clayey silt, some sand, roots Brown																
0.30																	
0.52	FILL, sandy silt, some clay, trace gravel Grey		1	SS	8												
	FILL, silty clay, trace to some sand, trace gravel, roots Firm Brown and grey		2	SS	7												
178.31																	
2.13	FILL, sand and gravel, some silt Dark brown Loose		3	SS	7												
177.70																	
2.74	CLAYEY SILT TILL, some sand, trace gravel Stiff to very stiff Grey		4	SS	20												
			5	SS	16												
			6	SS	13												
			7	SS	18												
			8	SS	14												
173.12																	
7.32	SAND, some gravel Compact Grey		9	SS	14												
172.67																	
7.77	CLAYEY SILT TILL, sandy, trace gravel Stiff Grey																
171.60																	
8.84	SANDY SILT TILL, some gravel, trace to some clay Compact Grey		10	SS	18												
			11	SS	26												
			12	SS	19												
167.33																	
13.11	SILT, some sand, trace to some clay, trace gravel Very dense Grey		13	SS	84												
166.27																	
14.17	END OF BOREHOLE Groundwater encountered at about elev. 178.3m during drilling on June 6, 2014.																

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

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 27/01/15

RECORD OF BOREHOLE No 602

1 OF 1

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE					
180.40	GROUND SURFACE						20	40	60	80	100					
0.00	TOPSOIL, clayey silt, some sand, roots Black															
0.24	FILL, clayey silt, some sand, trace gravel Firm Brown		1	SS	5											
			2	SS	6											
178.27																
2.13	FILL, sand, some silt, some gravel, with clayey silt pockets Very loose to loose Brown and black		3	SS	3											
			4	SS	6											
176.74																
3.66	CLAYEY SILT TILL, sandy, trace gravel Firm to very stiff Grey		5	SS	13											
			6	SS	17											
			7	SS	9											
			8	SS	10											
173.39																
7.01	SANDY SILT TILL, some clay, trace to some gravel Dense to very dense Grey		9	SS	46											
			10	SS	77											
169.58																
10.82	SAND, fine to medium, some gravel		11	SS	45											
169.27	Dense															
11.13	Grey															
	END OF BOREHOLE															
	Groundwater encountered at about elev. 178.3m during drilling on June 9, 2014.															
	Water level measured in Piezometer at elev. 178.27m following installation on June 9, 2014.															
	Water level measured in Piezometer at elev. 179.70m on June 11, 2014.															
	Water level measured in Piezometer at elev. 179.58m on June 16, 2014.															
	Water level measured in Piezometer at elev. 179.76m on July 9, 2014.															

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

RECORD OF BOREHOLE No 603

1 OF 2

METRIC

PROJECT 13-1132-0111

W.P. 3093-09-00

LOCATION N 4693846.9 , E 338293.6

ORIGINATED BY SL

DIST HWY 401

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

COMPILED BY WDF

DATUM GEODETIC

DATE June 12, 2014

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE							w _p w w _L			
								● QUICK TRIAXIAL × LAB VANE										
180.57	GROUND SURFACE						20	40	60	80	100				GR	SA	SI	CL
0.00	TOPSOIL, clayey silt, some sand, roots																	
0.24	Dark brown FILL, clayey silt, some sand, trace gravel, roots Firm Brown and grey		1	SS	8													
			2	SS	4													
178.44																		
2.13	FILL, sandy silt, some clay Loose Brown and grey		3	SS	4													
177.67																		
2.90	FILL, silty sand, some gravel, with clayey silt pockets Very loose Grey and black		4	SS	2													
176.91																		
3.66	CLAYEY SILT TILL, sandy, trace gravel Stiff to very stiff Grey		5	SS	19													
			6	SS	15													
			7	TO	PH													
			8	SS	14													
173.71																		
6.86	CLAYEY SILT, trace sand, silt pockets Stiff to very stiff Grey		9	SS	12													Possible methane gas pocket near elev. 173.9m
171.58																		
8.99	SANDY SILT TILL, some clay, trace gravel Compact Grey		10	SS	24													
170.51																		
10.06	SAND AND GRAVEL, trace silt Compact Grey		11	SS	23													61 37 (2)
169.60																		
10.97	SANDY SILT TILL, some clay, trace gravel Compact to very dense Grey		12	SS	28													
166.91																		
13.66	SILTY FINE SAND		13	SS	92													
166.55	Very dense Grey																	
14.02	SANDY SILT TILL, some clay, trace gravel Very dense Grey																	

Continued Next Page

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

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 603		2 OF 2	METRIC
W.P. <u>3093-09-00</u>	LOCATION <u>N 4693846.9 , E 338293.6</u>	ORIGINATED BY <u>SL</u>			
DIST <u> </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE</u>	COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>	DATE <u>June 12, 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					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE													
							20	40	60	80	100		10	20	30						
		SANDY SILT TILL, some clay, trace gravel Very dense Grey		14	SS	111													4 39 41 16		
163.78																					
16.79																					
16.92		SILT, trace sand, trace clay Very dense Grey END OF BOREHOLE Groundwater encountered at about elev. 174.8m during drilling on June 12, 2014.		15	SS	101													Possible methane gas pocket near elev. 164.1m		

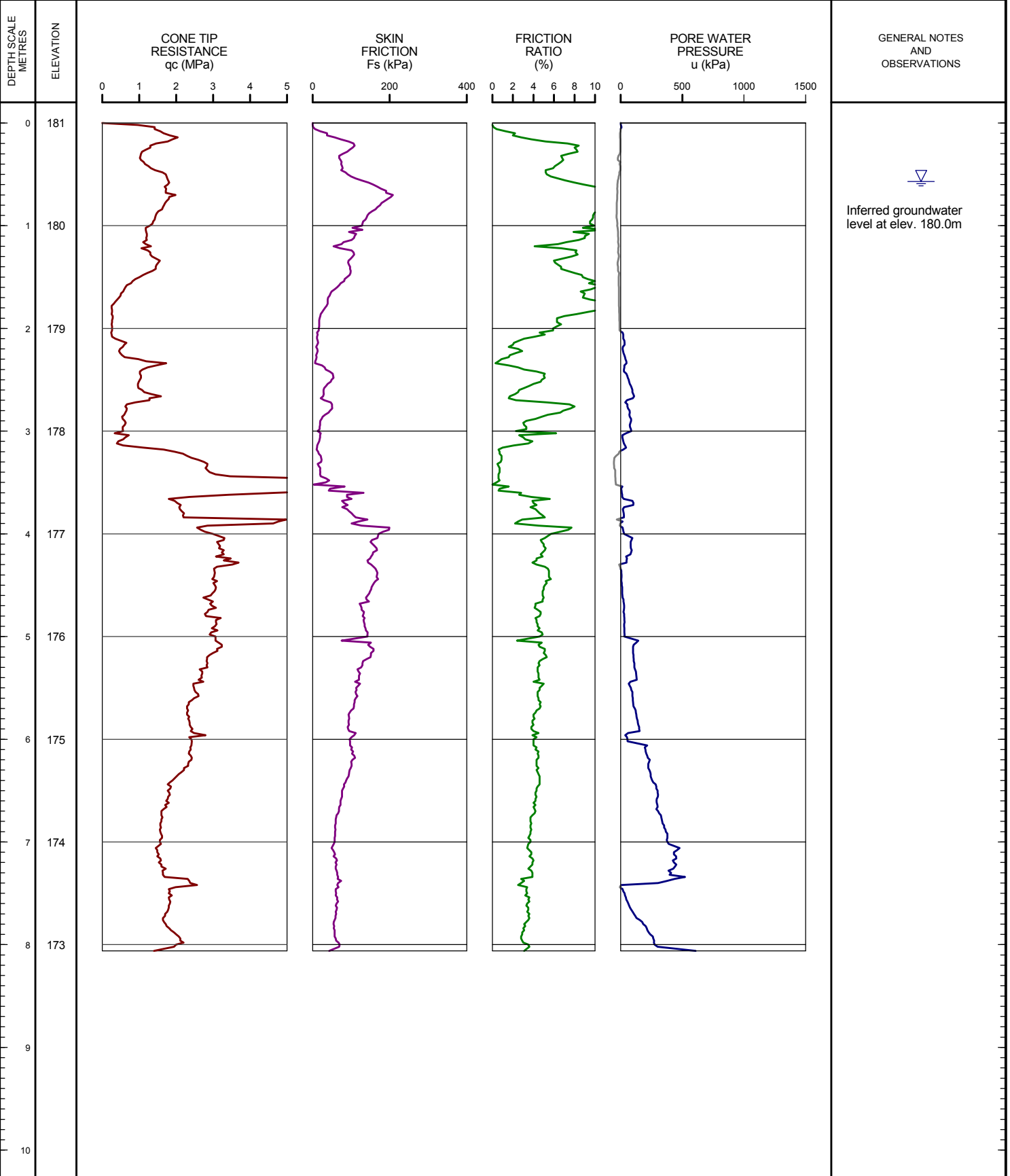
PROJECT: 13-1132-0111
LOCATION: N 4693848.0 ;E 338292.0

RECORD OF CONE PENETRATION TEST CPT-603

TEST DATE: June 10, 2014

SHEET 1 OF 1
DATUM: GEODETIC

GROUND SURFACE ELEVATION: 180.57m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.587 CORRECTION FACTOR B: 0.014



LDN_CPT_02-C 13-1132-0111.GPJ GLDR_LON.GDT 14/10/14 DATA INPUT:

DEPTH SCALE
1 : 50



OPERATOR: MK
CHECKED:

RECORD OF BOREHOLE No 604

1 OF 2

METRIC

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		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		
180.71	GROUND SURFACE							20 40 60 80 100						
0.00	TOPSOIL, clayey silt, some sand, roots							○ UNCONFINED + FIELD VANE						
0.15	Black							● QUICK TRIAXIAL × LAB VANE						
	FILL, clayey silt, some sand, trace gravel, roots							20 40 60 80 100						
	Soft to firm													
	Brown													
			1	SS	7									
			2	SS	3									
			3	SS	2									
177.81														
2.90	FILL, silty sand, some clay, trace gravel													
	Loose													
	Grey													
			4	SS	6									
177.05														
3.66	CLAYEY SILT TILL, some sand, trace gravel													
	Stiff													
	Grey													
			5	SS	13									
			6	SS	14									
			7	TO	PH									
			8	SS	10									
			9	SS	13									
172.48														
8.23	CLAYEY SILT, some sand, silt pockets													
	Hard													
	Grey													
			10	SS	34									
171.66														
9.05	SANDY SILT TILL, some clay, trace gravel													
	Dense													
	Grey													
			11	SS	25									
170.65														
10.06	SAND, some silt, some gravel, trace to some clay													
	Compact													
	Grey													
			12	SS	26									
169.89														
10.82	SANDY SILT TILL, some clay, trace gravel													
	Compact													
	Grey													
			13	SS	66									
167.91														
12.80	SAND, medium to coarse, trace to some gravel, trace to some silt													
	Very dense													
	Grey													
166.69														
14.02	SANDY SILT TILL, some clay, trace gravel													
	Very dense													
	Grey													

Continued Next Page


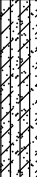
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

LDN_MTO_06 13-1132-0111.GPJ LDN_MTO.GDT 27/01/15

PROJECT <u>13-1132-0111</u>		RECORD OF BOREHOLE No 604		2 OF 2	METRIC
W.P. <u>3093-09-00</u>	LOCATION <u>N 4693799.9 , E 338254.4</u>	ORIGINATED BY <u>SL</u>			
DIST <u> </u> HWY <u>401</u>	BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM, MUD ROTARY/TRICONE</u>	COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>	DATE <u>June 12, 2014 - June 13, 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 (%) 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					w _p w w _L						
						20	40	60	80	100									
164.25	SANDY SILT TILL, some clay, trace gravel Very dense Grey		14	SS	120														
16.46	SAND, fine, some silt, sandy silt seams Very dense Grey		15	SS	78														
163.34																			
17.37	SAND AND GRAVEL, trace to some silt Very dense Grey																		
			16	SS	50/ 75mm														
160.75			17	SS	100														
19.96	END OF BOREHOLE																		
	Groundwater not established during drilling on June 12, 2014.																		
	Water level measured in Piezometer at elev. 175.44m on June 16, 2014.																		
	Water level measured in Piezometer at elev. 173.64m on June 20, 2014.																		
	Water level measured in Piezometer at elev. 175.53m on July 9, 2014.																		

PROJECT 13-1132-0111		RECORD OF BOREHOLE No 605		1 OF 1 METRIC	
W.P. 3093-09-00	LOCATION N 4693782.8 , E 338259.8			ORIGINATED BY SL	
DIST HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM			COMPILED BY WDF	
DATUM GEODETIC	DATE June 4, 2014			CHECKED BY	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	w _p	w	w _L					
181.54	GROUND SURFACE																			
0.03	TOPSOIL, silty sand, some gravel, roots																			
0.24	Black																			
	FILL, silty sand and gravel																			
	Brown																			
180.17	FILL, clayey silt, some sand, some gravel, black sand pockets		1	SS	5															
1.37	Firm																			
	Brown																			
179.41	FILL, silt, some sand		2	SS	7															
	Loose																			
	Grey																			
2.13	FILL, clayey silt, sandy, trace gravel		3	SS	5															
	Firm																			
	Grey																			
178.64	FILL, silty sand and gravel, with silty clay pockets, plastic and wood		4	SS	16															
	Compact																			
	Grey																			
177.52			5	SS	11															
4.02	CLAYEY SILT TILL, some sand, trace gravel																			
	Stiff to very stiff																			
	Grey																			
			6	SS	15															
175.75			7	SS	18															
5.79	END OF BOREHOLE																			
	Groundwater not established during drilling on June 4, 2014.																			
	Water level in open borehole at elev. 177.46m following drilling on June 4, 2014.																			

RECORD OF BOREHOLE No 606

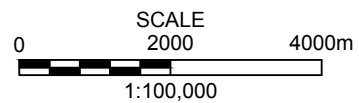
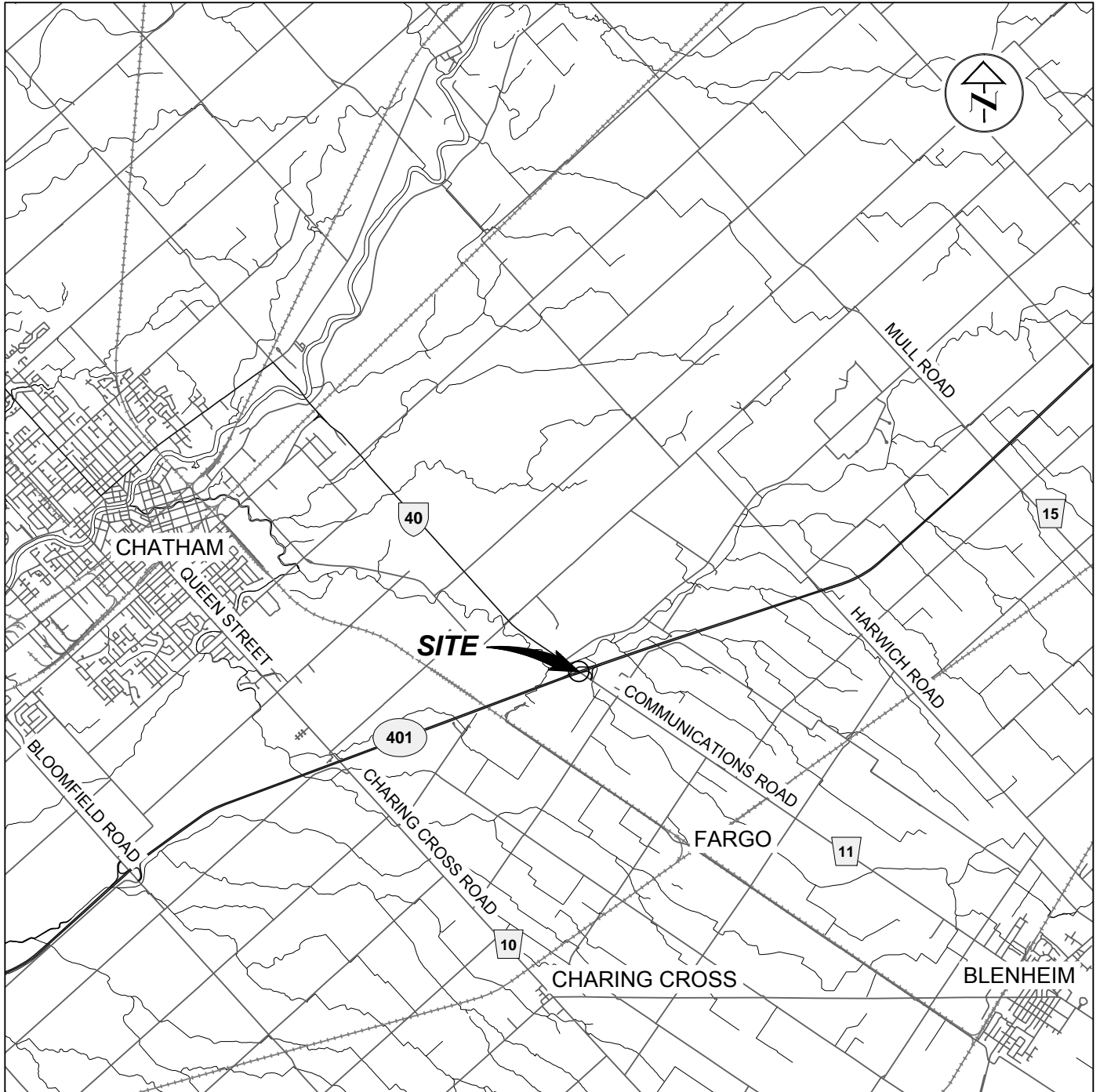
1 OF 1

METRIC

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

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										
								<div><div></div><div>20406080100</div></div>										
								<div><div>○ UNCONFINED</div><div>+ FIELD VANE</div><div>● QUICK TRIAXIAL</div><div>× LAB VANE</div></div>								WATER CONTENT (%)		
								<div><div>20406080100</div></div>										
180.49	GROUND SURFACE						180											
0.00	TOPSOIL, clayey silt, some sand, roots																	
180.03	Black																	
0.46	FILL, clayey silt, sandy, roots		1	SS	7													
	Soft to firm																	
	Brown		2	SS	3													
							179											
177.90			3	SS	4		178											
2.69	FILL, silty sand, some clay																	
2.71	Loose																	
	Grey																	
177.20	FILL, sand and gravel, with clayey silt pockets		4	SS	6		177											
3.29	Loose																	
	Grey to black																	
	CLAYEY SILT TILL, some sand, trace gravel		5	SS	16		176											
	Firm to very stiff																	
	Grey		6	SS	21													
175.31							175											
5.18	SILTY CLAY TILL, some sand, trace gravel		7	SS	20													
	Stiff to very stiff																	
	Grey		8	SS	13													
173.48																		
7.01	CLAYEY SILT TILL, some sand, trace gravel																	
	Hard																	
	Grey		9	SS	30		173											
172.41																		
8.08	END OF BOREHOLE																	
	Groundwater not established during drilling on June 6, 2014.																	
	Water level measured at elev. 172.96m following installation on June 6, 2014.																	
	Water level measured in Piezometer at elev. 173.02m on June 9, 2014.																	
	Water level measured in Piezometer at elev. 173.30m on June 16, 2014.																	
	Water level measured in Piezometer at elev. 173.91m on July 9, 2014.																	

+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

RETAINING WALL
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

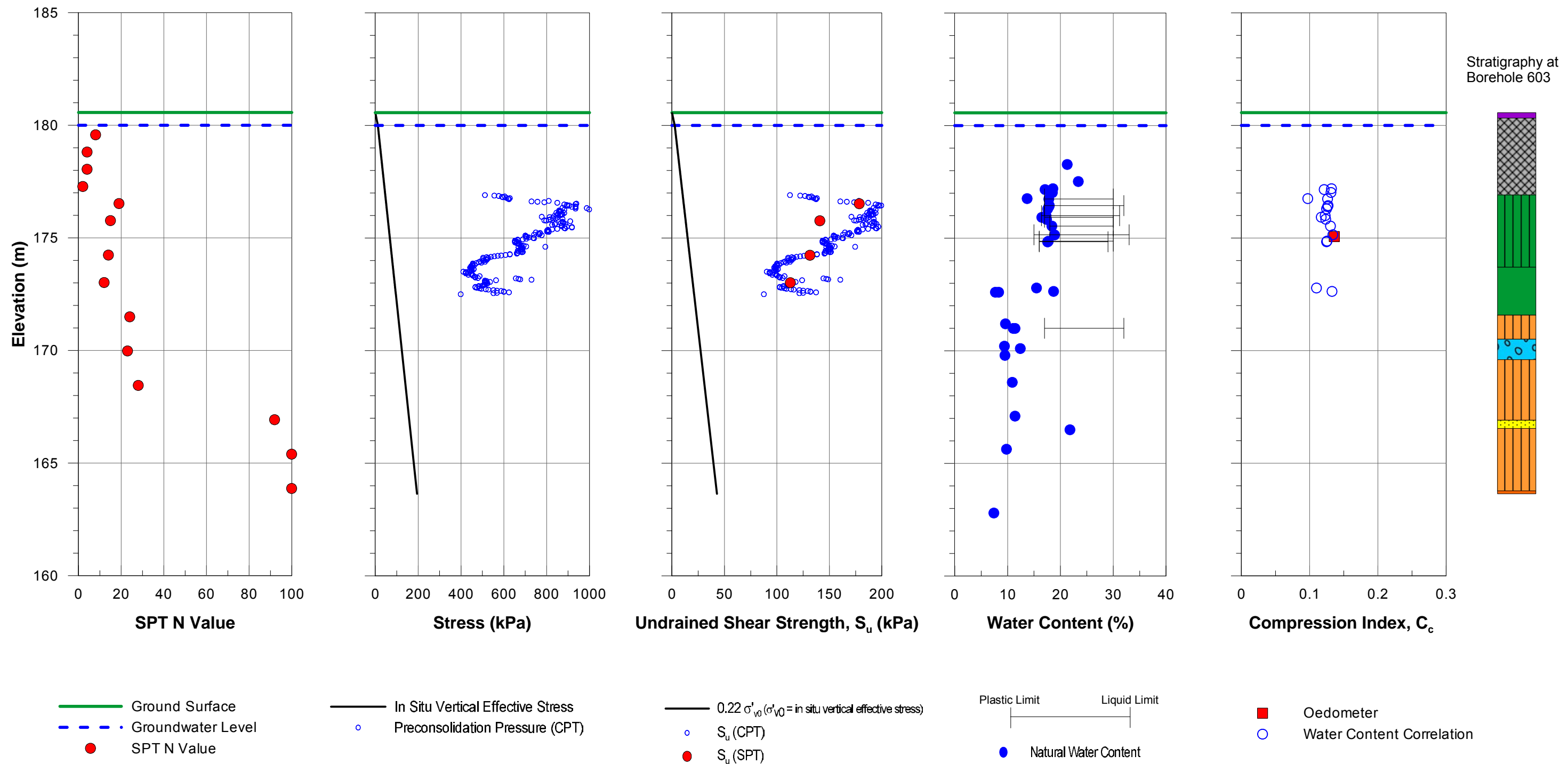
KEY PLAN



PROJECT No. 13-1132-0111		FILE No. 1311320111-1000-F07001	
CADD	LMK	July 17/14	SCALE AS SHOWN
CHECK			REV. 0


FIGURE 1

N:\active\2013\1132-Gold\1132-0111-132-0111 DILLON\3\MP 3093-09-00-HWY 40\40\Drilling\GRAPH-ER FILES\11320111-100-R07002.gif

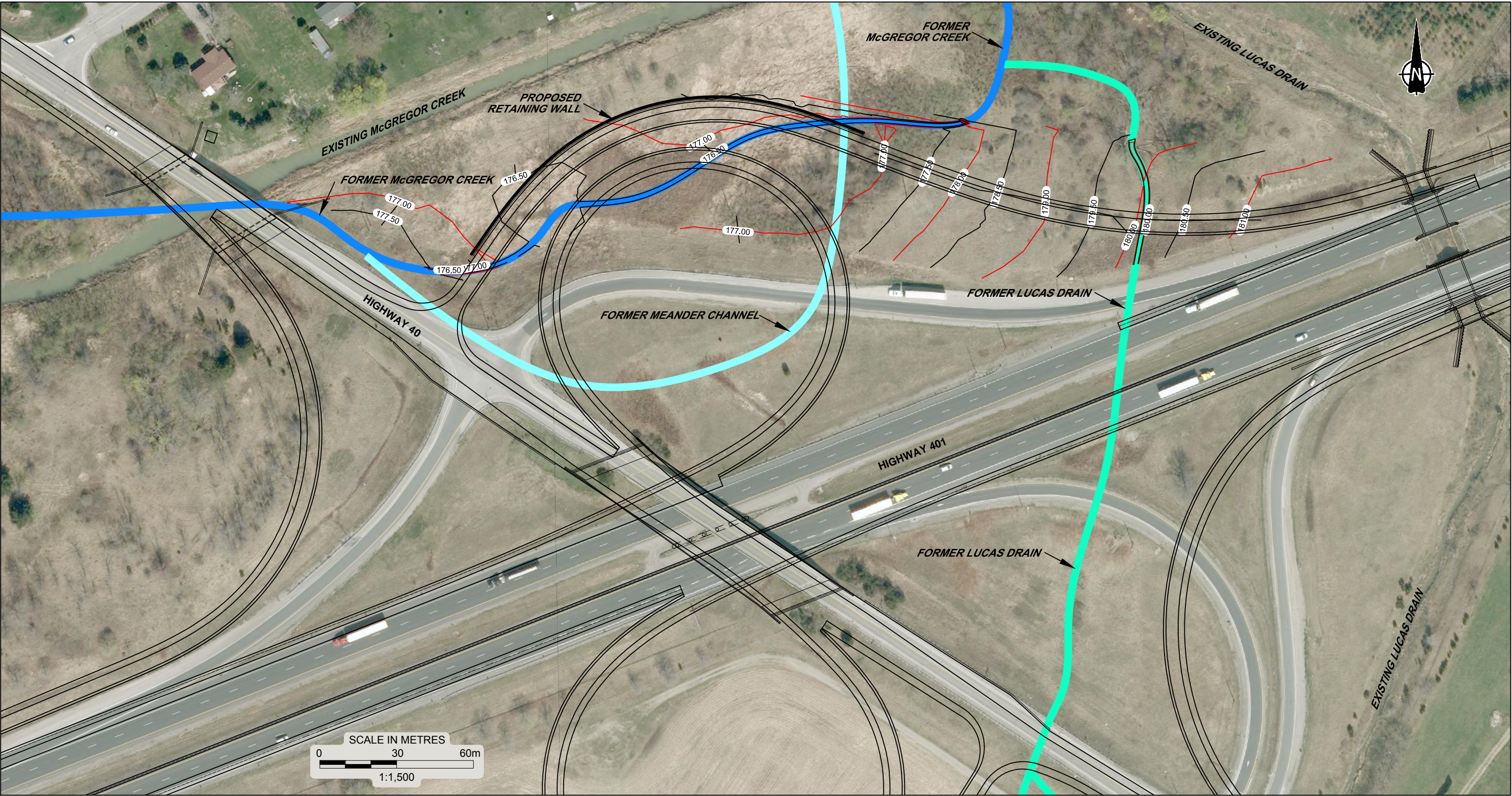


NOTES

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

PROJECT		RETAINING WALL	
		HWY 401/HWY 40 INTERCHANGE RECONFIGURATION	
		GWP 3093-09-00	
TITLE		SUMMARY OF SUBSURFACE TEST DATA	
		BOREHOLE 603/CPT 603	
	PROJECT No.	13-1132-0111	FILE No. 1311320111-1000-R07002
	DRAWN	MK	Oct. 1/14
	CHECK		
		SCALE	AS SHOWN
		REV.	0
		FIGURE 2	

Drawing file: 1311320111-1000-F07003.dwg Feb 26, 2015 - 1:48pm




REFERENCE

DRAWING BASED ON PLANS PROVIDED IN DIGITAL
FORMAT BY DILLON, RECEIVED FEB. 5, 2015; AND
1955 AERIAL IMAGE No. 55-4217/55-202.

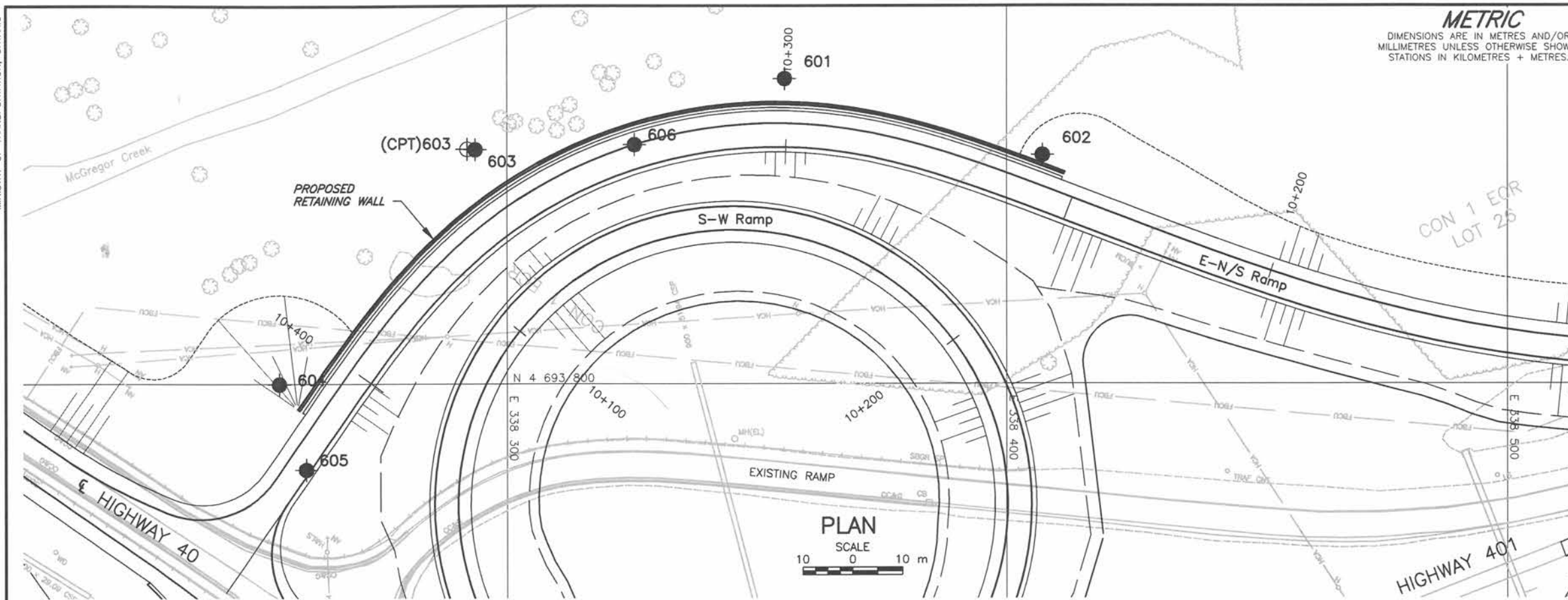
NOTES

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

PROJECT		RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00	
TITLE		FORMER LOCATION OF MCGREGOR CREEK, LUCAS DRAIN, AND MEANDER CHANNEL	
PROJECT No. 13-1132-0111		FILE No. 1311320111-1000-F07003	
CADD	DCH	Jan. 27/15	SCALE AS SHOWN REV. 0
CHECK			FIGURE 3



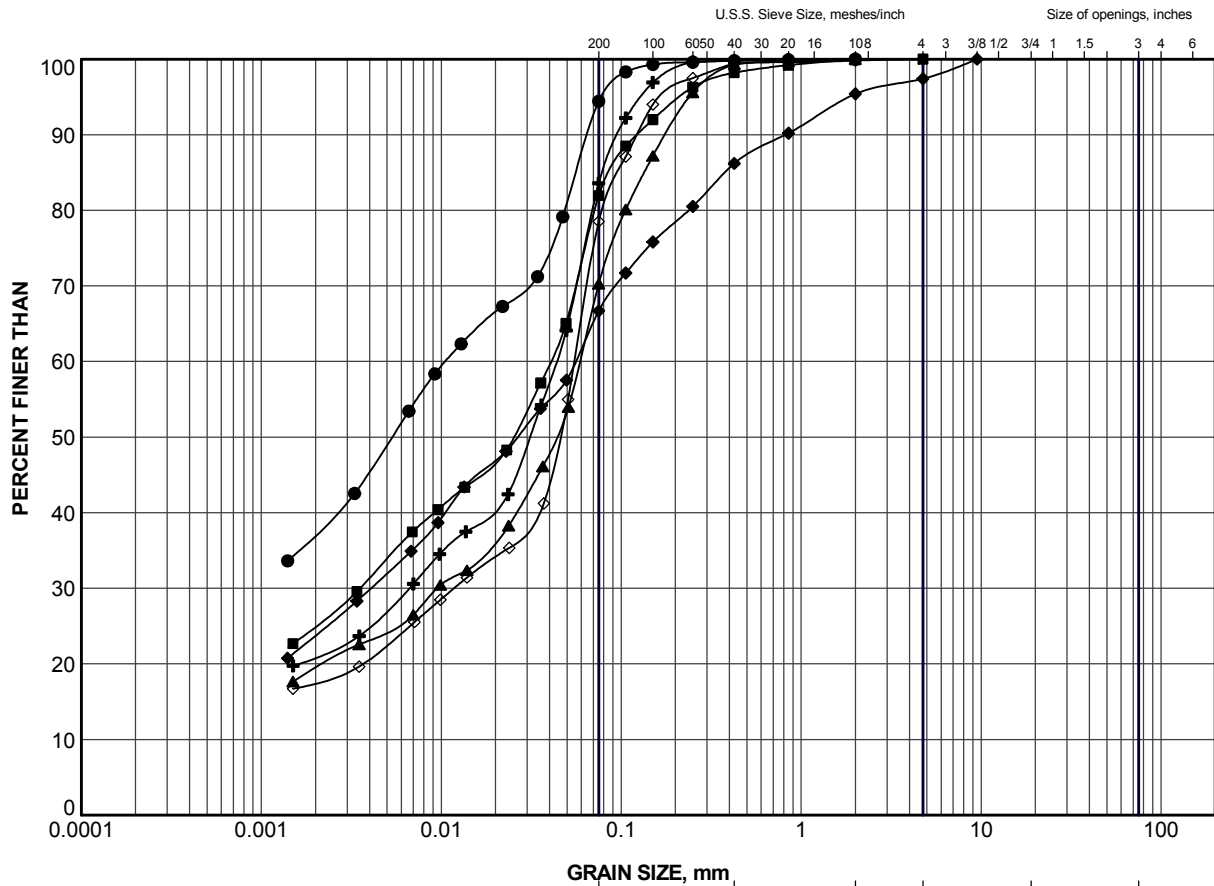
**Golder
Associates**
LONDON, ONTARIO





APPENDIX A

Laboratory Test Data – Routine Soils



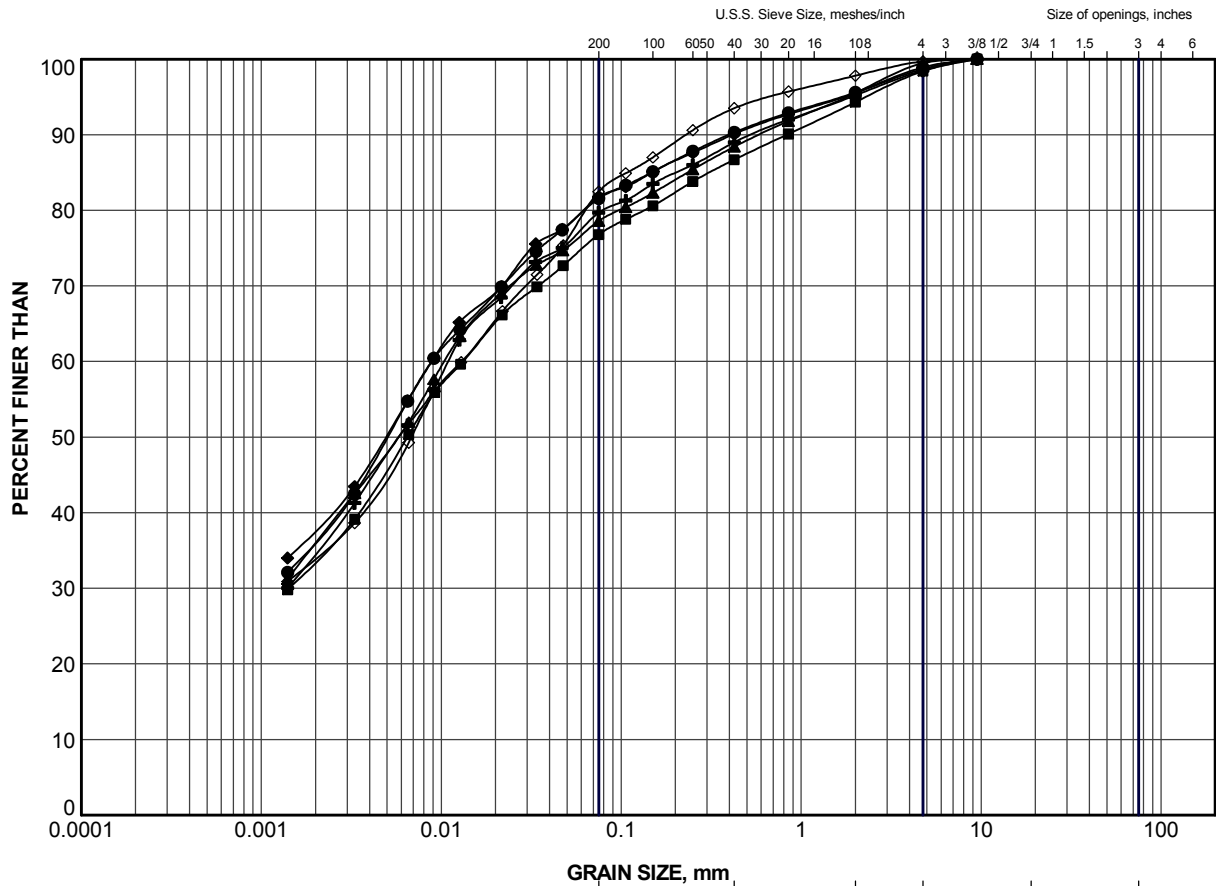
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	2	178.7
■	602	2	178.7
▲	603	3	178.1
+	604	2	179.0
◆	605	3	179.0
◇	606	2	178.7

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION FILL			
PROJECT No.		13-1132-0111		FILE No. 1311320111-1000-F070A1			
DRAWN		WDF		Sep 08/14		SCALE N/A REV.	
CHECK						FIGURE A-1	





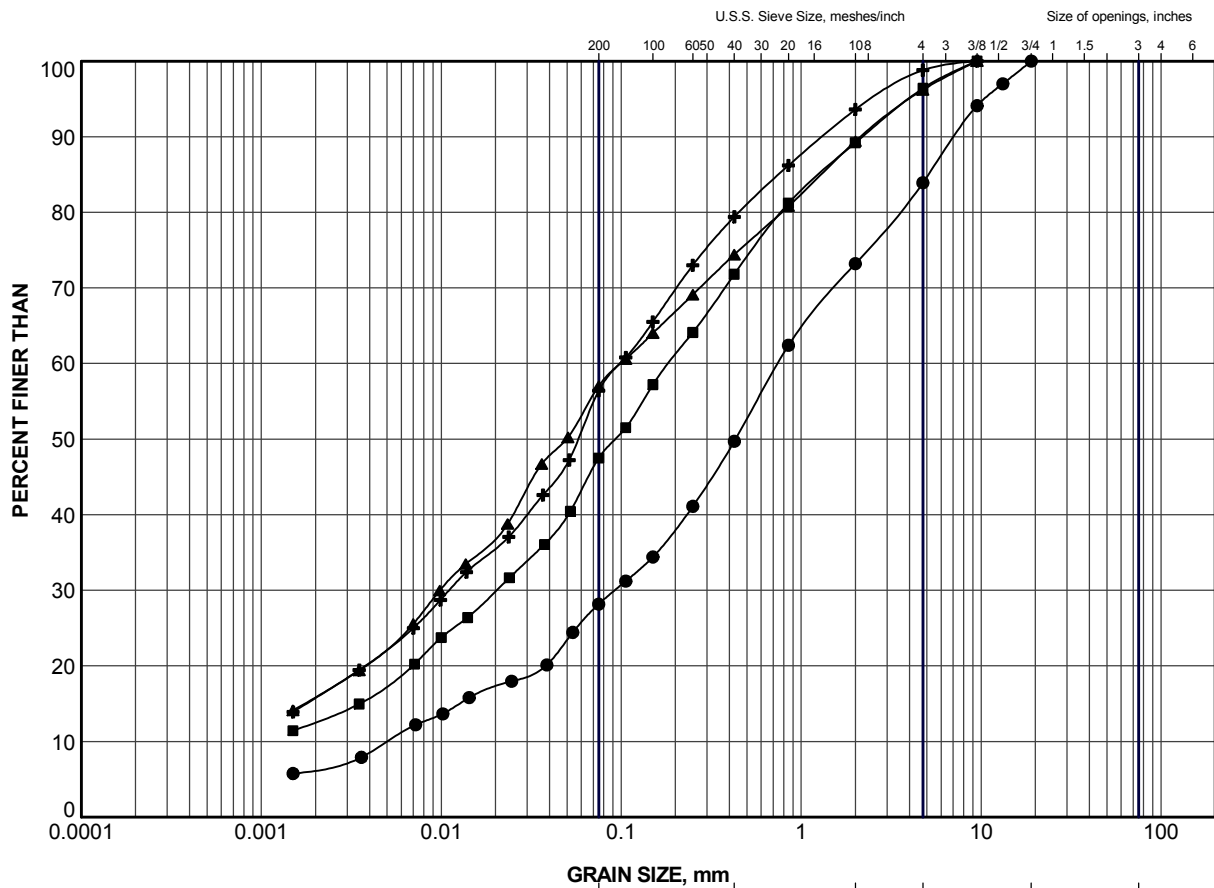
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	5	176.4
■	602	7	174.8
▲	603	6	175.8
+	605	6	176.7
◆	606	5	176.5
◇	606	9	172.6

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL			
PROJECT No.		13-1132-0111		FILE No. 1311320111-1000-F070A2			
DRAWN		WDF		Sep 08/14		SCALE N/A REV.	
CHECK						FIGURE A-2	





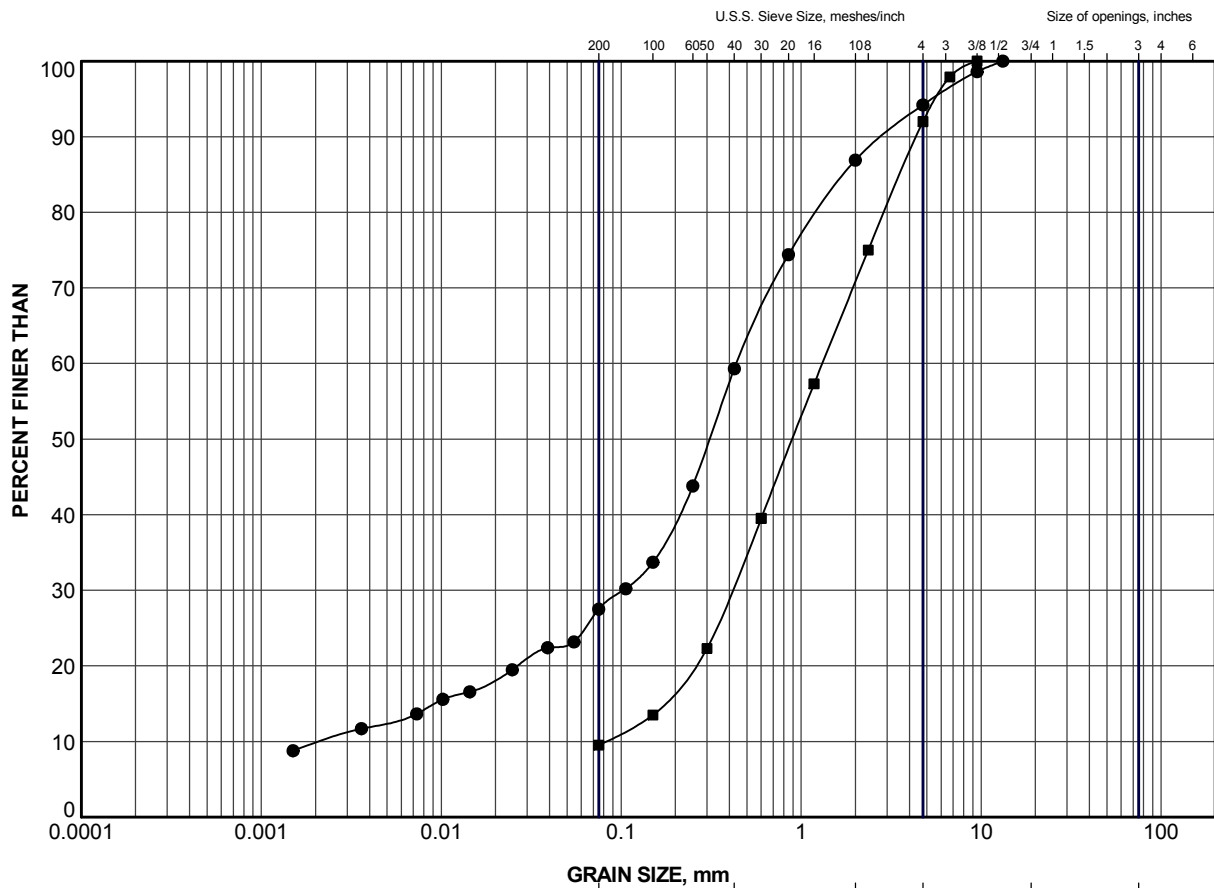
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	10	171.1
■	602	9	172.6
▲	603	14	165.4
+	604	12	168.6

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION SANDY SILT TILL			
PROJECT No.		13-1132-0111		FILE No. 1311320111-1000-F070A3			
DRAWN		WDF		Sep 08/14		SCALE N/A REV.	
CHECK						FIGURE A-3	





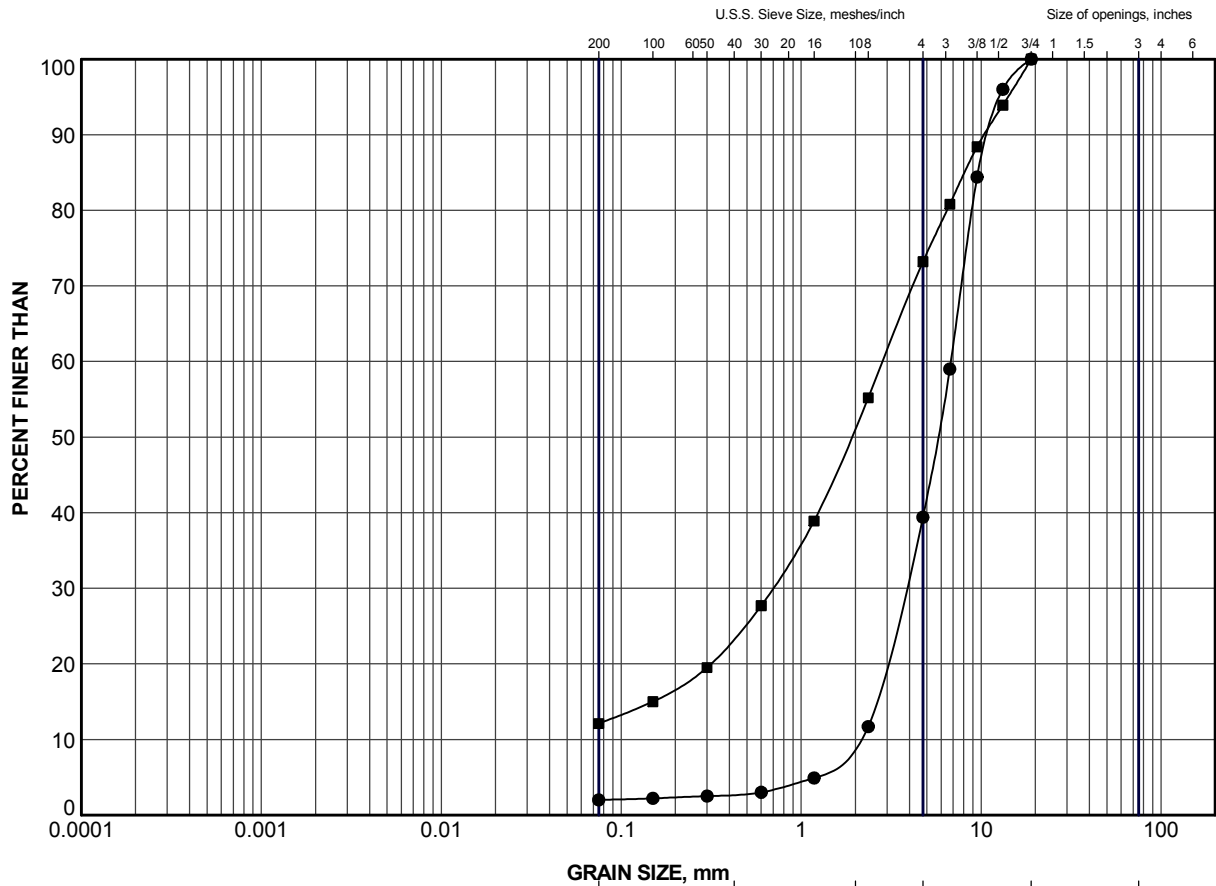
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

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

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND			
PROJECT No.		13-1132-0111		FILE No. 1311320111-1000-F070A4			
DRAWN		WDF		Sep 08/14		SCALE N/A REV.	
CHECK						FIGURE A-4	





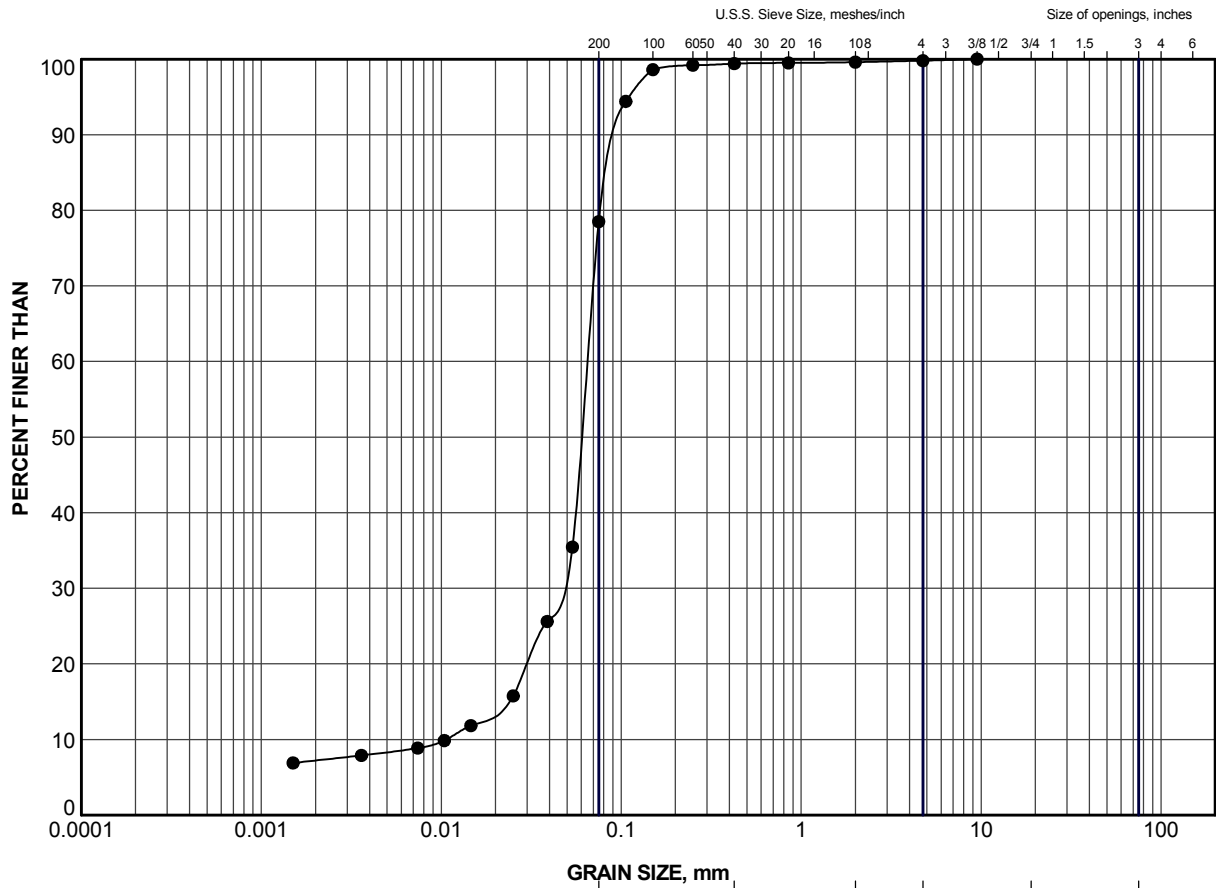
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	603	11	170.0
■	604	17	161.0

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				GRAIN SIZE DISTRIBUTION SAND AND GRAVEL			
PROJECT No.		13-1132-0111		FILE No.		1311320111-1000-F070A5	
DRAWN		WDF		Sep 08/14		SCALE N/A REV.	
CHECK						FIGURE A-5	




LDN_MTO_GSD_GLDR_LDN.GDT 08/09/14

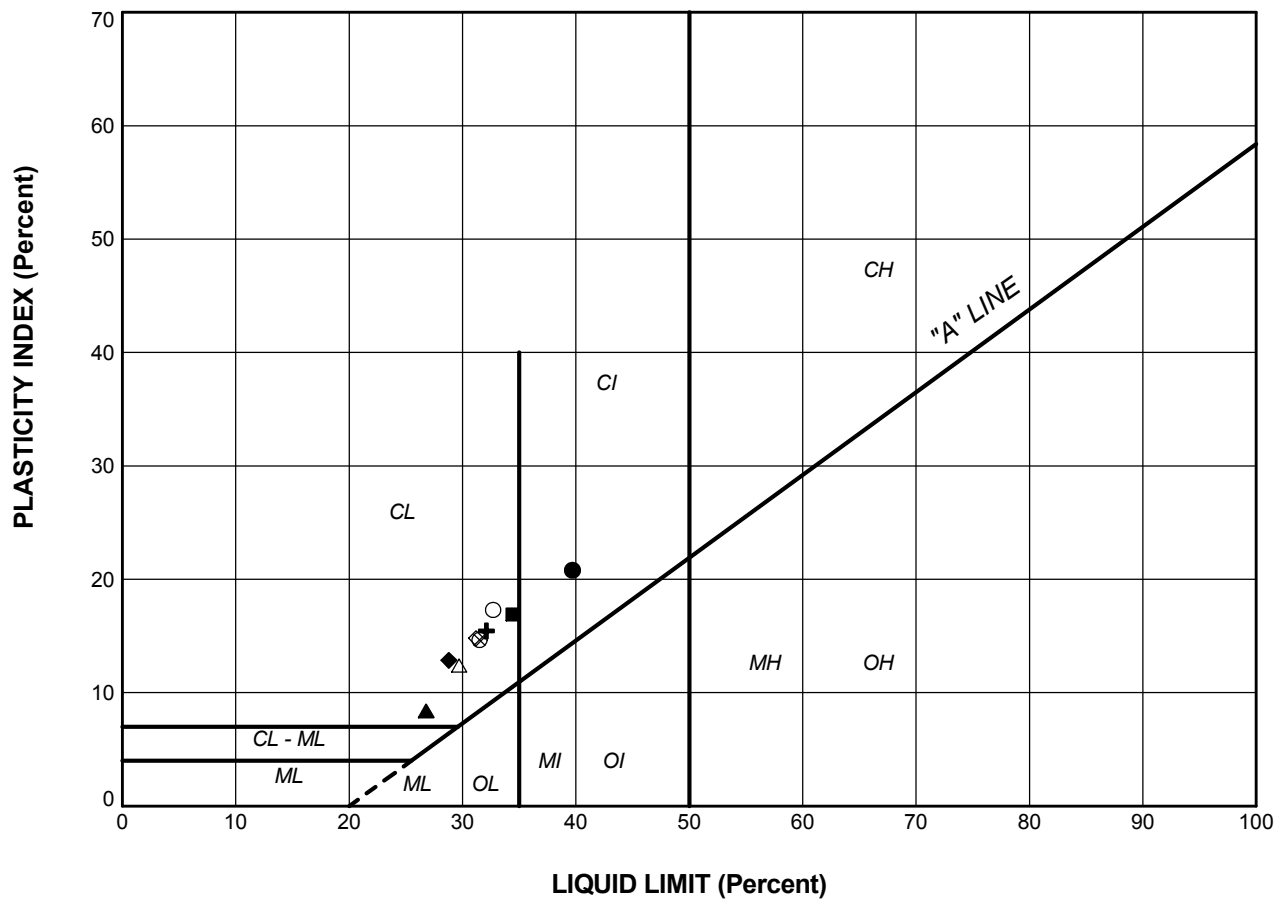


CLAY AND SILT						Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND			
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	601	13	166.5


PROJECT				
RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00				
TITLE				
GRAIN SIZE DISTRIBUTION SILT				
 Golder Associates LONDON, ONTARIO	PROJECT No. 13-1132-0111		FILE No. 1311320111-1000-F070A6	
			SCALE	N/A
	DRAWN	WDF	Sep 08/14	REV.
	CHECK			
			FIGURE A-6	

LDN_MTO_GSD_GLDR_LDN.GDT 08/09/14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	601	2	39.7	18.9	20.8 (FILL)
■	605	3	34.4	17.5	16.9 (FILL)
▲	606	2	26.8	18.4	8.4 (FILL)
+	601	5	32.1	16.7	15.5
◆	602	7	28.8	16.0	12.9
◇	603	6	31.2	16.4	14.8
○	604	7	32.7	15.4	17.3
△	605	6	29.7	17.3	12.4
⊗	606	5	31.5	16.9	14.7

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00				
TITLE								
PLASTICITY CHART								
 Golder Associates LONDON, ONTARIO		PROJECT No. 13-1132-0111		FILE No. 1311320111-1000-F070A7				
		DRAWN	WDF	Sep 08/14	SCALE	N/A	REV.	
		CHECK			FIGURE A-7			



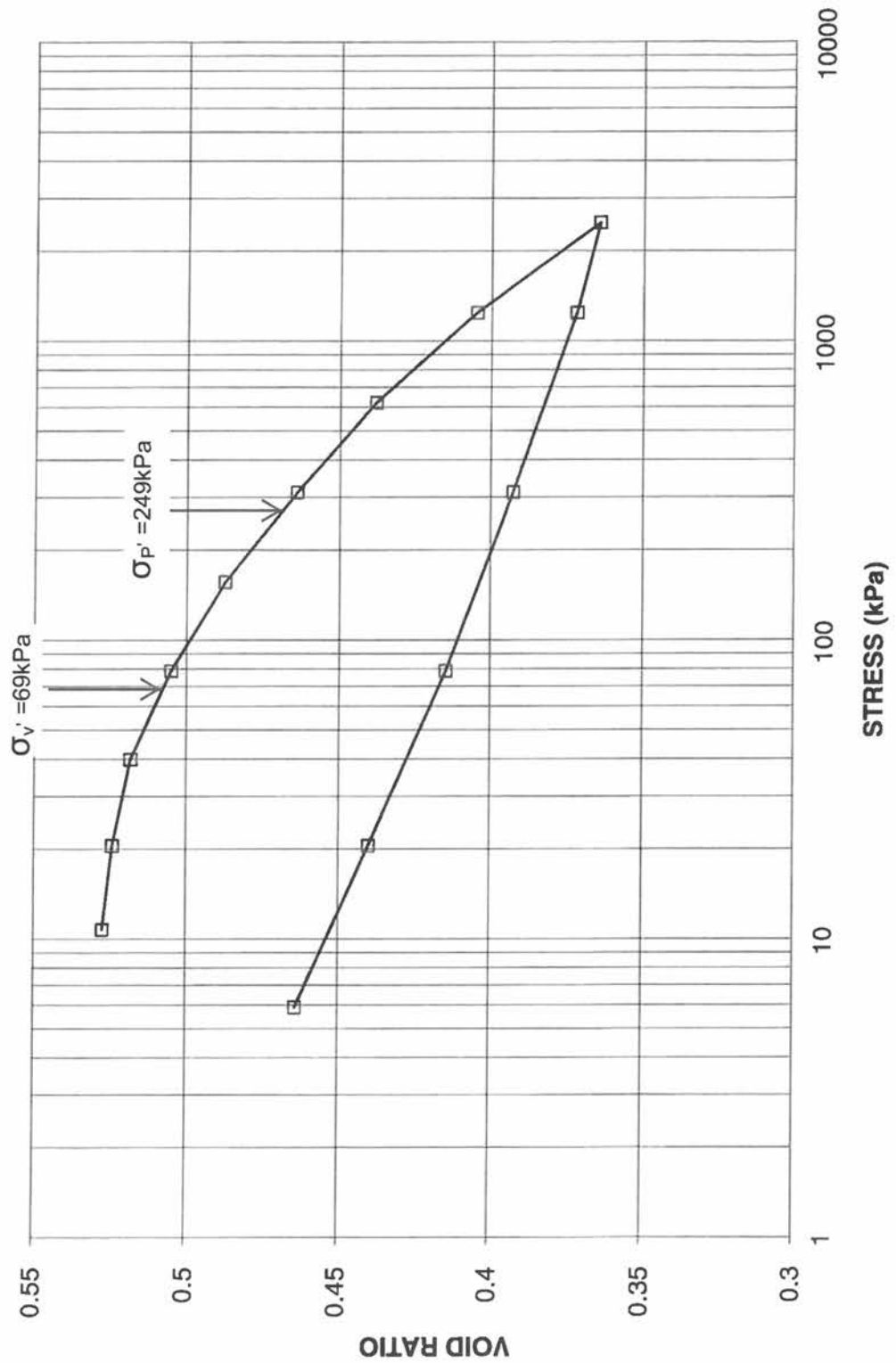
APPENDIX B

Laboratory Test Data – Consolidation Testing

**CONSOLIDATION TEST
VOID RATIO VS LOG STRESS**

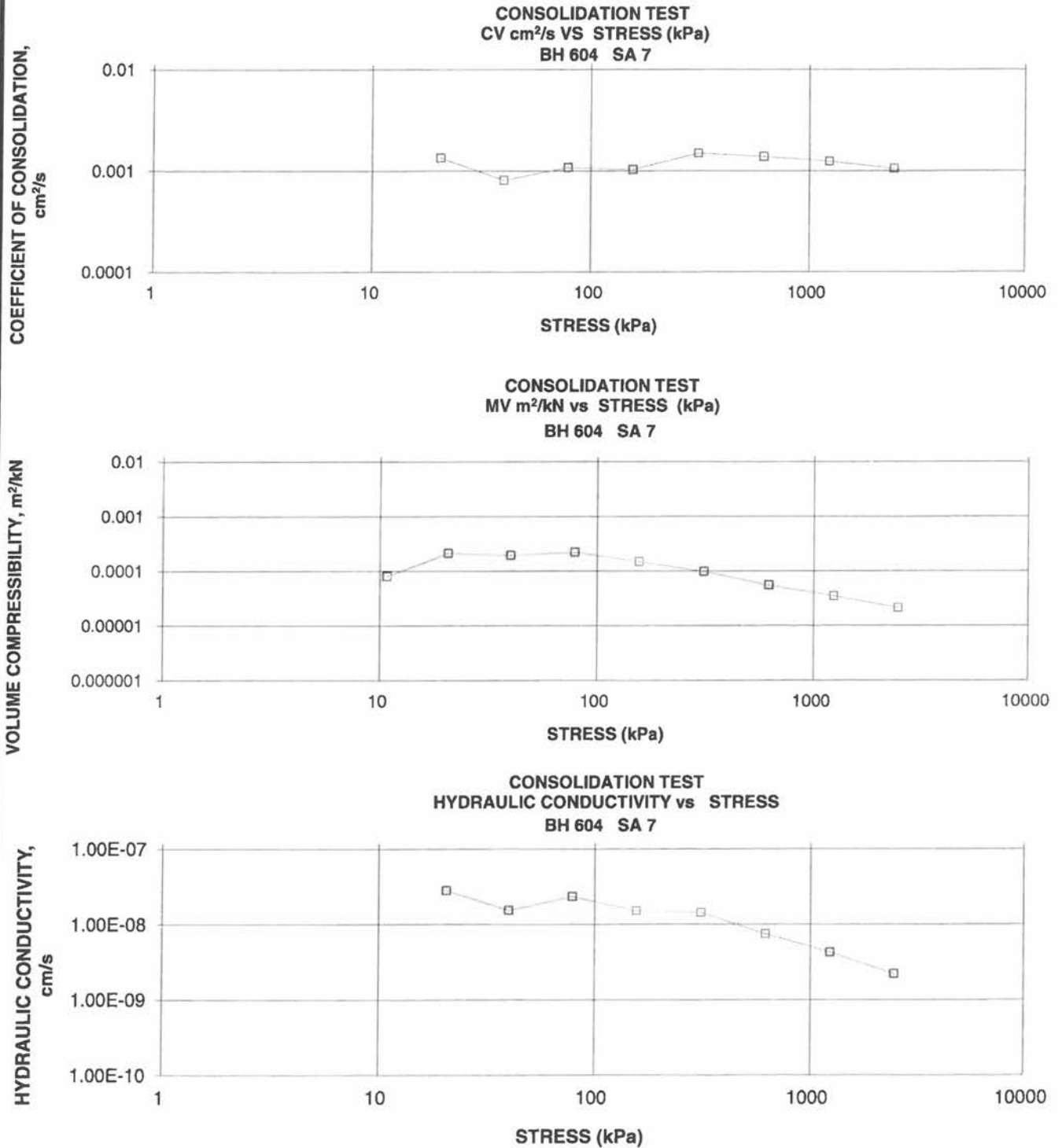
FIGURE B-1A

**CONSOLIDATION TEST
VOID RATIO vs STRESS
BH 604 SA 7**



CONSOLIDATION TEST SUMMARY

FIGURE B-1B



Project No. 13-1132-0111

Prepared By: RD

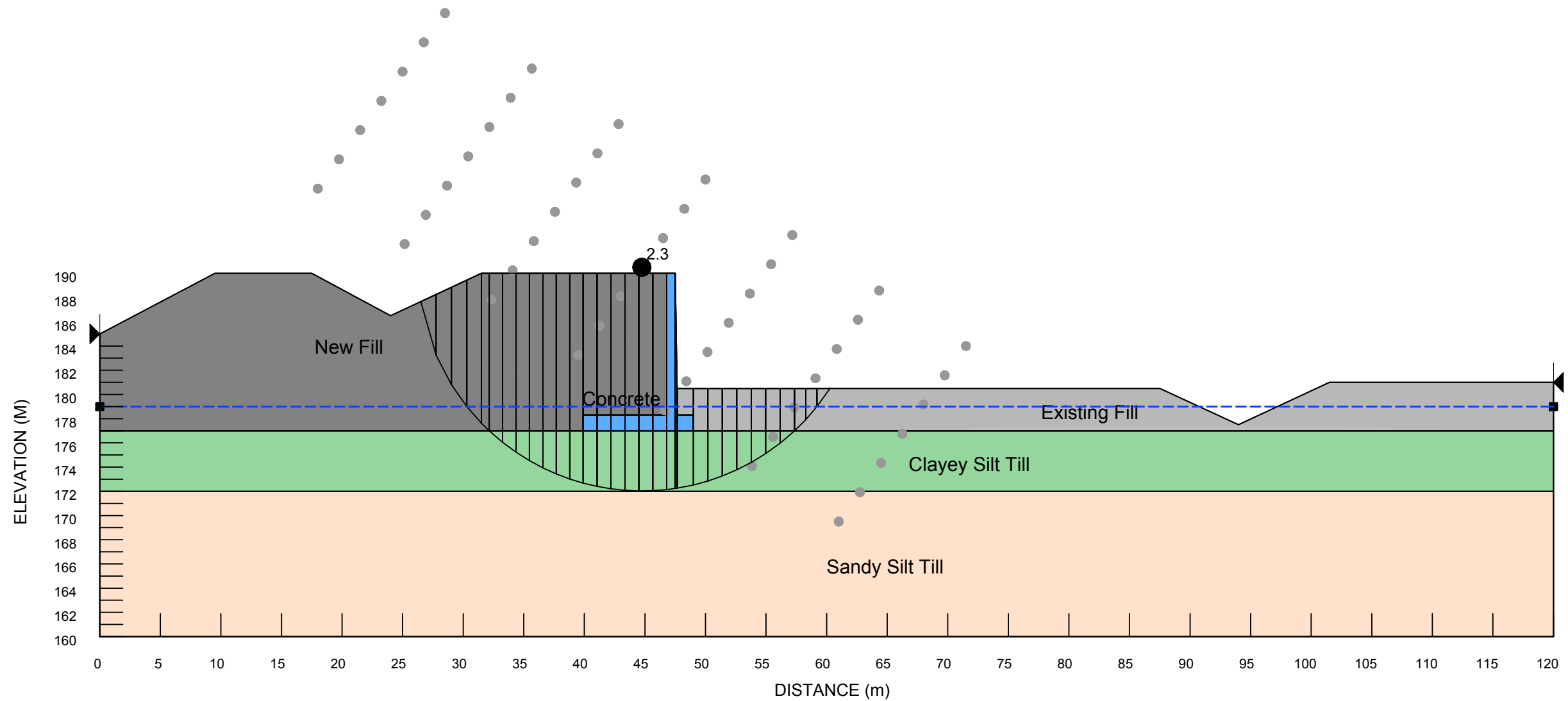
Golder Associates

Checked By: *[Signature]*



APPENDIX C

Slope Stability Analyses



CANTILEVER WALL - UNDRAINED CONDITION

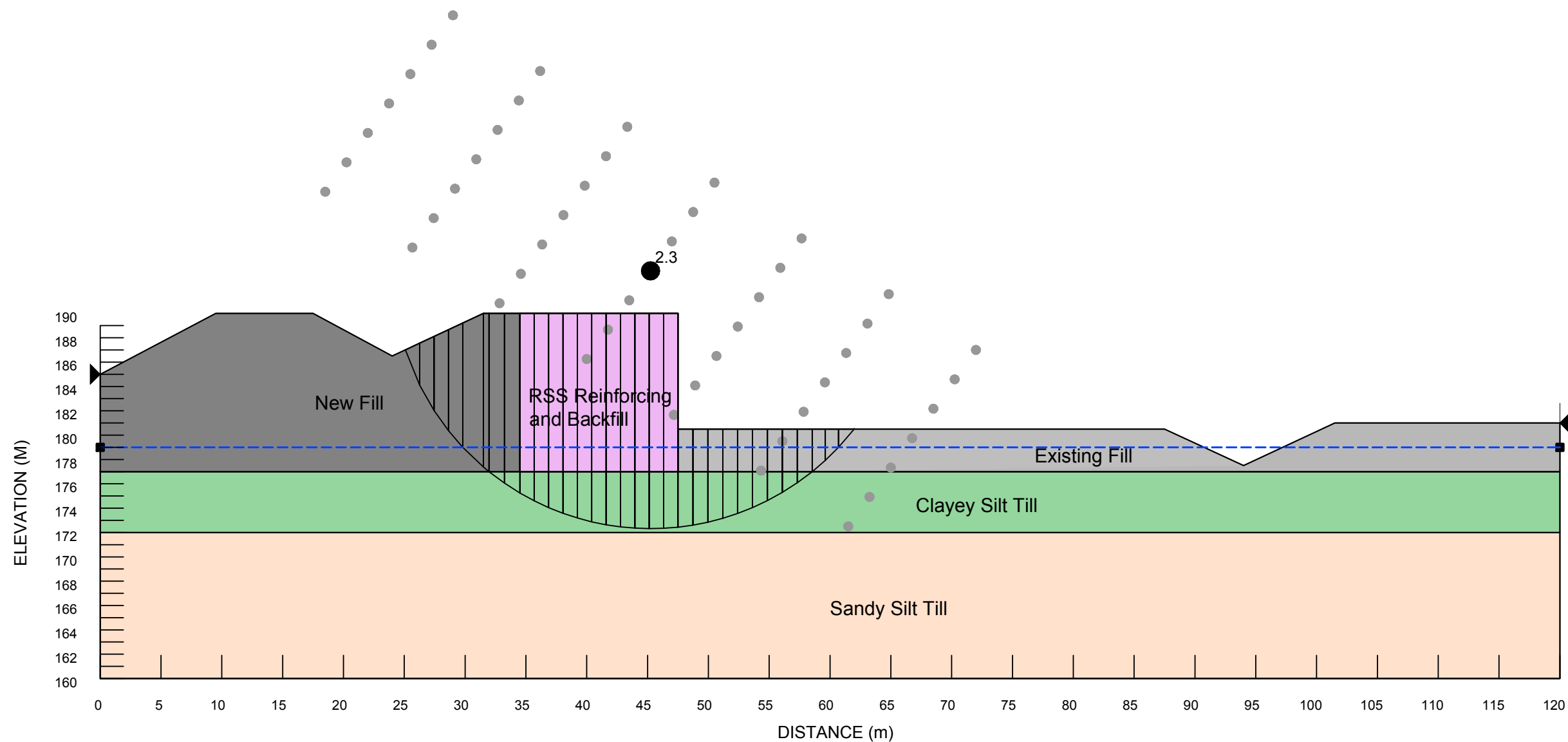
NOTES

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

SOIL PROPERTIES			
Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Existing Fill	20	0	28°
Clayey Silt Till	22	100	0°
Sandy Silt Till	22	0	34°
Concrete	24	0	90°

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				RESULTS OF SLOPE STABILITY ANALYSES			
PROJECT No.		13-1132-0111		FILE No.		1311320111-1000-F070C1	
CADD	WDF	Nov. 07/14		SCALE	AS SHOWN	REV.	0
CHECK				FIGURE C-1			





RSS WALL - UNDRAINED CONDITION

NOTES

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


SOIL PROPERTIES			
Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Existing Fill	20	0	28°
Clayey Silt Till	22	100	0°
Sandy Silt Till	22	0	34°
RSS Reinforcing & Backfill	22	0	90°

PROJECT

RETAINING WALL
HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION
GWP 3093-09-00

TITLE

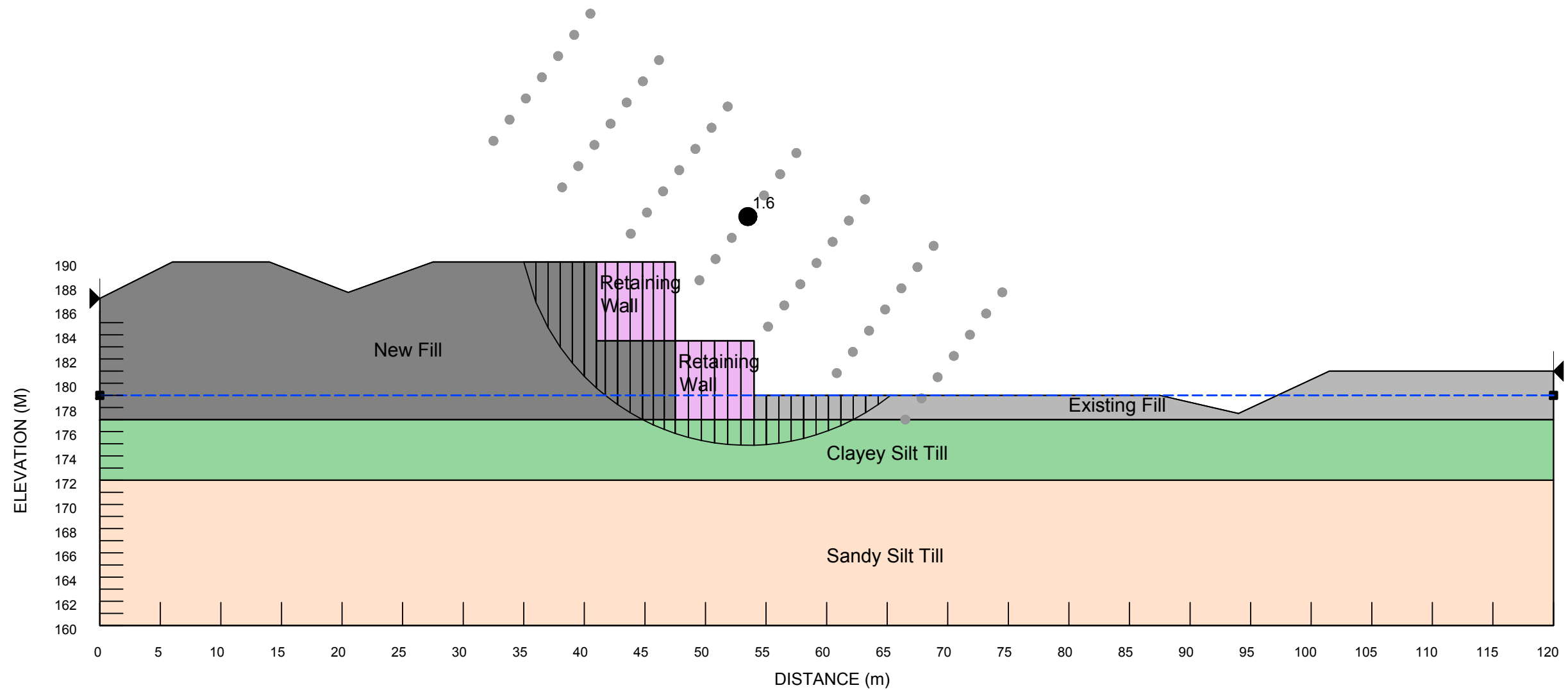
RESULTS OF SLOPE STABILITY ANALYSES

Golder Associates
LONDON, ONTARIO

PROJECT No.	13-1132-0111	FILE No.	1311320111-1000-F070C1
CADD	WDF	Nov. 07/14	
CHECK			

SCALE	AS SHOWN	REV.	0
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FIGURE C-2



TIERED WALL - DRAINED CONDITION

NOTES

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

SOIL PROPERTIES			
Material	Unit Weight (kN/m³)	Cohesion	Phi
New Fill	22	0	35°
Existing Fill	20	0	28°
Clayey Silt Till	22	100	0°
Sandy Silt Till	22	0	34°
Retaining Wall	22	0	90°

PROJECT				RETAINING WALL HIGHWAY 401 / HIGHWAY 40 INTERCHANGE RECONFIGURATION GWP 3093-09-00			
TITLE				RESULTS OF SLOPE STABILITY ANALYSES			
PROJECT No.		13-1132-0111		FILE No.		1311320111-1000-F070C1	
CADD	WDF	Nov. 07/14		SCALE	AS SHOWN	REV.	0
CHECK				FIGURE C-3			





APPENDIX D

Special Provisions - Lightweight Fill Materials

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:

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

TABLE 1 – MATERIAL PROPERTIES

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

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

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

7.2.2.2 Compressive Strength

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

7.2.2.3 Flexural Strength

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

7.2.2.4 Dimensional Stability

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

7.2.2.5 Flammability

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

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

7.2.2.6 Water Absorption

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

7.2.2.7 Chemical Resistance

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

7.2.2.8 Biological Resistance

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

7.2.2.9 Environmental

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

7.3 Polyethylene Sheeting

The polyethylene sheeting shall be 6 mil thick.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

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

9.2 Levelling Pad

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

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

9.3 Polystyrene Installation

- a) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- b) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- c) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with a maximum joint opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- d) Sloping end adjustments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- e) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- f) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- g) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.
- h) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- i) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- j) The top surface and side surfaces of the expanded polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
- k) The side slope of the rigid expanded polystyrene embankment shall be covered with fill material as detailed elsewhere in this contract.
- l) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted prior to the installation of the rigid expanded polystyrene embankments. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

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

10. EQUIPMENT

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

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

11. QUALITY ASSURANCE

11.1 Quality Assurance

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

11.2 Sampling and Testing

11.2.1 General

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

11.2.2 Sampling Frequency

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

11.2.3 Acceptance/Rejection

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

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

12.0 Measurement for Payment

12.1 Actual Measurement

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

13.0 Payment

13.1 Basis of Payment

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

CONCRETE PAD – Item No.

Special Provision

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

1.0 Scope

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

2.0 References

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

Ontario Provincial Standard Specifications, Construction:

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

Ontario Provincial Standard Specifications, Material:

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

3.0 Submission and Design Requirements

3.1 Submission of Shop Drawings

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

4.0 Materials

4.01 Concrete and Concrete materials

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

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

4.02 Burlap

Burlap shall conform to OPSS 1306.

4.03 Moisture Vapour Barrier

Moisture vapour barrier for curing shall conform to OPSS 1305.

4.04 Curing Compound

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

4.05 Water

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

4.06 Joint Materials

Expansion joint filler shall conform to OPSS 1308.

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

4.07 Reinforcement

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

5.0 Construction

5.01 General

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

5.02 Preparation Work

5.02.01 Setting Forms

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

5.03 Joints

5.03.01 General

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

5.03.02 Transverse Joints – Construction

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

5.04 Tolerance

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

5.05 Traffic

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

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

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

5.06 Measurement for Payment

5.06.01 Measurement – Concrete Pad

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

5.07 Basis of Payment

5.07.01 Concrete Pad

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

Special Provision

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

1.0 Scope

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

2.0 References

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

Ontario Provincial Standard Specifications, Construction:

OPSS 904 Construction Specification for Concrete Structures

National Standards of Canada

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

American Society for Testing and Materials (ASTM)

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

3.0 DEFINITIONS

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

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

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

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

4.0 QUALIFICATIONS

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

5.0 SUBMISSION AND DESIGN REQUIREMENTS

5.1 Submission of Shop Drawings

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

The contractor shall submit full details of the following.

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

6.0 MATERIALS

6.01 Concrete and Concrete materials

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

Minimum unconfined compressive strength at 28 days of 0.5 MPa.

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

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

6.02 Water

Water shall be free of any impurities, which would adversely affect the concrete. Mixing water shall conform to the requirements of CSA Standard A23.1. Water of questionable quality shall

not be used unless proven to produce specimens whose 28-day compressive strength is at least 90 % of those made with known acceptable water and an identical material mix.

6.03 Foaming Agent

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

7.0. EQUIPMENT

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

8.0 CONSTRUCTION

8.01 Foundation Excavation

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

8.02 Cellular Concrete Placement

- a) The placement area shall be free of standing water during placement of cellular concrete and until granular material or the next subsequent lift of cellular concrete is placed on top of the completed lift. Snow and ice must be removed from the area prior to placement.
- b) Any items to be fully or partially encased in the cellular concrete shall be properly set and stable prior to the installation of the cellular concrete. The Contractor shall provide positive means of preventing uplift and any other movement of embedded items during installation of cellular concrete.
- c) Where required, formwork shall be designed and installed to withhold cellular concrete, and may require lining with poly sheeting or similar impermeable membrane to prevent leakage.
- d) Cellular concrete may be placed during freezing conditions, provided measures are taken to prevent damage to the cellular concrete until sufficient strength has been attained. Care should be taken to avoid freezing before initial set and insulating systems or heat shall be provided to

prevent freezing of the cellular concrete. Cellular concrete must not be placed during precipitation.

- e) Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling. Initial discharge of cellular concrete that has accumulated in the discharge lines during prior placements or any cellular concrete mix that has not been fully aerated shall be wasted prior to discharge into the intended lift. Cellular concrete shall not be discharged into the intended lift after the foam generator has been turned off.
- f) The maximum lift thickness shall be determined based on density and any other considerations that may affect placement. Cellular concrete shall be cast in a formed area within 1 to 2 hours, to permit undisturbed curing. Foot traffic within the cellular concrete mass shall not be permitted.
- g) Finished surface elevation shall be within ± 25 mm of the design grades shown on the drawings. Cellular Concrete can be placed with a maximum slope of 1%. Slopes greater than 1% will require profiling by creating steps for the Cellular Concrete with formwork.
- h) Loading of, or traffic on the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfill can commence with cellular concrete supports foot traffic without leaving an indentation.

9. QUALITY ASSURANCE

9.01 Quality Assurance

- a) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted prior to the installation of the cellular concrete. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- b) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer, a minimum of one week prior to the commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Upon completion of the cellular concrete embankment filling the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the cellular concrete has been constructed in conformance with the installation procedures and specifications of the contract documents.

9.02 Sampling and Testing

9.02.1 General

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

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

9.02.2 Sampling Frequency and Methods

- a) Cellular concrete samples must be captured, cured, and tested to verify the compressive strength requirement is satisfied. One sample is comprised of one set of three cellular concrete cylinders. One sample should be taken for each placement, or every 100 m³, whichever is more frequent.
- b) Test cylinders shall be cast in 75 mm by 150 mm cylindrical plastic molds. The sample mold must be lined with “freezer paper” with the plastic side against the cellular concrete. Cellular concrete cylinders shall be cured and tested as per ASTM C495-99a, modified to represent the field curing conditions for geotechnical applications.
- c) Fresh cellular concrete density shall be measured and recorded once per production run, or once for every 50 cubic metres, or once per 20 minutes, whichever is more frequent. The density shall be maintained within +/- 10 % of the design density.
- d) A minimum of three cube or core samples of the in-place cellular concrete shall be cut by manual methods for each lift prior to placement of any subsequent lift. Core samples shall be tested for compressive strength in accordance with ASTM D 7012. Manually cut samples shall be tested for compressive strength in accordance with ASTM C109/109M. Wet and dry unit weight shall be tested for all samples. Samples shall be taken at top, middle and bottom of each lift.
- e) In the event of disagreement between the measurements of unit weight or compressive obtained from the test cylinders or those cut/cored from the in-place materials the test results from the in-place materials shall be considered representative.

9.03 Acceptance/Rejection

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

10.0 Measurement for Payment

10.1 Actual Measurement

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

11.0 Payment

11.1 Basis of Payment

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

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