



November 2016

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Ojibway Parkway/ETR Overpass
Sites 6-600/1 & 2 (Bridge B-1)
Highway 401 (Rt. Hon. Herb Gray Parkway)
GWP 3028-14-00
Ministry of Transportation, Ontario, West Region**

Submitted to:

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REPORT



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Geocres No: 40J6-71

Distribution:

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LIST OF ABBREVIATIONS

LIST OF SYMBOLS

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APPENDIX F

Record of Previous Boreholes and Laboratory Testing

Golder Associates Project No. 09-1132-0039-1000

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**PART A
FOUNDATION INVESTIGATION REPORT**

**OJIBWAY PARKWAY/ETR OVERPASS
SITES 6-600/1 & 2 (BRIDGE B-1)
HIGHWAY 401 (RT. HON. HERB GRAY PARKWAY)
GWP 3028-14-00
MINISTRY OF TRANSPORTATION, ONTARIO – WEST REGION**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detailed design work for GWP 3028-14-00. The project involves the design of a crossing for Bridge B-1, Sites 6-600/1 & 2 located at the western terminus of the Rt. Honourable Herb Gray Parkway (RHHGP); formerly Windsor-Essex Parkway, or WEP. This structure will deliver Herb Gray Parkway traffic over Ojibway Parkway, the Essex Terminal Railway (ETR) line and the Plaza Access Road (PAR). This report addresses the construction of Retained Soil Structures (RSS) located between the ETR and Ojibway Parkway.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure by drilling boreholes, conducting in-situ testing and carrying out laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P3-1132-0053 dated June 2015. Golder Associates has carried out extensive geotechnical investigations for the MTO's Windsor-Essex Parkway and the Canadian Inspection Plaza at the proposed Detroit River International Crossing (DRIC). These investigations included geotechnical information in the vicinity of Site B-1 and were utilized in the preparation of this report. The work was carried out in accordance with our Quality Control Plan for Foundation Engineering dated August 14, 2015.



2.0 SITE DESCRIPTION

2.1 General

The proposed extension of the RHHGP is located in the Township of Sandwich, Ontario, just southwest of Windsor, Ontario. The location of the project is shown on the Key Plan, Figure 1. The bridge will cross Ojibway Parkway, the ETR and the PAR. The RHHGP alignment in the immediate vicinity of the bridge generally trends in west to east direction in the area of the site.

The project area around the bridge structure extends from approximately 350 metres west of Ojibway Parkway to 540 metres west of Machette Road. The topography of the site is relatively flat with ground surface elevations gently sloping between about 177 and 179 metres. The ground surface is generally covered with a mixture of low vegetation and small trees. The site is situated on former residential areas.

2.2 Site Geology

The project area is located in the physiographic region of Southwestern Ontario known as the St. Clair Clay Plains, further subdivided into the Essex Clay Plain (encompassing Essex County and the southwestern part of Kent County). The clay plain was locally deposited during the retreat of recent ice sheets (late Pleistocene Era) when a series of glacial lakes inundated the area. Dependent on the glacial ice thickness and glaciolacustrine (glacial lake water) depths, the materials may have been directly deposited at the ice-bedrock contact or, as the lake levels rose when the ice sheets retreated, the soil and rock debris within and at the base of the ice were deposited in a shallow glacial lake water environment. The Essex Clay Plain exhibits grain size distributions consistent with that of a cohesive glacial till but these deposits do not have densities and strengths indicative of materials generally deposited below a grounded ice sheet¹. It is most likely that in the Windsor area, the soils were deposited from the underside of floating ice through a shallow water depth as a broadly graded mud and, therefore, carried little or no weight of the overlying ice.

The quaternary geological mapping indicates a major soil stratum, consisting primarily of silty clay and clayey silt and ranging in thickness of about 20.5 to 22 metres, exhibits a 'till-like' structure by a random distribution of coarser particles within the primarily fine-grained silt and clay matrix (also called 'diamict'). Predominantly, the near-surface clayey soils are generally firm to hard and contain weathering structures consisting of fractures and possible desiccation cracks. Underlying this 'crust', the soil becomes grey-brown and firm to stiff in consistency, indicating historical groundwater level. Below the groundwater level, the majority of soils in the western and southern areas of metropolitan Windsor are soft to firm silty clays and clayey silts. Therefore, it is considered that this deposit is geologically slightly over-consolidated, due to the lack of significant overburden stresses in the project area. The apparent pre-consolidation in the 'crust' is indicated as a result from wetting and drying cycles, fluctuations in the historical groundwater level, and cementation from carbonates and other minerals during the weathering process.

¹ Morris, T.F. 1994. Quaternary Geology of Essex County, Southwestern Ontario; Ontario Geological Survey, Open File Report 5886, 130p



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More typically layered glaciolacustrine silty clay, silty sand, silt, or sand overly the extensive stratum of 'till-like' silty clay, or clayey silt. This interlayering of sands and silts indicate the glacial lake and glacial ice depositional environment. In some areas of the site, a relatively thin stratum on the order of 1 to 6 metres in thickness of very dense, or hard, basal glacial till containing limestone clasts, was found. This stratum overlies the bedrock and is generally referred to as the Catfish Creek Till².

The bedrock encountered, overlying the Precambrian bedrock, is relatively horizontally oriented sedimentary rock of the Paleozoic era. This sedimentary rock formed in shallow marine environments within what is now geologically referred to as the Michigan Basin, a regional bowl-shaped depression permeating through Southern Ontario. The indicated geological mapping suggests bedrock occurring around 30 metres depth, or 143 to 145 metres in elevation. Previous boreholes in the area indicated bedrock occurring at depths between 23 and 28 metres. The bedrock in this area is the limestones of the Devonian Dundee Formation of the Hamilton Group of Formations, and the underlying limestone of the Devonian Lucas Formation of the Detroit River Group of Formations³.

² Morris, T.F. 1994. Quaternary Geology of Essex County, Southwestern Ontario; Ontario Geological Survey, Open File Report 5886, 130p

³ Morris, T.F. and Cousineau, G.R. 1994. Drift thickness, Essex County area (west half), southern Ontario; Ontario Geological Survey, Preliminary Map P.3255, scale 1:50,000



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between November 16th and 18th, 2015 during which time one borehole, BH15-001, and one Piezo-Cone Penetration Test (CPT), CPT-1001 were completed at the proposed abutments for the bridge extension structure. The locations of the current boreholes are shown on Drawing 1. The below table summarizes the borehole locations, ground surface elevations at the borehole locations, and the borehole depths.

Borehole	Location (m)		Ground Surface Elevation (m)	Depth of Borehole (m)
	Northing	Easting		
BH15-001	4682135.6	328411.5	178.81	27.08
CPT-1001	4682192.2	328420.3	178.79	19.5

The investigation was carried out using track-mounted drilling equipment (Soil Max 449) 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 millimetres outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of American Society for Testing and Materials (ASTM) Standard D1586. In the softer deposits, thin-wall tubes were used to procure relatively undisturbed samples. Rock coring procedures were conducted in accordance with ASTM D2113. The rock was cored utilizing NQ size equipment.

The recorded SPT N values are noted on the Record of Borehole sheets. According to ASTM D1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a 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. A hammer operated by a rope and cat-head system was used for BH15-001. The results of the SPT testing as presented on the Record of Borehole sheets 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. For the site stratigraphy, these larger sized particles may include glacial erratics such as cobbles and boulders.

Groundwater conditions in the borehole were observed throughout the drilling operations. The boreholes were backfilled in accordance with the current MTO procedures and Ontario Regulation 903 (as amended).

The field work was monitored on a full-time basis by experienced Golder Associates staff by locating the boreholes in the field, monitoring the drilling, sampling, in-situ testing operations, and logging of the boreholes. The samples were delineated 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 limit determinations were carried out on selected



soil samples. Samples were also sent to our Mississauga office for oedometer testing to determine consolidation properties of the soil and unconfined compression testing of the rock core. The results of the testing are shown on the Record of Borehole sheets with detailed results presented in Appendices A and B.

3.1 In-Situ Cone Penetration Test

The CPT is an in situ testing technique for site characterization studies. The CPT consists of a special cone tip equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a steel sleeve behind the conical tip, and pore water pressure. It is pushed at a constant rate into the ground using a drill rig (ASTM D5778). A nearly continuous (data obtained at 2 cm intervals) approximate stratigraphic profile together with engineering properties, such as undrained shear strength, can be inferred from the results of the CPT. The CPT equipment was advanced using the hydraulic ram system on the drill rig.

A CPT was conducted north of BH15-001. This test location was denoted as CPT-1001. A shallow borehole was advanced through the surface soils using hollow stem augers to a depth of 4.6 metres below ground surface in order to facilitate the start of the CPT. The CPT was advanced to a refusal depth of 19.5 metres below the ground surface or elevation 159.3 metres. The Record of Cone Penetration Test sheet with profiles of tip resistance, pore water pressure during pushing, and sleeve-friction has also been appended.

3.2 Previous Investigations

Golder Associates has carried out several field exploration and laboratory testing programs for the proposed Detroit River International Crossing (DRIC), the Canadian Inspection Plaza, and RHHGP at the Bridge B-1 vicinity. Characterization of the ground conditions was carried out using conventional boreholes with SPT, CPT, in-situ shear strength testing using conventional and Nilcon field vanes, shear tests and in-situ cross-hole and vertical seismic profile testing. The laboratory testing included oedometer testing of the compressible soils. The bulk of this work was reported under Geocres Report No. 40J6-27⁴. The Record of Borehole and Record of Cone Penetration Test sheets for previous boreholes located within the west approach and Bridge B-1 areas are presented in Appendix E along with the relevant results of laboratory testing in their original format. The tables below summarize the locations, ground surface elevations and depths of the previous boreholes.

Table 1 - Summary of Exploration Locations Reported in Geocres Report No. 40J6-27

Borehole/CPT	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)	Type of Hammer
	Northing	Easting			
CPT-165	4 682 188	328 458	178.98	2.29 (Borehole) 23.18 (CPT)	Cat-Head
BH-166	4 682 168	328 350	179.00	26.92	Cat-Head

⁴ Geocres No. 40J6-27, 2009 and 2010: Windsor-Essex Parkway, Geotechnical Data Report (June 2009); and Addendums No. 1 – Soil Chemistry Data (February 2010), Addendum No. 2 – In Situ Cross-Hole and Vertical Seismic Profile Testing (March 2010), Addendum No. 3 – Supplementary Cone Penetration Testing (February 2010), Addendum No. 4 – Supplementary Geotechnical Investigation (March 2010), Addendum No. 5 – Supplementary Laboratory Investigation (April 2010), Addendum No. 6 – Supplementary Geotechnical Investigation (May 2010) and Addendum No. 7 – Supplementary Geotechnical Investigation (June 2010).



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Borehole/CPT	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)	Type of Hammer
	Northing	Easting			
BH-166A (Piezometer)	4 682 168	328 350	179.00	15.39	Cat-Head
CPT-348	4 682 160	328 513	179.15	2.90 (Borehole) 23.40 (CPT)	Cat-Head
BH-349	4 682 136	328 496	179.08	27.79	Cat-Head

Golder Associates' investigations of the Canadian Inspection Plaza to the west of Bridge B-1 were carried out on behalf of Transport Canada. The factual information was reported in Golder Associates Report No. 09-1132-0039-1000-R02 titled "Geotechnical Data Report, Canadian Inspection Plaza, Proposed International Border Crossing of the Detroit River, Windsor, Ontario", dated April 2010. The Record of Borehole and Record of Cone Penetration Test sheets for previous boreholes located within the west approach and Bridge B-1 areas are presented in Appendix F along with the relevant results of laboratory testing in their original format. The table below summarizes the locations, ground surface elevations and depths of the previous boreholes.

Table 2 - Summary of Exploration Locations Reported in Golder Report No. 09-1132-0039

Borehole/CPT	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)	Type of Hammer
	Northing	Easting			
GBH-167	4 682 025	328 316	179.03	27.86	Cat-Head
CPT-167	4 682 027	328 313	178.91	23.12	N/A
CPT-169	4 682 230	328 209	178.57	3.05 (Borehole) 23.42 (CPT)	Cat-Head
GBH-170	4 682 409	328 159	178.70	25.02	Cat-Head
GBH-171	4 682 265	328 114	178.14	3.05 (Borehole) 22.76 (CPT)	Cat-Head
GBH-172	4 682 120	328 054	178.23	30.08	Cat-Head
GBH-193	4 682 284	328 307	178.85	28.24	Cat-Head



4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

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

The boreholes drilled at the site generally encountered the existing granular fill or topsoil overlying variable fill materials. Underlying the fill materials were native sand, sandy silt and silt layers of varying thicknesses. Beneath these layers, in sequence, occurs clayey silt, silty clay, clayey silt then limestone (bedrock). The stratigraphy encountered in BH15-001 was consistent with that revealed by previous investigations.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profile, are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and the simplified stratigraphic sequence encountered in the vicinity of Bridge Site B-1 is summarized below:

- **Topsoil or Pavement** structures
- **Fill:** Very loose to compact fill materials consisting primarily of sand, silt and gravel with evidence of organic material were encountered. Variation in fill thicknesses indicate extent of past construction activities.
- **Native Granular Deposits:** Relatively thin layers, on the order of about 2 metres or less of very loose to dense but generally compact surficial granular soils composed primarily of sand, gravel and silt were encountered in many boreholes and CPT locations.
- Extensive deposits consisting of silty clay to clayey silt were encountered and have been separated into two geologic units for the purpose of this report:

Upper Silty Clay/Clayey Silt: Above approximately elevation 163 metres, the soft to very stiff clayey silt to silty clay deposit exhibits significantly greater variability in water content and is typically firm and of lower strength than the soils below. Based on sample interpretation and water content variability, it is likely that this deposit is highly layered with silt and plastic clay (i.e. laminated or varved). The Upper Silty Clay/Clayey Silt is generally of low plasticity (CL) but contains discrete layers with borderline (ML/CL), intermediate (CI) and high (CL) plasticity.

Lower Silty Clay/Clayey Silt: Below approximately elevation 163 metres, the firm to hard clayey silt to silty clay deposit exhibits relatively little variability in water content and is typically stiff and of higher strength than the above soils, and, based on this evidence, is more homogeneous and is more “till-like” in composition. This layer is consistently of low plasticity (CL).

- **Bedrock:** Dolomitic limestone bedrock was encountered at depths of between 23 and 26 metres below the existing ground surface, or at elevations ranging from about 152 to 156 metres.



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Table 3 summarizes the soil stratigraphic sequence encountered along with ranges in standard penetration test and water content test data. Tables 4 and 5 summarize field vane shear test data for the two geologic units within the silty clay to clayey silt deposit. Table 6 summarizes the results of laboratory oedometer tests. Field and laboratory test data are provided on the Record of Borehole sheets and in Appendices A, E and F.

Table 3 - Stratigraphic Summary

Strata ⁵	Thickness Range, Average (m)	SPT N Value Range, Average	Water Content Range, Average (%)	Plasticity Index Range, Average
Topsoil ⁶	0 – 0.9, 0.4	NA	NA	NA
Fill	0.1 – 1.7, 0.7	6 – 12; 10	16 – 22; 20	NA
Granular Deposits	0.5 – 2.1, 1.4	2 – 34, 15	12 – 25, 18	NA
Upper Silty Clay/Clayey Silt	10.0 – 14.1, 11.6	WH ⁷ – 17, 4	16 – 62, 32	6 - 47, 17
Lower Silty Clay/Clayey Silt	7.2 – 12.7, 10.7	WH – 76, 13	8 – 31, 20	10 – 17, 13

Table 4 – Summary of Field Vane Shear Strength Data (Upper Unit)

Borehole	Approximate Elevation	Shear Strength (kPa)	Vane Sensitivity
BH15-001	173.5	49	3.1
BH15-001	173.3	46	1.5
BH15-001	172.6	29	2.0
BH15-001	172.3	36	1.7
BH15-001	171.0	22	2.7
BH15-001	170.7	20	1.4
BH15-001	169.5	23	2.1
BH15-001	169.2	27	2.7
BH15-001	168.0	19	1.9
BH15-001	167.7	23	1.8
BH15-001	166.5	32	1.6
BH15-001	166.2	30	1.3

⁶ 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.

⁷ WH indicates that the sampler was advanced using only the weight of the sampling hammer



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Table 5 – Summary of Field Vane Shear Strength Data (Lower Unit)

Borehole	Approximate Elevation	Undrained Shear Strength (kPa)	Vane Sensitivity
BH15-001	165.0	24	1.4
BH15-001	165.0	30	1.2
BH15-001	163.4	33	1.5
BH15-001	163.1	29	1.2
BH15-001	161.9	43	1.4
BH15-001	160.4	88	1.2
BH15-001	160.1	109	1.2
BH15-001	157.3	34	0.5
BH15-001	157.0	37	0.4

Table 6 - Oedometer Test Results

Borehole	Elevation (m)	Initial Void Ratio	Compression Index, C_c	Recompression Index, C_R	Interpreted Preconsolidation Pressure (kPa)
BH15-001	171.6	1.06	0.473	0.047	182
BH15-001	168.5	1.11	0.522	0.053	205
GBH-167	174.2	0.91	0.321	0.027	234
GBH-167	168.4	1.43	0.450	0.058	71
GBH-167	163.8	0.58	0.135	0.018	228
BH-349	173.4	1.40	0.491	0.054	155
BH-349	168.5	0.77	0.219	0.021	132
BH-349	163.9	0.58	0.168	0.021	236

The bedrock encountered in all boreholes was very fine to medium grained, brown to mottled coloured limestone with a slight to fresh weathering surface, slight to vuggy porosity with occasional pitting. The limestone exhibited some fossils and contained slight hydrocarbon staining. Table 7, below, summarizes rock core data for the limestone encountered.

Table 7: Summary of Rock Core Data

Borehole	Sample	Elevation (m)	TCR (%)	SCR (%)	RQD (%)
BH15-001	19	155.3 – 154.8	40	0	0
	20	154.8 – 153.3	99	91	91
	21	153.3 – 151.7	92	82	75
BH-166	15	155.7 – 154.3	89	89	81
	16	154.3 – 153.6	100	98	97



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Borehole	Sample	Elevation (m)	TCR (%)	SCR (%)	RQD (%)
	17	153.6 – 152.1	100	100	100
GBH-167	16	154.4 – 153.1	88	65	60
	17	153.1 – 151.5	94	93	91
	18	151.5 – 151.2	100	90	90
GBH-172	18	152.3 – 150.6	98	83	70
	19	150.6 – 149.1	93	92	93
	20	149.1 – 148.2	100	100	100
GBH-193	17	154.1 – 153.3	77	55	50
	18	153.3 – 151.5	100	100	100
	19	151.5 – 150.6	100	100	100
BH-349	20	156.1 – 154.6	100	93	92
	21	154.6 – 153.3	82	74	75
	22	153.3 – 152.7	100	92	98
	23	152.7 – 151.3	98	98	98
Average			92	84	82

4.2 Groundwater Conditions

The groundwater conditions in the overburden deposits were not established in BH15-001 during drilling due to the use of mud rotary drilling. However, artesian conditions were encountered at the bedrock interface. Groundwater monitoring instrumentation installed during the previous investigations consisted of a conventional piezometer in borehole GBH-166A, nested vibrating-wire piezometers (VWP) in GBH-349 and a single VWP in GBH-193. Table 8 presents the groundwater conditions observed on site during drilling activities.

Table 8: Summary of Elevations at which Groundwater First Encountered

Borehole	Ground Surface Elevation (m)	Encountered Elevation (m)	Date of Measurement
BH15-001	178.8	Artesian at 155.3	November 18, 2016
CPT-165	178.9	177.3	August 13, 2008
BH-166*	179.0	180.6	Sept 17, 2008
GBH-167*	179.0	177.7	May 25, 2009
CPT-169	178.6	177.4	June 4, 2009
GBH-170*	178.7	177.3	May 25, 2009
CPT-171	178.1	175.8	June 4, 2009
GBH-172*	178.2	177.2	May 19, 2009
GBH-193	178.9	177.5	June 8, 2009
CPT-348	179.2	177.8	April 27, 2010
BH-349	179.1	177.7	April 22, 2010

* Flowing artesian conditions encountered at bedrock interface or occurred during rock coring.



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Flowing artesian conditions were encountered at the overburden-bedrock interface during drilling in boreholes BH15-001, GBH-167 and GBH-170, and during rock coring in boreholes BH-166 and GBH-172. The artesian water flow often occurred in conjunction with a hydrogen sulfide odour. Where artesian conditions were encountered, the boreholes were sealed with a cement-bentonite grout. Table 9 presents piezometer installation details and measured groundwater pressure head elevations.

Table 9: Summary of Groundwater Pressures

Borehole	Ground Surface Elevation (m)	Installation Tip Elevation (m)	Pressure Head Elevation (m)	Date of Measurement
BH15-001	178.81	151.88	182.07	November 18, 2015
			181.92	January 13, 2016
			182.38	May 16, 2016
BH-166A	179.00	163.6	163.76	Sept 19, 2008
			165.19	Sept 22, 2008
			178.43	Jan 28, 2009
GBH-193	178.85	151.0	178.95	June 9, 2009
			180.35	Aug 26, 2009
			180.35	Nov 1, 2009
BH-349	179.08	173.4	180.4	June 2, 2010
		168.5	180.4	June 2, 2010
		164.0	178.9	June 2, 2010
		155.6	179.8	June 2, 2010

Groundwater conditions observed during drilling do not represent long-term static water levels because of the influence of drilling operations, localized variability of stratigraphic conditions and local variations in soil permeability. Groundwater conditions observed within the native granular soils and fill and groundwater pressures within the granular soils (where present) near the bedrock surface and within the bedrock will all be influenced by seasonal conditions and precipitation. Therefore, groundwater levels within the native granular soils and groundwater pressures near and within the bedrock should be expected to vary and should be measured before and at the time of construction.

4.3 Subsurface Gases

Hydrogen sulfide gas was encountered in boreholes GBH-167, GBH-170 and GBH-172 at the soil-bedrock interface. The presence of the characteristic 'rotten egg' odour signified hydrogen sulfide, but the levels were less than 10 parts per million (ppm), which is required to trigger the monitoring equipment used by the field personnel. Hydrogen sulfide gas has been reported in other exploratory holes within the Dundee Formation due to its



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reputation as being the largest oil producing formation in Southwestern Ontario⁸. The hydrogen sulfide gas was encountered with observed artesian water flows discussed in Section 4.2.

The groundwater in the project area contains dissolved hydrogen sulphide (H_2S) that is liberated from the water on exposure to atmospheric pressure. Hydrogen sulphide gas can frequently be detected by odour at concentrations on the order of 0.5 ppm and can be corrosive at concentrations of about 2 ppm to 3 ppm (Powers et al, 2007) as measured in the groundwater. Other investigations carried out near Ojibway Parkway and Sandwich Street encountered H_2S gas in concentrations sufficient to trigger personnel health and safety monitoring equipment on several occasions. Active ventilation of drilling areas with construction fans and use of controlled density drilling fluids were required to continue drilling at some locations for these nearby explorations. Similar precautions were undertaken for this project. Measurement of hydrogen sulfide gas (H_2S) concentrations in 29 water samples taken from the observation wells and boreholes completed for Geocres No. 40J6-28 indicated a range from a minimum value less than the detection limit to a maximum value of 238 ppm. For samples in which H_2S was detected, excluding the maximum value of 238 ppm non-detection values, the maximum and minimum values were 5.54 ppm and 0.03 ppm, respectively.

Dissolved methane, CH_4 , was also detected within the groundwater. Dissolved methane concentrations in the water ranged from less than 5 parts per billion (ppb) to a maximum measured value of 485 ppb. No trends in the data were observable with respect to the geographic observation well locations⁹.

4.4 Corrosivity Conditions

Analytical testing was carried out on a soil sample to assess the corrosivity of the soils for the design bridge structure. The analysis consisted of testing one sample from BH15-001, SA6 at a depth from 4.5 to 5 metres below the ground surface. The sample was submitted to a specialist analytical laboratory for testing and the summary is provided below:

Sample Location	BH101 – Sample 6
Sample Depth and Date	4.5 – 5.0 metres; Nov. 16, 2015
Soil Resistivity	840 ohm-cm
Soil Conductivity	1180 umho/cm
Redox Potential	+227 mV
Sulphate Concentration	1100 ug/g
Chloride Concentration	81 ug/g
Soil pH	7.75

The certificates of analyses are provided in Appendix G.

⁸ Luczaj et al, AAPG Bulletin, V. 90, No.11 (November 2006) pp.1787-1801: Fractured hydrothermal dolomite reservoirs in the Devonian Dundee Formation of the central Michigan Basin.

⁹ Geocres No. 40J6-28, June 2009 (revised January 2011): Subsurface Conditions Interpretation Report.



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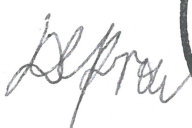
5.0 MISCELLANEOUS

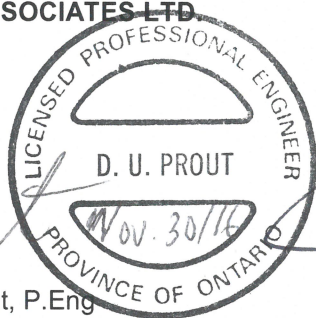
This investigation was carried out using equipment supplied and operated by Lantech Drilling Services, an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Anthony Pusic, E.I.T., under the direction of Field Investigation Manager, Mr. David Mitchell. The CPT testing was carried out by Mr. Alex Szot, E.I.T.


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

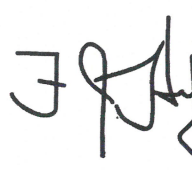
The report was prepared by Mr. William Hanson, E.I.T. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Dr. Storer J. Boone, P.Eng, a Principal with Golder Associates. An independent quality review of this report was carried out by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

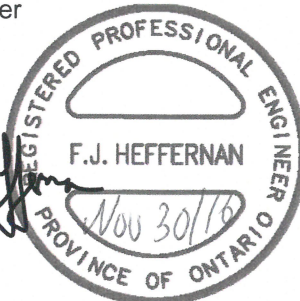
GOLDER ASSOCIATES LTD.


Dirka U. Prout, P.Eng.
Project Engineer




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


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



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**PART B
FOUNDATION INVESTIGATION REPORT**

**OJIBWAY PARKWAY/ETR OVERPASS
SITES 6-600/1 & 2 (BRIDGE B-1)
HIGHWAY 401 (RT. HON. HERB GRAY PARKWAY)
GWP 3028-14-00
MINISTRY OF TRANSPORTATION, ONTARIO – WEST REGION**



6.0 ENGINEERING RECOMMENDATIONS

This section of the report provides recommendations for the foundation aspects of the design of the Ojibway Parkway/ETR Overpass, Bridge B-1. 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 of 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 bridge structure is to be constructed across the Ojibway Parkway, ETR and the Plaza Access Road (PAR) under construction at the time this report was prepared. The foundations are to be designed such that traffic loads, bridge loads and soil loads are sufficiently carried. At the time of this report, the east approach of the bridge structure was constructed and paved. The west approach of the bridge structure was constructed under a contract to the Windsor-Detroit Bridge Authority (WDBA). Fill placement was completed in Spring 2016. The east approach was constructed to a maximum height of 9.5 metres above the original ground surface and the west approach is to be constructed to a maximum height of 10.5 metres above the original ground surface. The ground surface elevation, previous to construction activities, along the length of the bridge structure varies from about 180.6 to 178.8 metres.

It has been indicated by AECOM that two primary bridge design options have been considered:

- i) a single bridge structure; and
- ii) a two-structure crossing with intermediate abutments with an elevated roadway section located between the ETR and Ojibway Parkway. The intermediate roadway section will be supported by fill materials enclosed within a Retained Soil System (RSS) structure hereafter known as 'the island' concept.

According to the General Arrangement (GA) drawing the single bridge structure is proposed to consist of either three or four spans. The bridge structure is to extend from approximately Station 9+860 to 10+030.

Based on the GA dated May 2015, the two structure alternative for Bridge B-1 is proposed to consist of a two span structure over the proposed PAR and the ETR and a single span structure over the Ojibway Parkway. The western structure is planned to include integral abutments and the eastern structure is to have conventional semi-retaining wall abutments. The island between the structures is proposed to be built using two-stage RSS walls. The westerly structure is to span from approximately Station 9+864.1 to 9+934.1, the island from Station 9+934.1 to 9+978.8 and the easterly structure from approximately Station 9+978.8 to 10+023.8. Based on a November 27, 2015 meeting with the project team, the single bridge structure option was selected as the preferred technical alternative. Table 1 presents a comparison of the structure types from a geotechnical perspective.



6.2 Foundations

The soils encountered at this project site typically consist of surficial topsoil and/or fill underlain by silty clay and clayey silt over bedrock. The bedrock was encountered at about elevation 155.7 metres at the west approach and at about elevation 156.1 metres at the east approach. Groundwater was encountered in the upper granular deposits between elevations 177.3 and 177.8 metres. Flowing artesian conditions were encountered at the soil-bedrock interface.

It is recommended that the bridge pier and abutment foundations consist of end-bearing piles driven to refusal on the bedrock. Shallow foundations will not provide the support required for the bridge piers and/or abutments due to the presence of relatively low-strength, normally consolidated clayey silt to silty clay deposits.

6.2.1 Deep Foundations

The abutments and piers can be founded on end-bearing steel H piles driven to refusal onto the underlying bedrock. Steel tube piles are feasible but are not considered further due to concerns with vibrations and soil displacements on nearby infrastructure, the artesian groundwater conditions at the bedrock interface and the proposed use of integral abutments. Steel tube piles are seldom used for integral abutment bridges. Most of the piles will be driven in close proximity to existing utilities. To minimize the potential for vibration related damage, the steel tube piles must be driven open ended for low displacements. However, this will result in a conduit for artesian groundwater flow, H₂S and methane migration if the interior of the tube is not effectively sealed or driving shoes are used. From a geotechnical perspective, drilled shafts (caissons) are a feasible option. However, the installation of caissons may not be desirable from a constructability or economic viewpoint. The depth to bedrock is in the range of 22 to 24 metres. Due to the depth range and the presence of flowing artesian groundwater conditions within the bedrock, it would be necessary to construct the drilled shaft using temporary casing and controlled-density slurry methods. Pneumatic pumps may be required to remove debris at the base of the hole and the subsequent concrete placement will need to be placed using tremie methods. Further, specially formulated drilling fluids will be necessary to control the artesian groundwater pressure to prevent possible 'blowouts' and the release of hydrogen sulphide, if and when present. For these reasons, use of driven steel H-piles is preferred for deep foundations. A comparison of various foundation alternatives is presented on Table II. The costs are relative estimates meant to provide an order of magnitude comparison for the alternatives for foundation engineering purposes and should not be considered to be indicative of actual construction costs.

Driven HP 310 x 110 or HP 310 x 132 piles may be designed using factored geotechnical resistances at Ultimate Limit States (ULS) of 2,000 kilonewtons (kN) and 2,400 kN, respectively. A geotechnical reaction at Serviceability Limit States (SLS) is not provided since bedrock is considered an unyielding medium for the purpose of geotechnical design. A load in excess of the structural pile capacity would be required to exceed the MTO specification of 25 millimetres of settlement. Frost protection must be provided for the pile caps in the form of 1 metre earth cover or thermal equivalent. The factored geotechnical resistance at the abutment areas will have to be adjusted due to downdrag load arising from the settlement of the compressible clayey silts to silty clays upon placement of the embankment fills.



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The following table presents the approximate pile driving depths based on the encountered bedrock surface and the design information provided for the two single structure options.

Design Option	Pile Location	Assumed Cut-off Elevation ¹ (m)	Approximate Pile Tip Elevation (m)	Approximate Pile Length (m)
Single Structure	West Abutment	186.0	155.7	30.3
	Pier #1	177.0	155.6	21.4
	Pier #2	178.0	155.6	22.4
	East Abutment	185.0	156.1	28.9

Note: 1) Cut-off elevations based on General Arrangement Drawing dated June 2016 and submitted for 90% Design Review Submission.

The steel H-piles should be installed and monitored in accordance with Ontario Provincial Standard Drawing (OPSD) 3000.150, OPSS 903 and SS103-11 (Pile Driving Control). A pile note should be added to the foundation drawing that states that the piles are to be driven to bedrock using a maximum ultimate resistance of two times the factored ULS value.

If integral abutments are required, pre-augering is not required. However, installation of the driven piles in pre-augered holes is recommended in Section 6.12.2 to reduce vibration related damage. If a corrugated steel pipe (CSP) liner is required, then it should be filled with loose uniform sand meeting the MTO specifications placed in the upper 3 metres of pile.

6.3 Negative Skin Friction

Depending on the relative timing of embankment fill placement at and near the abutments, and pile installation, embankment fills could induce significant downdrag loading that will need to be accounted for in the assessment of the structural loading of the piles. Construction of the west approach embankment had been recently completed at the time this report was prepared. The west approach embankment work was carried out by the WDBA as part of an advanced contract for the DRIC Canadian Inspection Plaza (CIP) which involves construction of the PAR, utility relocations and early grading. Golder Associates prepared a geotechnical design report for this early contract. Downdrag loads at this location can be mitigated by:

- installing the piles as long as practicable after fill placement;
- use of lightweight fill in the abutment area that could consist of expanded polystyrene (EPS blocks), tire derived aggregate (TDA), cellular concrete, water-cooled blast furnace slag or a combination of these;
- use of friction reducers such as bitumen coating;
- use of heavier pile sections;
- installation of isolators which prevent direct contact between the pile and soil such as pile sleeves or bentonite slurry only to the extent, however, that these would not facilitate artesian water flow.

Delaying installation of the piles, use of lightweight fills and heavier pile sections are considered the most feasible and cost effective mitigation options. Friction reducers and isolators are relatively expensive. The latter may not adequately control upward artesian water flow.



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The 9 metre high east approach embankment between Stations 10+030W and 10+500W was constructed in three stages during the period April 2012 to April 2013. A 1 metre high surcharge layer was placed in April 2013 during the third fill stage and was removed in June 2014 immediately prior to placement of the pavement granulars. A test embankment was constructed between Stations 10+500W and 10+600W during the period of May 2012 to May 2013. The 1 metre high surcharge layer was placed in late April/early May 2013 during the third fill stage and was removed in June 2014 immediately prior to placement of the pavement granulars. The east approach and test embankment were paved approximately 6 months after placement of the roadbase granulars. Both embankments were instrumented with vibrating wire piezometers, settlement plates and inclinometers. Collection and interpretation of the instrument data was carried out by a geotechnical consultant retained by the Windsor-Essex Mobility Group (WEMG). The most recent report is titled "The Windsor-Essex Parkway Project, Geotechnical Instrumentation Monitoring Progress Report, Phase 3 – Report 20 (Dec. 2011 – Mar. 2015)", dated April 2015. Based on our review of the data provided in this report, primary consolidation (the first stage of settlement which typically comprises 85 to 90 per cent of the total soil settlement) is nearly complete.

Piles at the west abutment should be installed no sooner than 12 months after completion of full embankment construction including fill between the end of the WDBA section and the new bridge abutment. In this case, in the absence of using lightweight fills, the estimated downdrag load is 625 kilonewtons per pile. A 35 per cent reduction in downdrag load is expected if cellular concrete backfill is used. Special staging requirements for piles installed at the east abutment are not necessary since primary consolidation is near complete. The estimated downdrag load, assuming the use of lightweight fills to replace approximately 10 per cent of the embankment volume within 20 metres of the east abutment area, is 56 kilonewtons.

The pavement surface of the proposed RSS "island" and west abutment approach will be 9 to 10 metres above the existing ground surface. Although pre-fabricated vertical drains (PVD's) can be installed prior to construction of the west approach embankment infilling (between the WDBA approach section and the new west abutment) and an RSS-enclosed "island", it will not be practical to preload or place surcharge fill on the RSS "island". To minimize the effects of differential post-construction settlement at the pile supported abutments and the approaches, infill for the west approach and RSS-enclosed "island" (if constructed) should consist wholly or partially of lightweight fills. Assuming that piles are driven prior to fill placement, the estimated downdrag load for each fill option is summarized in the following table:

Backfill Type	Estimated Downdrag Load (kN)
Conventional	880
Slag	800
Cellular concrete	565
EPS	410

The magnitude of the downdrag loads is a function of the size of the loaded area which includes relative downward movement of the soil mass around the piles and the amount of settlement remaining after the piles are installed. The depth of influence, or depth over which negative skin friction develops, dependent on both the size of loaded area and weight of applied load. If piles can be installed after the majority of settlement has occurred, downdrag loads will be reduced as indicated for the east approach embankment.



6.4 Pile Lateral Resistance

The lateral loading of the bridge structure could be resisted fully or partially by the use of battered driven piles for the case of conventional or semi-integral abutments. Vertical piles must provide the specified resistance to the lateral loading of the bridge structure if fully integral abutments are used. The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

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

d = pile width or diameter (m)

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

z = depth below ground surface grade (m)

s_u = shear strength (MPa)

The following table shows the calculated subgrade reaction dependent on the soil types encountered on site:

Soil Type	Elevation (m)		n_h	s_u
	From	To	(MPa/m)	(MPa)
Compact fill (sands)	Ground Surface	178	-	-
Compact sands and silts	178	177	3 - 5	-
Soft to hard clayey silt to silty clay	177	155	-	40 – 50

No values are provided for the sand fill since this material would have likely been removed during stripping and preparation of the embankment subgrades.

The MTO's Bridge Office expects the following deflections at the west and east abutments based on the current integral abutment design:

	Deflection (mm)	
	West Abutment	East Abutment
SLS (-)	-29.53	-43.53
SLS (+)	49.064	25.926
Total	78.592	69.454



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LPILE, version 2013, a software program produced by Ensoft for the analysis of piles under lateral loading using p-y curves was used to obtain lateral resistances for free-headed HP 310 x 110 integral abutment piles and conventional abutment piles. The lateral resistances and applicable deflection for each case are summarized in the following table below for HP 310 x 110 piles. If heavier HP 310 x 132 piles are used, the factored lateral resistance values for the Serviceability Limit State provided in the table below can be increased by a factor of about 1.4 for the same deflection values. In this case, however, the factored ULS values remain the same since the ultimate resistance is governed by the resistance of the soil at failure which is a function of the pile geometry and not the pile stiffness that otherwise influences the load-displacement behaviour.

Abutment/Pile Type	Deflection (mm)	Factored Lateral Resistance at ULS (kN)		Lateral Resistance at SLS (kN)	
		West Abutment	East Abutment	West Abutment	East Abutment
Integral Abutment HP 310 x 110 (Strong axis loading)	10	130	115	25	25
	25			55	55
	50			100	95
Integral Abutment HP 310 x 110 (Weak axis loading)	10	30	35	10	10
	25			25	25
	50			40	40
Conventional abutment HP 310 x 110 (Strong axis loading)	10	75	45	35	30
	25			60	45
	50			85	60

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

Pile Spacing in Direction of Loading D = Pile Diameter	Subgrade Reduction Factor, R ¹⁰
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The pile caps should be provided with a minimum of 1.0 metres of soil cover for frost protection.

¹⁰ Foundations and Earth Structure Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).



6.5 Liquefaction Potential

The site is located within the City of Windsor, Ontario. According to Table A.3.1.1 of the Canadian Highway Bridge Design Code (CHBDC), 2006, the zonal acceleration ratio, A , applicable to this site is 0.00. The corresponding acceleration-related seismic zone, Z_a , is 0.

Bridge B-1 is in Seismic performance Zone (SPZ) 1, based on a CHBDC classification as an “Emergency Route Bridge”. Based on the site stratigraphy, the soil profile type is categorized as Type III with a seismic site response coefficient, S , of 1.5 based on the CHBDC criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

6.5.1 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the Federal Highway Administration (FHWA) recommended procedures¹¹. The susceptibility of the fine-grained deposits were also assessed using the Bray et al. criteria described in the 4th edition of the Canadian Foundation Engineering Manual (CFEM). The liquefaction potential is considered to be low based on the soil profile type, age of the deposits, relative density, and the historically low regional seismicity. As such, a detailed evaluation of the liquefaction potential is not considered warranted.

6.6 Lateral Earth Pressures

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

- If conventional fill is used, select free-draining granular fill meeting the specifications of the Ontario Provincial Standard Specification (OPSS) Granular A or Granular B Type III should be used for backfill behind the abutments and walls. This conventional fill should be compacted in loose lifts no greater than 200 millimetres thick in accordance with OPSS.PROV 501.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment walls, in accordance with the Figure 6.6 of the CHBDC. Compaction equipment should be used in accordance with OPSS.PROV 501.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granulars. Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in

¹¹ FHWA, 1997: “Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles.” *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



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accordance with Ontario Provincial Standard Drawings (OPSDs) 3101.150, 3121.150 and 3190.100, as appropriate.

- The granular fill may be placed either in a zone with a width equal to at least 1.0 metres behind the back of the stem (Case A from Commentary on CHBDC Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear of the footing (Case B from Commentary on CHBDC Figure C6.20)
- For Case A, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM).

Soil unit weight: 20 kN/m³

Coefficients of lateral earth pressure:
At rest, K_o 0.50

- RSS 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 Case B for unrestrained walls, the pressures are based on the nature of the fills placed and the following parameters (unfactored) may be assumed:

Parameter	Granular A	Granular B (Type III)	Slag	EPS	Cellular Concrete
Unit Weight (kN/m ³)	22	21	13	0.3	6
Active Earth Pressure, K_a	0.27	0.31	0.27	N/A	0.25
Passive Earth Pressure, K_p	0.43	0.47	3.7	N/A	N/A

Lightweight fill may be used as backfill in the abutment areas. With the exception of blast furnace slag, neither EPS nor cellular concrete are frictional in nature. It should be noted that current design practice for retaining structures backfilled with cellular concrete is to assume that it behaves as a frictional material.

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

6.7 Lateral Resistance of Integral Abutment Backfill

If integral or semi-integral abutment design allows for movement of the bridge deck ends, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. The movement to allow passive pressures to develop within the backfill may be taken as:

- Rotation of approximately 0.1 about the base of a vertical wall;



- Rotation of approximately 0.02 about the top of a vertical wall;
- Horizontal translation of 0.05 times the height of the wall; or
- A combination of the above.

A resistance factor equal to 0.5 should be applied to the calculated total passive resistance (in accordance with Table 6.6.2.1 of the CHBDC). It should be noted that the passive pressures in front of the abutment wall (i.e. on the front embankment slope) should not be relied on in design given the downward sloping surface of the fill in this area, the potential for disturbance of the fill due to frost action and the potential for lack of compaction in this area.

6.8 Embankments

6.8.1 Embankment Stability

The global stability of approach embankments constructed with slopes no steeper than 3 horizontal to 1 vertical (3H:1V) and the island were assessed using SLOPE/W 2012, a limiting equilibrium slope stability software program produced by Geo-Slope International. Embankments constructed at this site are considered to be stable with a static factor of safety greater than 1.5 and dynamic factor of safety greater than 1.1 for long-term conditions.

In the short-term, the stability of the west approach embankment, currently under construction, and the island are highly dependent on the rate of fill placement. Due to the presence of the relatively soft clayey silt to silty clay deposit, it will be necessary to stage the embankment construction to permit dissipation of excess pore water pressures and related increases in shear strength between fill stages. Golder Associates developed a series of recommendations to be applied to construction of the west embankment based on the results of our geotechnical engineering analyses and review of the instrumentation reports for Phase 3 of the WEP construction. These recommendations were included in our design report for the WDBA's early grading contract for the DRIC CIP and formed the basis of the construction specification for the ongoing work:

- Side slope of embankment fill (non-reinforced) should be no steeper than 3H:1V;
- Fill should be placed to total heights of 5 metres above the average surrounding (undisturbed) pre-construction grade elevation without restriction on the placement rate;
- No additional fill should be placed for a minimum period of 30 calendar days after a maximum initial fill height of 5 metres;
- Where the fill and surcharge surface is to be greater than 5 metres above the average pre-construction grade elevations, the fill should be placed at a rate such that the vertical surface height is increased no faster than 1 metre per 50 calendar days; and
- Monitoring of settlement and pore water pressures in the underlying clayey deposits should be carried out during and after placement until substantial completion which is scheduled for April 2016. The monitoring should be analyzed to determine timing for placement of additional fill removal of the surcharge and paving of the roadway¹².

¹² Golder, June 2015. Geotechnical Design Report, Canadian Inspection Plaza and Perimeter Access Road Early Contract, Report No. 1527030-R01.



The above recommendations should also be generally applicable to construction of the island embankment option, if pursued as the final design option.

6.8.2 Settlement

Interpretation of Geotechnical Engineering Parameters for Clayey Silt to Silty Clay

A limited geotechnical investigation was carried out in the Bridge B-1 area to supplement extensive geotechnical studies carried out by Golder Associates for the adjacent DRIC project and previously constructed portions of the WEP. The stratigraphy and results of index and consolidation testing of samples obtained during the 2015 investigation were found to be consistent with the findings of previous work conducted with the study area. Therefore geotechnical engineering parameters interpreted for the native deposits for the early contract for the CIP were used in analyses for this project (Golder Associates Report No. 1527030).

Significant spatial variability with respect to sample depth and plan location was observed for the interpreted geotechnical engineering parameters. Therefore, correlations between laboratory, field and published data with review of the response to fill placement for the test embankment and east approach embankment were used to assess the geotechnical engineering parameters.

Undrained shear strength profiles of the clayey silt to silty clay were interpreted based on field vane shear tests and CPT data based on the following equation:

$$S_{U(CPT)} = \frac{q_c}{N_c}$$

Where:

$S_{U(CPT)}$	= undrained shear strength as derived from the CPT (kPa)
q_c	= tip resistance (kPa)
N_c	= cone factor

A correlation value of 1.0 was adopted for the results of conventional field vane shear tests carried out in the boreholes. This value was selected based on recommended correction values for materials with the range of plasticity index expected for this project^{13,14}. The interpreted undrained shear strength values were developed through comparison with conventional field vane tests and controlled rate of strain field vane shear tests from adjacent projects and site-specific profiles of the CPT data. In Golder's experience, conventional field vane shear tests in some soils can over-estimate the undrained shear strength on the order of 10 to 30 per cent since it is difficult to manually control the strain rate.

One-dimensional single and multi-stage consolidation tests, or oedometer tests, were carried out on relatively undisturbed samples of the clayey silt to silty clay from this site, the DRIC project, the WEP project and the US Inspection Plaza directly across the Detroit River. Relevant data from Golder Associates projects from the area along the west Windsor waterfront were also evaluated. The apparent pre-consolidation pressures obtained from

¹³ L. Bjerrum, 1972: *Embankments on Soft Ground*. Proceedings, ASCE, Speciality Conference on Performance of Earth and Earth-Supported Structures, Volume 2, 1-54.

¹⁴ L. Bjerrum, 1973: *Problems of Soil Mechanics and Construction on Soft Clays*. Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Volume 3, Moscow, 111-159.



the one-dimensional consolidation tests were compared with the interpreted preconsolidation pressures obtained from the CPT data using the following relationships:

$$S_{u(ref)} = 0.22\sigma'_p \text{ (Mesri, 1975)}$$

Where:

$S_{u(ref)}$ = reference undrained shear strength (kPa)

σ'_p = preconsolidation pressure (kPa)

The results of the CPT and oedometer comparisons indicated the clayey silt to silty clay deposit as overconsolidated with in a relatively thin crust, and nearly normally consolidated to lightly overconsolidated at depth.

The stress-strain properties of the clayey silt to silty clay were primarily established on the one-dimensional consolidation (oedometer) tests carried out on undisturbed samples from the plaza site and data from the WEP, US Plaza and Golder projects in the immediate vicinity. The following correlations were derived from the oedometer test results:

C_c = $0.012w_n - 0.102$, virgin compression index

w_n = natural water content (%)

C_r = $0.11 C_c$, recompression index

C_{α} , $C_{\alpha r}$ = $0.028 C_c$ and $0.028 C_r$, respectively, for the long-term secondary compression indices for all stress ranges

The oedometer test data was used to estimate the vertical coefficient of consolidation, C_v , and the horizontal coefficient of consolidation, C_h . The laboratory and in situ testing data were correlated to provide best estimates of the geotechnical engineering parameters throughout the full soil profile. A back-analysis of the nearby test embankment was completed to assist with defining and calibrating the engineering properties. This was done because actual field performance can differ from what is predicted using laboratory testing and correlations alone due to the limited number of boreholes and spatial variability within the clayey silt to silty clay deposit. A summary of the geotechnical parameters used for the settlement analyses is presented on Table 5 in Appendix H.

Performance of Existing Embankments – Test Embankment

The test embankment was approximately 150 metres in length along the RHHGP and 92 metres in width. Each of the three embankment lifts, or stages, were overbuilt with temporary side slopes extending to the design footprint at 2H:1V. The permanent side slope inclination will be 3H:1V. The final stage included a 1.0 metre surcharge placed to accelerate primary consolidations settlement and reduce long-term secondary (creep) compression of the underlying clayey soils. The fore slopes of the test embankment were designed to be constructed at approximately 5H:1V. Construction began in October 2011 and included stripping and grubbing, subgrade preparation, installation of prefabricated vertical drains (PVDs or 'wick' drains) along with a drainage blanket. The



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approximately 18 metre long wick drains were installed to a tip elevation of 161.0 metres. The PVDs were installed on a triangular layout, spaced at 1.5 metres centre-to-centre in the central zone and 2 metres in the approximately 15 metre wide outer zones to the north and south. The test embankment was instrumented with settlement plates, vibrating wire piezometers, a magnetic settlement gauge and a horizontal profiler between the period of August 2011 to September 2011 following construction of the drainage layer. Any settlement monitoring instrumentation that was not damaged was decommissioned in June 2014 when the surcharge was removed. A summary of the test embankment settlement is presented in the following table:

Stage	Period	Activity	Incremental/ Cumulative Consolidation Period (months) ¹	Peak Height (m) ²	Settlement (mm)		
					Centre	Crest	Slope
1	Mar – May 2012	Fill to El. 185.0 m	5/5	5.0	244	244	136
2	Oct - Nov 2012	Fill to El. 186.8 m	5/10	6.8	425	363	219
3	Apr – May 2013	Fill to El. 188.0 m (including 1 m surcharge)	6/16	8.0	570	477	285
	May 2013	Bridge B-2 piling completed	-/16		-	-	-
	Nov 2013	West abutment and RSS wall completed	-/22		-	-	-
	June 2014	Surcharge removed and granular roadbase placed	14/29		654	Damaged	Damaged
	December 2014	Paving completed ³	6/36		-	-	-

Notes: 1) The incremental consolidation period is taken as the elapsed time since completion of the filling stage prior to commencement of the subsequent stage of filling, or to the last settlement reading available. The cumulative duration of the consolidation period is based on the start of monitoring in December 2011. For example an incremental/consolidation period of 5/10 means that Stage 2 fill placement began 5 months after the May 2012 completion of Stage 1 filling and the total duration of consolidation at that time was 10 months.

2) The embankment height is taken as the surface elevation of the fill at stage completion minus an assumed undisturbed grade elevation of about 180 m based on the monitoring report text. The monitoring report indicates that the ground was stripped of topsoil, grubbed and grade prior to placing the drainage layer. The surface of the drainage layer was reported to be at elevations 179.7, 179.6, and 178.8 metres.

3) Results of post-paving settlement monitoring pending.

Performance of Existing Embankments – East Embankment

The east embankment was constructed concurrently with the test embankment discussed above. During the construction process, VWP's, magnetic settlement gauges, settlement plates, survey pins and an inclinometer were installed above the drainage layer to detect settlement magnitudes. Fill Stage 1 was completed between April and May 2012 to a fill elevation of approximately 185 metres, or 5 metres above the drainage layer. In July 2012, wick drains were installed along the east embankment and a second fill stage occurred between October and November 2012. Fill Stage 2 brought the embankment elevation up to 186.8 metres. Fill Stage 3, completed in May 2013, brought the fill and a 1 metre surcharge up to 188.0 metre elevation. The surcharge along the top



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of the embankment was removed in June 2014 and the placement of the granular road base material was completed. Asphalt paving was completed in December 2014. Any settlement monitoring instrumentation that was not damaged was decommissioned in June 2014 when the surcharge was removed. A summary of the test embankment settlement is presented in the following table:

Stage	Period	Activity	Incremental/Cumulative Consolidation Period (months) ¹	Peak Height (m) ²	Settlement (mm)
1	Apr - May 2012	Fill to El. 185	2/2	5	280
2	Sep - Oct 2012	Fill to El.186.8	6/7	6.8	680
3	Mar - Apr 2013	Fill to El. 188.0 (including 1 m surcharge)	5/13	9.0	950
	May 2013	Piling completed for Bridge B-2	-/14		-
	June 2014	Surcharge removed and granular base placed	13/27		970
	December 2014	Asphalt placed	6/31		-
	January 2015	Bridge B-2 completed	1/32		-

Notes: 1) The incremental consolidation period is taken as the elapsed time since completion of the filling stage prior to commencement of the subsequent stage of filling, or to the last settlement reading available. The cumulative duration of the consolidation period is based on the start of monitoring in March 2012. For example an incremental/consolidation period of 5/13 means that Stage 3 fill placement began 6 months after the May 2012 completion of Stage 2 filling and the total duration of consolidation at that time was 13 months.

2) The embankment height is taken as the surface elevation of the fill at stage completion minus an assumed undisturbed grade elevation of about 179 m based on the monitoring report text. The monitoring report indicates that the ground was stripped of topsoil, grubbed and grade prior to placing the drainage layer. The surface of the drainage layer was reported to be at elevations 178.7, 178.6, and 178.8 metres.

6.8.3 Settlement Analyses

Estimates of settlement performance of the proposed embankments and island were developed using Settle^{3D} Version 2.0, a three-dimensional settlement analysis software package developed by Rocscience. These settlement estimates were also calibrated to the existing embankment performance described above. Both embankment and the island were modelled assuming wick drains were installed to accelerate settlement. It should be noted that MTO's Embankment Settlement Criteria is in practice most commonly applied to high fill sections with heights of 4.5 metres or greater.¹⁵ This criteria was applied for the entire length of the approach embankments since the clayey deposit has a moderately high compressibility with a significant portion being normally consolidated or nearly so.

¹⁵ Ministry of Transportation Ontario, July 2, 2010. Embankment Settlement Criteria for Design.



West Approach Embankment

At the time this report was prepared the west approach embankment had recently been constructed under a contract to the WDBA. This embankment is approximately 600 metres long. The embankment height will decrease from a maximum height of 10.5 metres at the west abutment to 2 metres approximately 375 metres west. It eventually ends a further 200 metres west. The eastern toe of slope was terminated approximately 20 m west of the future eastern face of the Bridge B-1 west abutment. Further, the WDBA embankment was constructed with an eastern slope (facing the Bridge B-1 west abutment) of about 5 horizontal to 1 vertical, resulting in a significant approach embankment gap that must be filled to complete this bridge and its approaches.

The embankment was constructed with conventional earth fill after installation of wick drains and a surcharge was placed. The surcharge should remain in place for a minimum of 12 months, though the time duration between completion of surcharge placement and construction of the west abutment for Bridge B-1 is somewhat uncertain. It is understood that the project team is giving consideration to starting construction of Bridge B-1 from the east approach proceeding westwards. In this case, the west abutment will likely be completed around November 2017 approximately 18 months after the end of embankment construction. For the purposes of these analyses, it was assumed that installation of piles, construction of the west abutment and paving would be completed six months after removal of the surcharge.

The settlement analysis was carried out considering use of conventional fill only as presently planned, and scenarios where a portion of the conventional fill was replaced with lightweight fill immediately after removal of the surcharge material. Scenarios which included the incorporation of lightweight fill examined options which reduced the embankment weight by 20 to 50 per cent. For each scenario, the resulting post-construction settlement was compared to the MTO's criteria for maximum post-construction settlement for new freeway embankment built on compressible soils. The results of the analyses are presented in the table below with the west abutment being used as the transition point:

Distance from West Abutment	0	0 – 20 m	20 – 50 m	50 – 75 m	> 75 m
Maximum Embankment Height During Preload, including surcharge (m)	5	5	5 – 6.5	6.5 – 10	<10
Final Embankment Height (m)	10.5	10.3 – 10.5	10 – 10.3	9.5 – 10	<9.5
MTO Criteria (mm) (maximum post-construction settlement)	5	25	50	75	100
Estimated Total Settlement	800	775-800	725-775	725	<=725
Post-Construction Settlement	200	200	110 – 200	75 – 110	<=75
Post-Construction Settlement (20%) ¹	90	90	25 – 90	<75	<75
Post-Construction Settlement (30%) ¹	55	55	<55	<75	<75
Post-Construction Settlement (50%) ¹	<5	<25	<25	<75	<75

Notes: 1) This scenario assumes that following removal of the surcharge, the total embankment weight is reduced by 20, 30 or 50 per cent through replacement of the existing embankment fill with lightweight fill.

Settlement monitoring instrumentation has been included in the embankment works by the WDBA. The MTO has reported that the WDBA will provide the collected data to them upon the award of the Bridge B-1 project. The recommendations provided in this report are based on use of a settlement analytical model that has been



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calibrated against the actual performance of the test embankment and east approach embankment. However variations in the subsurface conditions, delays in WDBA's approach embankment completion schedule, if any, and the schedule and types of fill placement in the gap between the WDBA's approach embankment toe and the Bridge B-1 abutment will affect the settlement performance of the west embankment.

Following a series of design and cost optimization iterations, at the time this report was prepared, the gap between the WDBA approach embankment and the new abutment was planned to consist of the following components:

- installation of pre-fabricated vertical drains between the toe of the WDBA approach embankment and the future west abutment at a centre-to-centre triangular grid spacing of 1.5 m extending to 3 m beyond the future toes of the approach embankment slopes;
- construction of a 5 m high temporary RSS wall to create a surcharge at the location of the future west abutment and this RSS system is to be constructed using OPSS Granular B Type III;
- the orientation of the temporary RSS wall is to be perpendicular to the centre line of the highway and extend the full width of the approach embankment;
- the backfill zone, as measured from the face of the temporary RSS wall, is to extend 10 m toward the WDBA embankment at the ground surface and 20 m at the top elevation of the RSS wall;
- all other temporary fill between the temporary RSS wall and its granular backfill zone is to consist of OPSS select subgrade materials (SSM);
- following a preloading period and detailed evaluation of the monitoring results, the SSM is to be partially or fully removed and replaced with lightweight fill materials as required to control post-construction settlement to meet MTO tolerances; and
- the temporary RSS walls will be fully removed and replaced with RSS walls composed of conventional RSS wall components and cellular concrete backfill where the final RSS wall configuration will "wrap around" the abutment with the wing-walls parallel to the highway.

Schematics illustrating the proposed preload and required settlement monitoring instrumentation are presented on Figure 2. Settlement monitoring Non-Standard Special Provisions (NSSPs) should be included in the Contract Documents. Example Special Provisions for instrumentation (vibrating wire piezometer and settlement rods) have been provided in Appendix J. The vibrating wire piezometer installed in BH15-001 should be maintained during and after construction. It is outside the fill areas and provides an independent check of the groundwater level.

The analyses upon which the recommendations in this report are based indicate that if the west embankment construction remains on schedule and the surcharge is left in place for one year, a reduction of 30 per cent of the embankment weight would be required to achieve post-construction settlement that are generally within the MTO's criteria. However, at distance of up to 50 metres behind the abutment, padding of the asphalt pavement may be required for three years after completion of paving. The final determination for the magnitude of lightweight fill substitution and the timing of surcharge removal, piling works and other aspects of the staging must be carried out after review of the settlement data collected by the WDBA and the Foundations Engineering Specialist retained to monitor settlement of the preload. Assuming that the preload is constructed soon after project award, it will be in place for approximately six months prior to completion of the west abutment. In this case, the magnitude of post-construction settlement will be similar to the values shown in the preceding table assuming a reduction of the



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embankment/preload weight by 30 per cent. If the preload period is extended over the winter 2017/2018 season, the magnitude of post-construction settlement will be less and the volume of light-weight fill to be substituted could be reduced. The Contract should have an NSSP which requires review of this data by a qualified foundations engineering consultant and provision of appropriate recommendations at that time.

East Approach Embankment

At the time the east abutment is planned to be constructed, the approach fills would have been in place for approximately 5 years except in the gap between the as-built embankment fore-slope and future abutment. Based on design information at the time this report was prepared, the gap between the existing eastern approach embankment and future east abutment is to be preloaded with a temporary RSS wall system and OPSS Granular B Type III fill, similar to the planned work for the west abutment. Filling of this eastern gap will not include use of OPSS SSM fill in contrast to the west abutment since the gap length (in the highway centreline direction) is significantly smaller for the east abutment as compared to the west abutment. Settlement analyses carried out for this report considered use of conventional fill only, and scenarios where a portion of the conventional fill within 20 metres of the abutment is replaced with lightweight fill. Scenarios which included the incorporation of lightweight fill examined options which reduced the embankment weight by 5 per cent and 10 per cent. The following table presents the results of the settlement analyses as compared to the MTO's criteria for maximum post-construction settlement for new freeway embankment built on compressible soils. The east abutment was used as the transition point.

Distance from East Abutment	0	0 – 20 m	20 – 50 m	50 – 75 m	> 75 m
Maximum Embankment Height During Preload (m)	5	5.5	5.5 – 8.5	8.0 – 8.5	<8
Final Embankment Height (m)	9.5	9 – 9.5	8.5 – 9	8.0 – 8.5	<8
MTO Criteria (mm) (maximum post-construction settlement)	5	25	50	75	100
Post-Construction Settlement	200	35 – 200	35 – 55	50 – 55	<=50
Post-Construction Settlement (5%) ¹	40	25 - 40	40 – 55	50	<=50
Post-Construction Settlement (10%) ²	20	15 – 20	40 – 50	50	<=50

Notes: 1) This scenario assumes that following removal of the surcharge, the total embankment weight is reduced by 5 or 10 per cent through replacement of the existing embankment fill with lightweight fill.

It has been estimated that substitution of approximately 10 per cent of the existing embankment fill weight with lightweight fill may be necessary within 20 metres of the east abutment. Some padding may be required in an area within 20 metres behind the east abutment for three to five years after completion of the abutment.

As noted previously in the discussion with the west abutment, interpretation of the settlement monitoring data obtained during the preload period by a Foundations Engineering Specialist would be required to determine the timing for preload removal and the final magnitude of light-weight fill substitution. If due to schedule constraints, construction of the east abutment must proceed immediately after award, then cellular concrete must be used within the RSS backfill zone within 20 metres of the abutment. The remaining backfill is to comprise EPS.

These recommendations represent estimates given the absence of settlement monitoring data after June 2014. The March 2015 monitoring report notes that the deep settlement monitoring instrumentation was eliminated when



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the surcharge was removed in June 2014. A series of pavement survey pins (PSPs) were to be installed following completion of the paving in December 2014. However, the December 2015 monitoring report noted that installation of the PSPs was still outstanding.

The Island

At the time this report was prepared, the concept of including the centre “island” between the bridge spans, along with the associated RSS wall systems and abutments, was not going to be carried to final design. However, for completeness, this concept has been addressed in this report.

The island’s location between Ojibway Parkway and the ETR and design as an RSS structure effectively precludes preloading and surcharging to control/limit settlement. However, if conventional earth or lightweight fills are placed in the absence of PVDs, the time to complete primary consolidation (the first stage of settlement which typically comprises 85 to 90 per cent of the total settlement) will be greater than 60 months. If wick drains are installed, the duration will be reduced to 12 to 14 months from the end of fill placement. Although wick drains will be beneficial in reducing the time to complete primary settlement, partial or complete replacement of the conventional backfill with lightweight fill is recommended to reduce the magnitude of settlement and impacts on the adjacent railway and buried utilities. In the absence of lightweight fill, the total settlement will exceed the MTO’s criteria. The maximum anticipated total settlement of 640 millimetres will occur in the middle of the loaded area. The magnitude of total settlement around the perimeter of the island will range from 30 to 50 per cent of this value. The following table summarizes the anticipated total settlement by fill type and location:

Fill Type	Maximum Settlement (mm)	Abutment Settlement (mm)	
		Abutment #2	Abutment #3
Conventional Fill	640	300	400
Slag Fill	435	175	265
Cellular Concrete	140	60	95
EPS	65	35	65

If EPS fill is utilized, a condition of net-zero loading and negligible settlement can be achieved if the soil within the island footprint is excavated to a depth where the weight of the excavated soil corresponds to the weight of the new structure. Excavations in the range of 1.5 to 2.0 metres would be required to achieve this goal. An RSS structure cannot be built using EPS fill. The walls of the island will have to be structurally tied together with this option. Depending on the chemistry of the selected slag fill, consideration will also have to be given to the potential for corrosion of metallic strip reinforcements, if used, and protection against moisture uptake which leads to increases in unit weight. Use of lightweight fills is feasible for this site from a geotechnical perspective.

To be effective, wick drains must be installed within and outside of the island footprint. It should be noted that it will be necessary to modify the location and extent of the wick drain installation west of Abutment #2 and east of Abutment #3 due to the presence of existing Hydro One ductwork and a sanitary forcemain in those areas. Golder understands that these utilities will not be relocated.



Compared to the approach embankments, the range of settlement mitigation measures that can be applied at the island is limited. The design option containing the island is complex given the requirement for staged construction of the RSS structure, increased costs due to use of lightweight fill as the primary method of settlement mitigation, and risk posed to the adjacent railway and utilities arising from fill-induced post-construction settlement. As such in November 2015, the design team (including the MTO) carried out cost-benefit and risk analyses to decide if the RSS structure flanked by pile-supported bridge spans should be abandoned in favour of a single pile-supported structure. The pile-supported bridge span option was unanimously selected as the preferred technical alternative during a November 27, 2015 teleconference with the MTO and the design team.

6.9 RSS Walls

The abutments will each feature RSS walls with lightweight backfill. The RSS retaining walls are to be designed in accordance with the MTO RSS Guidelines (2008) and Special Provision (SP) 599S22. The design of the RSS walls must incorporate subsurface and surface drainage elements.

RSS walls may be designed such that the facing blocks are built on a levelling pad constructed with Granular A to a minimum thickness of 300 millimetres. Depending on the design selected by the RSS supplier, it may not be necessary to provide 1.0 metres of earth cover or thermal equivalent for frost protection. However, the foundations must have adequate embedment to provide a stable structure. Typically, the embedment depth, defined as the distance between the top of the levelling pad and the top of the adjoining finished grade, is a minimum of 1 metre. The RSS designer should confirm the internal stability of the structure. The embankments and island structure were found to be externally stable with a long-term factor of safety greater than 1.3, provided that construction staging, fill placement schedule and use of lightweight fills are designed and constructed in accordance with the recommendations in this report. This long-term factor of safety is not applicable to RSS systems constructed using conventional backfill materials.

The native soils will not provide sufficient bearing resistance to adequately support the RSS panels. The RSS panels may be constructed on an engineered fill pad consisting of a minimum thickness of 600 millimetres of compacted OPSS Granular A that extends at least 600 mm from the edges of the facing support blocks in all directions. A factored geotechnical resistance at ULS of 225 kilopascals and a geotechnical reaction at SLS of 150 kilopascals may be used for design of foundations founded on a granular pad with the minimum dimensions as described above.

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).

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 'obstruction', splaying, or full or partial omission of RSS wall reinforcement in the area of the obstruction. The adjacent reinforcement must be designed in such a way to accommodate the additional loading resulting from removal of reinforcing elements in the area of the infrastructure.



Drainage Requirements for Cellular Concrete Backfill

If cellular concrete is used as backfill, the upper surface must have a minimum slope of 2.5 per cent toward the rear limit of the backfill materials and a vertical drainage system. Based on past experience, achieving this slope during cellular concrete placement and finishing stage is not reliably successful since cellular concrete is self-levelling. The slope should be provided by a normal cast-in-place concrete cap by cutting and trimming once the cellular concrete achieves initial set or suitable alternative. A granular drainage blanket connected to a positive outlet must be provided behind and beneath the reinforced section.

6.10 Overhead Sign No. 1 at Station 10+033 Rt

A new overhead sign is planned to be constructed near Station 10+033 Rt. Typically, overhead signs are supported by drilled shaft (caisson) foundations that penetrate some 5 to 10 m below the local ground surface, depending on soil conditions. The planned location places the foundations on, within or through the lightweight fill and, potentially, the RSS wall reinforced backfill zone. For this project the planned sign location is a disadvantage for foundation design since none of the lightweight fill materials will provide suitable lateral resistance, which is likely to be the most important mode of loading from wind forces. Further, the embankment area in the immediate vicinity of the sign foundations may be subject to settlement, depending on the timing and type of lightweight fill placement as compared to sign foundation installation. For these reasons, a non-standard foundation design will be required.

If a deep foundation is necessary, it is preferable from a geotechnical perspective, to support the overhead signs using a driven pile foundation where at least two piles are used for each sign end. Piles for each sign end are to be separated in the east-west direction by an appropriate distance to minimize direct lateral loads on the piles and resistance to moments from wind loads on the sign is developed through uplift and downward force on the piles. If possible, battered piles would be best for these sign foundations (battered downward and away from the sign in the east-west directions). Because of the need for lightweight fill and the problems that could occur from drilling or driving piles through these materials, it will be necessary to install a corrugated metal sleeve, akin to the pipes used for integral abutment construction, through the lightweight fill materials prior to or as they are placed to allow for later pile foundation construction. Geotechnical vertical resistance values for driven steel H piles are provided in Section 6.2.1 of this report; however, lateral resistance values must be reviewed in detail once the final geometry and fill materials are determined. For design purposes, lateral resistance within lightweight fill materials should not be relied on.

As an alternative to use of deep foundations, if the sign structure can tolerate long-term differential settlement on the order of 25 to 50 mm in the north-south direction (median to shoulder), it may be appropriate to use an oversized spread foundation for sign support. Spread foundation dimensions will likely be governed by overturning and uplift forces. For foundation design purposes, factored geotechnical resistance values at ULS and SLS of 150 kPa and 100 kPa, respectively, may be used, provided that the final geometry and loading conditions for the underlying lightweight fill materials are reviewed for the final design configuration. If the foundation loads might influence design of any RSS walls or could influence embankment stability, the final geometry and loading conditions should be reviewed in detail. In general, if these foundations are to bear on an area that will be composed of polystyrene fill, it will be essential that the polystyrene blocks are separated from the foundations by approximately 1 m of compacted granular fill. The foundation design dimensions should be selected that maximum edge pressures associated with overturning moments are less than the resistance values provided above. Further, the foundation dimensions should be selected such that the force resultant remains within the centre third of the



foundation unit. For lateral sliding resistance considerations, an unfactored friction factor ($\tan \delta$) of 0.45 may be used for concrete cast directly on engineered granular fill or 0.3 for concrete cast on compacted silty clay (existing east approach embankment materials). Uplift resistance should be based on the total deadweight of the foundations and any engineered backfill materials placed directly on the foundations. Given the relatively shallow embedment depth of spread foundations, shear resistance within the overlying and adjacent backfill materials should be ignored. The factored ULS uplift resistance may be taken as 0.5 times the total deadweight of backfill and foundation materials and the SLS factor for uplift resistance may be taken as 0.3.

6.11 Additional Design Considerations

6.11.1 Corrosion Assessment and Protection

Soil corrosivity may affect the pile installations, and concrete pile caps. The long-term performance and durability of the structures are directly related to their respective corrosion resistance. Generally, the corrosivity of a structure depends on the soil resistivity, hydrogen ion concentration, salts (chloride and sulphate) concentrations and redox potential.

An assessment of the corrosion potential for the steel piles was carried out by comparing the results of the analytical testing for corrosion related parameters to the California Department of Transportation (CalTrans) corrosion guidelines.¹⁶ This policy indicates that for representative soil samples, if the soil has a minimum resistivity less than 1,000 ohm-cm then they should be tested for chlorides and sulphates. Low resistivity values are indicative of increased high concentrations of soluble salts and are susceptible to increased attacks on the protective oxide film on steel piles. Also the potential for macrocell corrosion, where metal oxidation is prevalent at one site and reduction is predominant at another site is increased in low resistivity soils.¹⁷ A site is considered to be corrosive if one or more of the following three conditions are met:

- Chloride concentration is 500 parts per million (ppm) or greater;
- Sulphate concentration is 2,000 ppm or greater; or
- pH is 5.5 or less.

A single sample of native silty clay near elevation 174 metres was submitted for analytical testing. Although the measured resistivity was 840 ohm-cm and less than 1,000 ohm-cm, the piles will be installed in an environment that is generally considered to be not corrosive. Therefore no provision for metal loss/corrosion allowance is necessary. The results of the analytical testing are compared to the guidelines in the following table:

Parameter	Condition	Analytical Result
Chloride (ppm or ug/g)	Greater than or equal to 500 ppm	81
Sulphate (ppm or ug/g)	Greater than or equal to 2,000 ppm	1,100
pH	Less than or equal to 5.5	7.75

¹⁶ California Department of Transportation, 2012: Corrosion Guidelines, Version 2.0, California Department of Transportation, Division of Engineering Services, Materials Engineering Testing Services, Corrosion and Structural Concrete, Field Investigation Branch.

¹⁷ Transportation Research Board, 1998: National Cooperative Highway Research Program (NCHRP) Report 408: Corrosion of Steel Piling in Nonmarine Applications, National Academy Press, Washington, D.C.



No testing was carried out on the fills or native granular soils overlying the cohesive deposits. Oxygen is most available in the upper portion of the pile, within fills or soils disturbed by excavation and near the surface of the groundwater which has been inferred at elevation 177.5 metres. Although the overall pile environment is considered to be non-corrosive, there is an increased probability of pitting corrosion between the bottom of the pile cap to approximately 1 metre below the groundwater level.

The soil pH and water soluble sulphate content were examined to evaluate the subsoil conditions for possible sulphate attack on concrete. According to Tables 2 and 3 of CSA Standard, CAN/CSA-A23.1-14, the degree of exposure to sulphate attack is moderate. Therefore Type S-3 cement can be used in the subsurface concrete. The minimum specified 56-day compressive strength should be 30 megapascals and the maximum water/cementing ratio should be 0.50.

According to the MTO Gravity Pipe Design Guidelines, the soil corrosiveness is moderate to severe based on its resistivity magnitudes, sulphate content and redox potential. The corrosivity will affect both the unprotected driven piles and the concrete, therefore non-corrosive concrete mixtures should be considered along with proper lining of the piles. Specific design details regarding protection for the piles and concrete should be carried out by a specialist design firm.

6.11.2 Lightweight Fills

Lightweight fill will be used on this project due to the presence of deep and compressible subsurface deposits where long-term post-construction creep settlements will affect the performance of the highway and where there is no available time in the construction schedule for a sufficient preload or surcharge period. When using lightweight fill, the following factors should be taken into consideration:

- Provision of soil cover: embankments construction with polystyrene fills should be constructed with 2 horizontal to 1 vertical or flatter side slopes given the need for granular fill for a levelling pad and conventional soil cover on side slopes for protection from chemical spills, fire and ultraviolet light;
- Cost premium: significant costs can accrue depending on the type and volume of lightweight fill required; and
- Buoyancy forces: it is not appropriate to install lightweight fill below surface or groundwater levels unless temporary dewatering is undertaken and the final vertical loads are sufficient to overcome buoyancy forces.

The overall goal of using lightweight fill for this bridge site is to minimize the new embankment net loads on the underlying soil to the degree practicable. Three lightweight fill materials are available to achieve this purpose, listed in order of increasing unit weight:

- Expanded Polystyrene (EPS): EPS is formed in blocks typically measuring about 1.2 by 0.6 by 0.2 metres ranging up to 2.0 by 0.75 by 0.75 metres with unit weights ranging from about 0.1 to 0.4 kilonewtons per cubic metre (kN/m^3), though EPS meeting the minimum compressive strength criteria for roadway applications is typically about 0.2 kN/m^3 ;
- Cellular Concrete: Cellular concrete is a product of cement, water, a foaming agent, and air placed by injecting air and foaming agent into a cement-water slurry to produce a cured concrete-like material with unit weights typically on the order of 4 to 8 kN/m^3 and unconfined compressive strengths of 0.5 MPa or greater; and



- Blast Furnace Slag: Granular, water-cooled blast furnace slag can be used as a lightweight fill and, for MTO applications, typically exhibits unit weight values ranging from less than 12.5 kN/m^3 (“ultra-lightweight blast furnace slag”) to about 14.5 kN/m^3 or less. Blast furnace slag is susceptible to crushing if over compacted. If this material is used in the final design, an NSSP for lightweight fill material which discusses methods of constructing embankments with blast furnace slag and means of preventing overcrushing.

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

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

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

6.12 Vibration Considerations

Energy produced by driving piles into the earth travels through the ground from the pile to structures within and on top of the ground by seismic, or stress waves. The amplitude of this energy depends on many factors, including pile depth, ground hardness, ground uniformity and energy delivered to the pile. Vibrations in the form of shear waves are generated from friction along the pile shaft and shear and compression waves arise from compression and displacement of soils at the pile tip. Due to the proximity of the Bridge Site B-1 to underground utilities, railway tracks and other structures, vibrations caused by pile driving should be accounted for in design, specification development, construction planning and construction monitoring.

Multiple utilities are located within the Bridge B-1 corridor including several gas mains, a sanitary forcemain and a watermain. A detailed assessment of the impact of pile driving on these utilities will be carried and the findings reported in the final report. This section of the report specifically addresses vibrations arising from pile driving activities (foundation piles and sheet piles). Vibrations may arise from other construction equipment, traffic from Ojibway Parkway and the ETR rail line and other sources. However, these sources are cumulatively considered to be contributors that form existing “background” conditions and are outside the scope of this report. The remainder of this section provides comments and recommendations regarding ground vibrations arising from pile driving activities. These comments are based on a vibration assessment carried out for this project. The results of this assessment were summarized in Golder Associates Technical Memorandum 13-1132-0053-1002-M01 which is attached to this report in Appendix K.



6.12.1 Anticipated Levels of Vibration

Peak particle velocity (PPV) is a key measure of the amplitude of ground vibrations. Typically, humans perceive vibrations at very low PPV values on the order of 0.2 to 2 mm per second, depending on frequency. Vibrations are commonly considered “disturbing” or “strongly perceptible” in the range of 10 to 20 millimetres per second (mm/sec) for steady state vibrations (e.g. vibratory pile drivers or compaction equipment) and on the order of 20 to 50 mm/sec for transient vibrations (e.g. impact pile drivers).¹⁸ In general, structures and buried utilities are more tolerant of vibrations before damage occurs as compared to human perceptions of severity. Buried utilities are less susceptible to vibration effects compared to above ground structures (Harrison 2009) and restrained in-ground resonance is unlikely. Reported PPV limits for pressurized gas transmission pipelines and for fibre-optic transmission lines range from about 100 to more than 125 mm/sec.¹⁹ In addition to damage resulting directly from vibration amplitudes, structures and utilities can be damaged by vibration-induced densification of loose granular soils and consequent ground settlement.

Commonly, limits on PPV and frequency arising from pile driving activities are recommended or imposed by structure or utility owners that might be affected by nearby construction. Typical limits for various types of infrastructure range from about 10 to 240 mm/sec, depending on the type, age and location of the facility. Common limits for buried utilities of good condition are 25 to 50 mm/sec. A summary of typical vibration limits is presented in Table I in Appendix K.

Vibrations emanating from pile driving operations are expected to be most pronounced when the pile tip is passing the invert level of the utility, driving through a zone with obstructions such as cobbles and boulders and is being driven to refusal on the underlying bedrock. A critical utility is one that is partially or fully located within loose granular layers that are susceptible to settlement due to pile driving, is constructed with joints, or has been identified as being old and in potentially in poor condition such as the watermain and sanitary forcemain.

The level of vibrations generated depends on the pile type, hammer type and operational frequencies. Non-displacement piles, such as the preferred foundation type H-piles, generate lower vibration amplitudes compared to displacement piles such as tube piles. Non-impact pile drivers that utilize mechanical hydraulic systems to slowly force the pile into the ground can be used for installation, but these may not be available or cost effective for this project. Consideration could also be given to resonance-free vibratory pile drivers which do not vibrate during pile start up and shut down which are usually the stages where excessive vibrations occur. Past project experience with common vibratory and impact pile driving equipment and similar conditions indicates that at the location of the piles (sheet piles or driven H piles), the peak particle velocity could be in the range of 50 to 150 mm/sec. For similar ground conditions, the peak particle velocity should diminish in a logarithmic pattern such that at a distance of 2 metres, the PPV is on the order of having the maximum value, reducing to about 10 per cent of the maximum value at a distance of about 10 metres from the source.

A ground vibration prediction model was developed for this site. This model was used to examine and predict the vibration levels resulting from use of impact and vibratory hammers on this site. The estimated PPV, for various stand-off distances were higher for an impact hammer than for a vibratory hammer. Therefore, estimates of maximum ground vibrations at different separation distances from the piling operations were based on the impact

¹⁸ CalTrans, 2004: Transportation- and Construction-Induced Vibration Guidance Manual, California Department of Transportation Environmental Program, Environmental Engineering, Noise, Vibration, and Hazardous Waste Management Office.

¹⁹ Harrison, D, 2009: Monitoring Pile Driving Vibrations – Problem Avoidance and Case Studies. 2009 International Foundation Congress and Equipment Expo Proceedings. Contemporary Topics in Deep Foundations. American Society of Civil Engineers.



hammer. A summary of the anticipated PPV at each utility identified for this project is presented on Table 3 of Appendix K.

Only one utility, the gas line situated approximately 3.2 metres west of Pier #1 is estimated to be above the proposed 50 mm/sec limit. The actual ground vibration limits should be confirmed through on site monitoring during the pile driving operations. If the ground vibrations recorded during monitoring exceed the review level of the vibration control limit of 80%, the pile driving operator shall take steps to reduce the PPV level. If these steps are unsuccessful, or the Contractor exceeds 100% of the ground vibration control limit, piling shall cease. The Contractor should submit a plan to reduce the vibrations for approval by the Contract Administrator.

The analyses and model used to predict the anticipated vibration levels were based on literature derived site constants for empirical formulae. Therefore, vibration monitoring should be carried out as described in the NSSP titled 'Pile Driving Vibrations' in Appendix J. This specification also details pile driving requirements and restrictions, the method of particle velocity control, applicable limiting particle velocities and vibration monitoring requirements. These and other aspects of vibration control are briefly addressed in the following discussion.

6.12.2 Recommended Vibration Mitigation Measures

Vibration mitigation measures involve three categories:

- Modification of equipment or installation procedures to reduce vibrations at the source;
- Monitoring of vibration when pile driving is occurring within 5.0 metres of a utility;
- Installation of a wave barrier between the vibration source and structure to be protected; and/or
- Pre-drill and install temporary casings through the native granular soils and into the native clay soils to below the utility invert.

Wave Barriers

Wave barriers can consist of a trench or thin wall like a trench box or sheet piles which reflect or absorb wave energy and decrease propagation between the source and the receiver. To be effective, the wave barrier must be sufficiently deep and long. Since vibrations at the receiving utilities are expected to be significant when the pile tip is passing by the utility invert elevation, the wave barrier must extend to 1 to 1.5 metres below this invert elevation. Assuming a typical wave frequency of 17 Hertz, and an average wave velocity of 100 mm/sec, it should also extend a minimum of 6 metres beyond the longitudinal extent of each pile group. In this case, any sheet piles installed for control of groundwater in the native granular soils might suffice for both purposes.

Modifications of Pile Driving Equipment and Procedures

The energy delivered to each pile can be reduced by selection of a smaller sized hammer or shorter hammer stroke. This will be important when driving through zones with expected cobbles and boulders such as approaching the bedrock surface. When driving near the bedrock surface or through hard zones, monitoring ground vibrations at the utilities will be critical to evaluate propagation and attenuation of vibration energy from the tip to the elevation of the utilities. Monitoring at pre-selected distances from the pile and adjusting the frequency of driving, if vibratory



hammers are used, will assist in adjusting pile driving operational parameters to minimize the energy and consequent ground displacements transmitted to the utilities.

6.12.3 Addressing Vibration Concerns

An NSSP should be prepared specifically related to pile driving vibrations. Vibration concerns should be addressed in the contract documents and specifications as follows:

- 1) As a minimum, carry out a pre-construction and post-construction survey of all buried utilities within 25 metres of each pile group as recommended by the current CHBDC. Consideration could also be given to surveying other structures situated within 120 metres of the pile groups. Surveys must be completed by experienced personnel and carefully documented with sketches and photographs. Utility inspections will likely require vacuum excavation and exposure of the utility to better identify the as-built location and examine pipe materials and conditions. If possible, vacuum excavation should be planned to expose joint areas as well as areas between joints.
- 2) Inform utility owners and other stakeholders about the potential construction related consequences of pile driving including those arising from vibrations. This should include a careful description of human perceptions versus vibration levels actually required to cause damage. This will help manage expectations and reduce claims.
- 3) Implement a procedure to collect, investigate and deal with vibration related complaints.
- 4) Incorporate vibration reduction measures as described in Section 6.12.2
- 5) Monitor and record vibration levels in real-time during construction as noted in the CHBDC. Collection of this data will assist in evaluation of claims and will augment MTO's database on pile driving related vibrations. Review and alert levels at the utility or structure of concern should be set at 80% and 100% of the PPV, respectively. If either the review or alert level is achieved, the Contractor should cease operations and submit a report detailing corrective actions. These actions must be approved by the Contract Administrator prior to the resumption of piling.

6.13 Construction Considerations

6.13.1 Excavations and Groundwater Control

Excavation and backfilling operations for the proposed RSS walls, driven piles and embankment construction 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 along with any cohesionless materials encountered below the groundwater level at this site would be classified as Type 3 or 4 soils. The native clayey silt to silty clay 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. It may be necessary to blanket the side slopes with coarse granular material to maintain slope stability.



Excavations for the removal of the uncontrolled fill and for retaining wall footings, or levelling pads, will extend through the surficial topsoil and fills and terminate in the native soft to firm silty clay. These excavations may extend below the inferred groundwater level of approximately 177.5 metres. Seepage should be expected from the saturated sand layers within the existing fill. Due to the fines content of the granular fill and native soils, it may be difficult to effectively dewater them using gravity methods. Use of vacuum well points and the like may be necessary. Seepage near the boundary between the fill or native granular soils and the underlying silty clay will cause the fill and granular soils to rapidly progress from fast-ravelling to flowing behaviour and can lead to large ground losses. Even with installation of closely spaced sump pits, pumps and other active dewatering systems, difficulties in controlling water and ground behaviour near this interface should be expected. In order to reduce the dewatering effort, consideration could be given to installing relatively short driven steel sheet piles terminating in the lower permeability silty clay to clayey silt deposit. In order to limit dewatering requirements and improve material handling and movability in this area, it is suggested that work excavations be carried out during dry periods to the extent practical. Surface water runoff should be directed away from open excavations at all times. The Contract documents should include a NSSP alerting the Contractor to the need for effective control on groundwater.

6.13.2 Driven Piles

It should be noted that the presence of cobbles and boulders in the soils, as well as the potential for debris within the existing fill materials, may be encountered and affect pile driving/drilling operations. An NSSP should be included in the Contract documents warning about the potential for encountering obstructions and cobbles and boulders including between about elevation 161.5 and 160.5 metres at the east abutment. Elevated levels of hydrogen sulphide gas were encountered in the artesian aquifer located at the bedrock/soil interface. The NSSP should also warn about the presence of hydrogen sulphide and methane gas during pile installation. Temporary liners may be required during any pre-drilling for pile driving operations due to the presence of saturated granular fill materials. Pile driving controls to mitigate against settlement and damages relating to excessive vibrations are addressed in Section 6.12.

Due to the presence of artesian conditions at or near the bedrock surface, the possibility of groundwater migration around the pile annulus exists. To minimize the creation of a potential void around the pile as it is driven, driving shoes, reinforcement to flanges, splice plates and the like should add as little as possible to the pile cross-sectional dimensions. Further, it is recommended that the piles be driven from a 0.5 metres thick blanket of Granular A which would act like a filter should sustained groundwater flow occur. The blanket should be constructed in accordance with the Drainage Blanket Detail sheet provided in Appendix I. Consideration should be given to use of the drainage blanket installed for the wick drains for this purpose.

6.13.3 Railway Track Protection

At the time of this report, the ETR has not provided any agency-specific requirements for excavations adjacent to their railway tracks. As a result, guidelines used by the Canadian National Railway (CNR) and the American Railway Engineering and Maintenance of Way Association (AREMA) for track protection have been applied for this project.



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Based on the CNR guidelines, track protection is typically not required for excavations up to 3.7 metres in depth with slopes offset a minimum distance of 0.5 metres from the edge of tie and inclined at a maximum slope of 1.5H:1V. This applies to cut slopes where the groundwater level is below the base of the excavation, or the soils are properly dewatered. Excavations that do not meet this requirement must include a temporary protection system (track protection) designed in accordance with AREMA guidelines. The track protection system should be designed to the performance level 1b of OPSS.PROV 539. At this site, the stability of any temporary excavations more than 2 metres in depth should be evaluated for stability because of the presence of the soft silty clay to clayey silt soils.

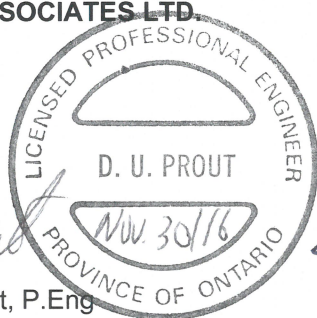
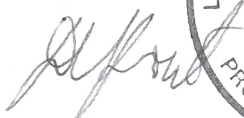


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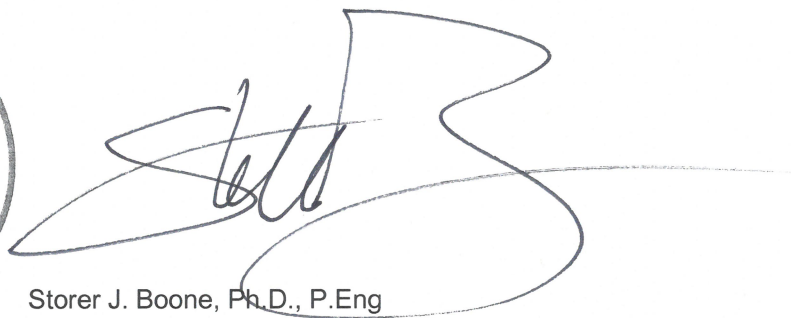
7.0 MISCELLANEOUS

This report was prepared by Mr. William Hanson, E.I.T. and the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by the Team Leader Dr. Storer J. Boone, P.Eng., a senior geotechnical engineer and Principal with Golder. An independent quality review of this report was carried out by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.


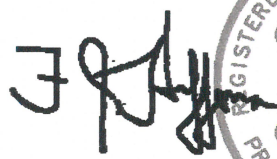
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TABLE I

COMPARISON OF STRUCTURE ALTERNATIVES

Proposed RHHGP Ojibway Parkway/ETR Overpass
 Bridge B-1
GWP 3028-14-00

STRUCTURE OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Single (Three or Four-Span) Structure	<ul style="list-style-type: none"> Feasible (Three-Span is preferred technical alternative). 	<ul style="list-style-type: none"> Settlement mitigation required at abutments only Dewatering requirements limited to footprints of pile caps Alternative with least construction time Downdrag issues eliminated for piers between the ETR and Ojibway Parkway Least impact on adjacent buried utilities, ETR and roadways Uniform settlement performance between spans Option with most straightforward construction Settlement monitoring not required Lowest risk option 	<ul style="list-style-type: none"> Depending on selected design, potential for higher long-term maintenance costs 	<ul style="list-style-type: none"> 1.0 	<ul style="list-style-type: none"> Low risk In absence of proper temporary shoring and groundwater control, ETR and adjacent utilities may be impacted by pile cap excavations

COMPARISON OF STRUCTURE ALTERNATIVES

STRUCTURE OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Two structures with 'Island' constructed with RSS walls and cellular concrete fill	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Island alternative with light-weight fill and reduced construction duration Island alternative which provides second greatest reduction of settlement and downdrag loads 	<ul style="list-style-type: none"> Settlement mitigation required at abutments and Island Although wick drains can be used to accelerate time to complete primary consolidation, alternative/complementary measures such as preloading or surcharging will be difficult to implement due to schedule and site configuration Dewatering of surficial granular deposits may be required in order to construct RSS wall foundations or to off-set fill loads Island alternative with low unit weight and relatively short construction duration; will likely require fewer stages to reach full height compared to slag fill and conventional fill options RSS walls backfilled with cellular concrete are prone to durability concerns in the absence of proper drainage Downdrag loads must be considered in design particularly for conventional or semi-integral abutments as piles must be driven before filling; magnitude will be intermediate between EPS and slag fill alternatives Impact due to fill-induced settlement on adjacent buried utilities, ETR and roadways will be intermediate to slag fill and EPS fill options Two-stage RSS wall may not be required, but is recommended Settlement monitoring required Increased engineering time and costs should problems arise during fill placement 	<ul style="list-style-type: none"> 3.5 	<ul style="list-style-type: none"> Moderate risk ETR and buried utilities within 15 m of Island subject to 10 to 50 mm (total) of fill-induced settlement and damage Low to moderate risk of post- construction differential settlements/ongoing padding of Island approaches Drainage and deterioration due to freeze/thaw and water infiltration if RSS internal drainage is not adequately controlled Improper setting of cellular concrete/water accumulates in voids and loss of bond or friction resisting with reinforcing elements

COMPARISON OF STRUCTURE ALTERNATIVES

STRUCTURE OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Two structures with 'Island' constructed with RSS walls with conventional fill	<ul style="list-style-type: none"> Marginally feasible 	<ul style="list-style-type: none"> Island option with most economic fill alternative Compared to EPS, slag fill and cellular concrete fill, special handling or additional design details are not required 	<ul style="list-style-type: none"> Settlement mitigation required at abutments and Island Although wick drains can be used to accelerate time to complete primary consolidation, alternative/complementary measures such as preloading or surcharging will be difficult to implement due to schedule and site configuration Dewatering of surficial granular deposits may be required in order to construct RSS wall foundations or to off-set fill loads Island alternative with highest unit weight fill and longest construction time since fill must be placed in stages to permit dissipation of excess pore pressure and strength gain of underlying foundation soils Downdrag loads must be considered in design particularly for conventional or semi-integral abutments as piles must be driven before filling Option with greatest impact on adjacent buried utilities, ETR and roadways due to fill-induced settlement Highest potential for differential settlement between abutments and island approach slabs Settlement monitoring required Increased engineering time and costs should problems arise during fill placement Two-stage RSS wall likely necessary 	<ul style="list-style-type: none"> 2.0 	<ul style="list-style-type: none"> Highest risk option ETR and buried utilities within 15 m of Island subject to 5 to 200 mm (total) of fill-induced settlement and damage High risk of post-construction differential settlements /ongoing padding of Island approaches

COMPARISON OF STRUCTURE ALTERNATIVES

STRUCTURE OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Two structures with 'Island' constructed with RSS walls with blast furnace slag fill	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Most economical lightweight fill alternative 	<ul style="list-style-type: none"> Settlement mitigation required at abutments and Island Although wick drains can be used to accelerate time to complete primary consolidation, alternative/complementary measures such as preloading or surcharging will be difficult to implement due to schedule and site configuration Dewatering of surficial granular deposits may be required in order to construct RSS wall foundations or to off-set fill loads Island alternative with lower unit weight; may require fewer stages to reach full height compared to conventional fill option Blast furnace slag is susceptible to crushing if overcompacted Slag chemistry may adversely impact corrosion rates of metallic reinforcement strips Protection against excessive moisture uptake and associated gain in slag unit weight required Downdrag loads must be considered in design particularly for conventional or semi-integral abutments as piles must be driven before filling; magnitude will be intermediate between conventional fill and cellular concrete fill alternatives Possible two-stage RSS wall Settlement monitoring required Increased engineering time and costs should problems arise at placement 	<ul style="list-style-type: none"> 2.5 	<ul style="list-style-type: none"> Moderate to high risk ETR and buried utilities within 15 m of Island subject to 15 to 65 mm (total) of fill-induced settlement and damage Moderate risk of post-construction differential settlements, ongoing padding of Island approaches Crushing of slag fill during compaction/increase in unit weight Excessive moisture uptake/in absence of moisture barriers and effective drainage will result in increase in unit weight Corrosion can occur if appropriate corrosion protection not provided for metallic reinforcing strips

COMPARISON OF STRUCTURE ALTERNATIVES

STRUCTURE OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
Two structures with 'Island' constructed with EPS fill with pre-cast concrete facing panels	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Island alternative with least unit weight fill and shortest construction duration; it may be possible to build embankment to full height in a single stage Island alternative which eliminates or provides greatest reduction of settlement and downdrag loads With net zero loading approach, wick drains not required Island alternative with least impact on adjacent buried utilities, ETR and roadways; if zero net-loading used, impact may be comparable to four span structure Lightweight fill option with least potential for differential settlement between abutments and Island approach slabs 	<ul style="list-style-type: none"> Settlement mitigation required at abutments and Island unless net zero load approach implemented Although wick drains can be used to accelerate time to complete primary consolidation, alternative/complementary measures such as preloading or surcharging will be difficult to implement due to schedule and site configuration Excavations in range of 1.5 to 2.0 m required for net zero load approach Dewatering or cut-off of surficial granular deposits would be required in order to erect facing panels if a net zero load approach is used EPS fill is the most expensive lightweight fill option Facing panels must be structurally tied together EPS blocks need protection from hydrocarbon or chemical spills, fire, UV light and buoyancy in a net zero load application Settlement monitoring required Increased engineering time and costs should problems arise during fill placement 	<ul style="list-style-type: none"> 5.0 	<ul style="list-style-type: none"> Low to moderate risk Buried utilities within 10 m of Island subject to 15 to 40 mm (total) of fill induced settlement; further reductions possible if zero net load approach used To prevent fire damage blocks must have soil cover and facing panels must have min. 2 hour fire rating Hydrocarbon or chemical spills to prevent damage from blocks must be protected by hydrocarbon-resistant geomembrane A 125 mm thick concrete slab must be provided to minimize crushing of the EPS and reflection cracking of pavements associated with EPS block joints UV, wind and water exposure must be limited Sheet piles may be needed to protect utilities and limit dewatering effort

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: WH/DUP

Checked By: SJB

TABLE II

COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES

Bridge B-1
Ojibway Parkway/ETR Overpass, Windsor, Ontario
GWP 3028-14-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
End bearing steel H-piles to refusal on bedrock	<ul style="list-style-type: none"> Feasible (preferred technical alternative). 	<ul style="list-style-type: none"> Compatible with integral abutments Non-displacement pile Minimal mitigation of artesian groundwater conditions and subsurface gases required Ease of construction 	<ul style="list-style-type: none"> Vibration related damage may still occur if proper controls not enacted 	<ul style="list-style-type: none"> 3.0 	<ul style="list-style-type: none"> Low to moderate risk. Risk dependent on care and control taken during installation
End bearing steel pipe piles driven to refusal on bedrock	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Subject to problems with flowing artesian conditions and gases near bedrock surface depending on driving shoe (or cap) size and methods for filling interior and surface area 	<ul style="list-style-type: none"> Not typically compatible with integral abutments Displacement piles resulting in higher vibration magnitudes compared to H-piles 	<ul style="list-style-type: none"> 3.0 	<ul style="list-style-type: none"> Moderate risk, generally incompatible with integral abutments

COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	ESTIMATED RELATIVE COST FACTOR¹	RISKS/ CONSEQUENCES
End bearing concrete filled caissons socketed into bedrock	<ul style="list-style-type: none"> • Geotechnically feasible but not recommended 	<ul style="list-style-type: none"> • Non-displacement installation • Minimal vibration concerns 	<ul style="list-style-type: none"> • Artesian water flows and subsurface gases will cause installation complexities 	<ul style="list-style-type: none"> • 4.0+ 	<ul style="list-style-type: none"> • Moderate risk • Risk dependent on anticipated and actual water pressures and flow rates during installation • Risk of subsurface gas migration • Artesian conditions and subsurface gases preclude proper inspection of base • Loss of ground if temporary casings not used
Shallow Foundations	<ul style="list-style-type: none"> • Not feasible, acceptable only for island structure foundations 	<ul style="list-style-type: none"> • Least expensive foundation option • No vibration concerns 	<ul style="list-style-type: none"> • Geotechnically unfeasible; soils do not provide strength required for loads 	<ul style="list-style-type: none"> • 1.0 	<ul style="list-style-type: none"> • High, unacceptable risk

- 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: WH/DUP
 Checked By: SJB



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

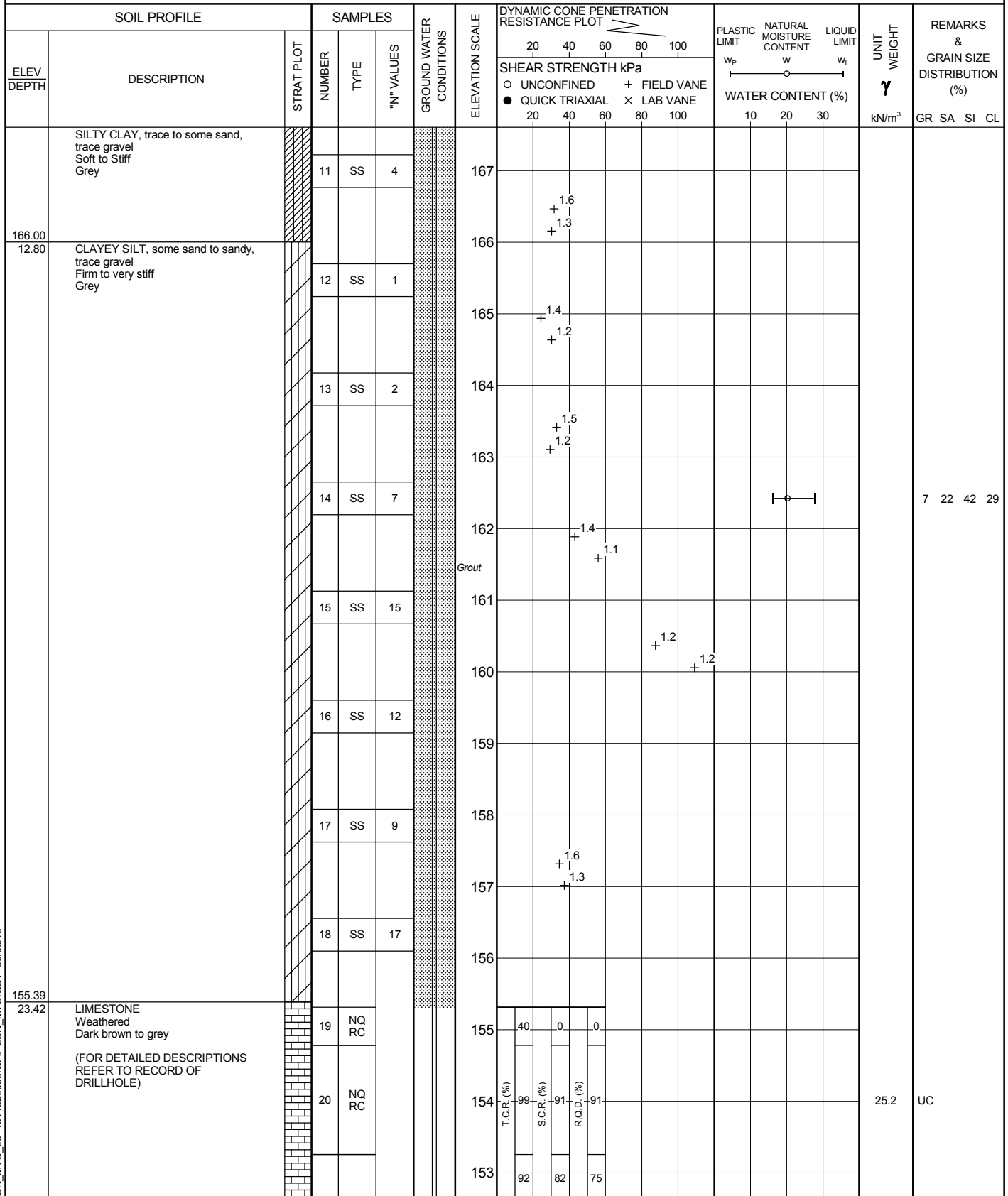
PROJECT <u>13-1132-0053</u>		RECORD OF BOREHOLE No BH15-001		1 OF 3	METRIC
W.P. <u>3028-14-00</u>	LOCATION <u>N 4682135.6 , E 328411.5</u>	ORIGINATED BY <u>AP</u>			
DIST <u></u> HWY <u>RHHGP</u>	BOREHOLE TYPE <u>HOLLOW STEM, MUD ROTARY, WASHBORE</u>	COMPILED BY <u>DCH</u>			
DATUM <u>GEODETIC</u>	DATE <u>November 16 - 18, 2015</u>	CHECKED BY <u></u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)					
							20 40 60 80 100	20 40 60 80 100	10 20 30				GR SA SI CL	
178.81	GROUND SURFACE						▽							
0.00	FILL, sand, with organics Compact Black													
177.76			1	SS	11									
1.04	SAND, some silt Compact Brown													
177.23														
1.63	SAND, some silt, with organics Compact Black		2	SS	20									
176.55														
2.26	SAND Compact Black		3	SS	9									
	SILTY CLAY, trace to some sand, trace gravel Soft to Stiff Grey													
			4	SS	5									
			5	SS	7									
			6	SS	5									
			7	SS	4									
			8	TO	PH									
			9	SS	3									
			10	TO	PH									

Continued Next Page

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

PROJECT <u>13-1132-0053</u>		RECORD OF BOREHOLE No BH15-001		2 OF 3	METRIC
W.P. <u>3028-14-00</u>	LOCATION <u>N 4682135.6 , E 328411.5</u>	ORIGINATED BY <u>AP</u>			
DIST <u> </u> HWY <u>RHHGP</u>	BOREHOLE TYPE <u>HOLLOW STEM, MUD ROTARY, WASHBORE</u>	COMPILED BY <u>DCH</u>			
DATUM <u>GEODETIC</u>	DATE <u>November 16 - 18, 2015</u>	CHECKED BY <u> </u>			



LDN_MTO_06 1311320053.GPJ LDN_MTO_GDT 06/06/16

Continued Next Page

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

PROJECT <u>13-1132-0053</u>		RECORD OF BOREHOLE No BH15-001		3 OF 3	METRIC
W.P. <u>3028-14-00</u>	LOCATION <u>N 4682135.6 , E 328411.5</u>	ORIGINATED BY <u>AP</u>			
DIST <u></u> HWY <u>RHHGP</u>	BOREHOLE TYPE <u>HOLLOW STEM, MUD ROTARY, WASHBORE</u>	COMPILED BY <u>DCH</u>			
DATUM <u>GEODETIC</u>	DATE <u>November 16 - 18, 2015</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	REMARKS & GRAIN SIZE DISTRIBUTION (%)								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)												
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE		W _p	W	W _L										
								20	40	60	80	100	20	40	60	80	100	10	20	30	kN/m ³	GR	SA	SI	CL
151.73			21	NQ																	24.4	UC			
27.08	END OF BOREHOLE																								
	Artesian groundwater conditions observed at bedrock interface on November 17, 2015. Flow rate of 1 L/S and 1.4 L/S measured on November 17, 2015 and November 18, 2015, respectively Water level measured at elev. 182.07m on November 18, 2015 after installation. Water level measured at elev. 181.92m on January 13, 2016. Water level measured at elev. 182.38m on May 16, 2016.																								

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR:

DATUM: GEODETIC



**Golder
Associates**

CHECKED:

1 : 75

DN_ROCK_08 1311320053-ROCK.GPJ GLDR_LDN.GDT 29/11/16 DATA INPUT: DCH

PROJECT: 13-1132-0053

RECORD OF CONE PENETRATION TEST CPT15-1001

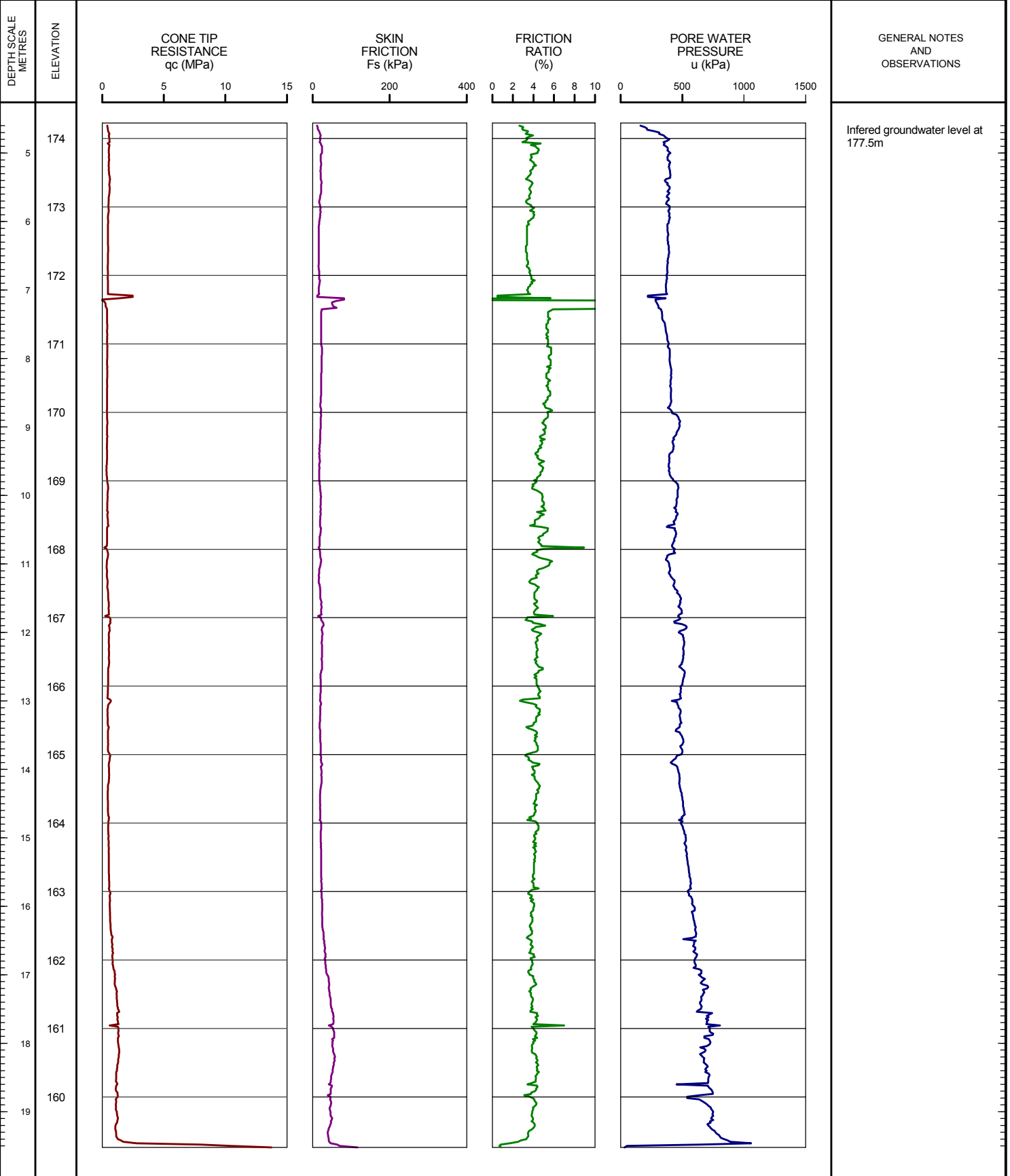
SHEET 1 OF 1

LOCATION: N 4682192.2 , E 328420.3

TEST DATE: November 16, 2015

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.79m PREDRILL DEPTH: 4.56m CORRECTION FACTOR A: 0.59 CORRECTION FACTOR B: 0.014



LDN_CPT_01 1311320053.GPJ GLDR_LON.GDT 24/02/16 DATA INPUT:

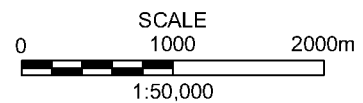
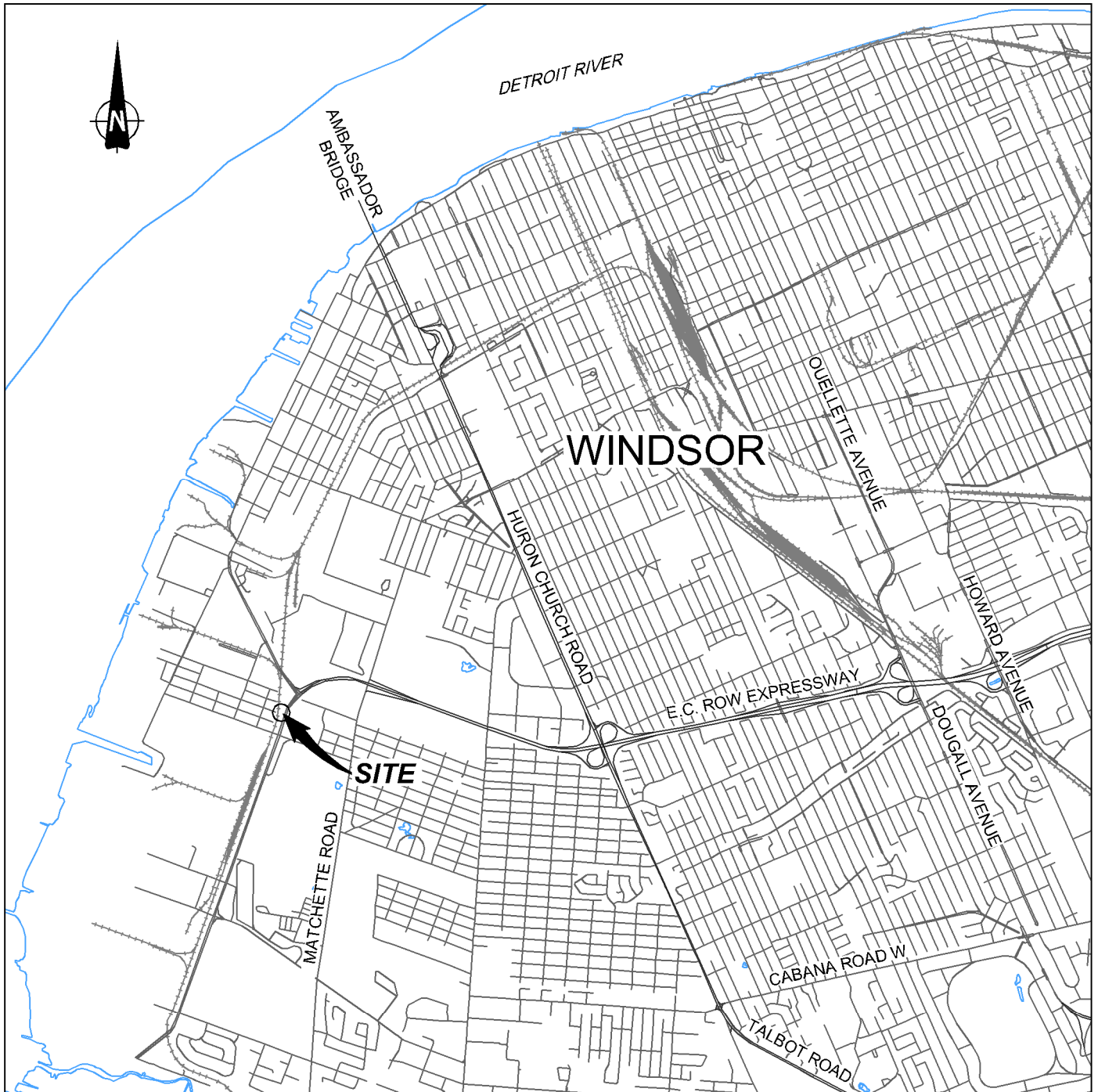
DEPTH SCALE

1 : 75



OPERATOR: AS

CHECKED:



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

NOTE

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

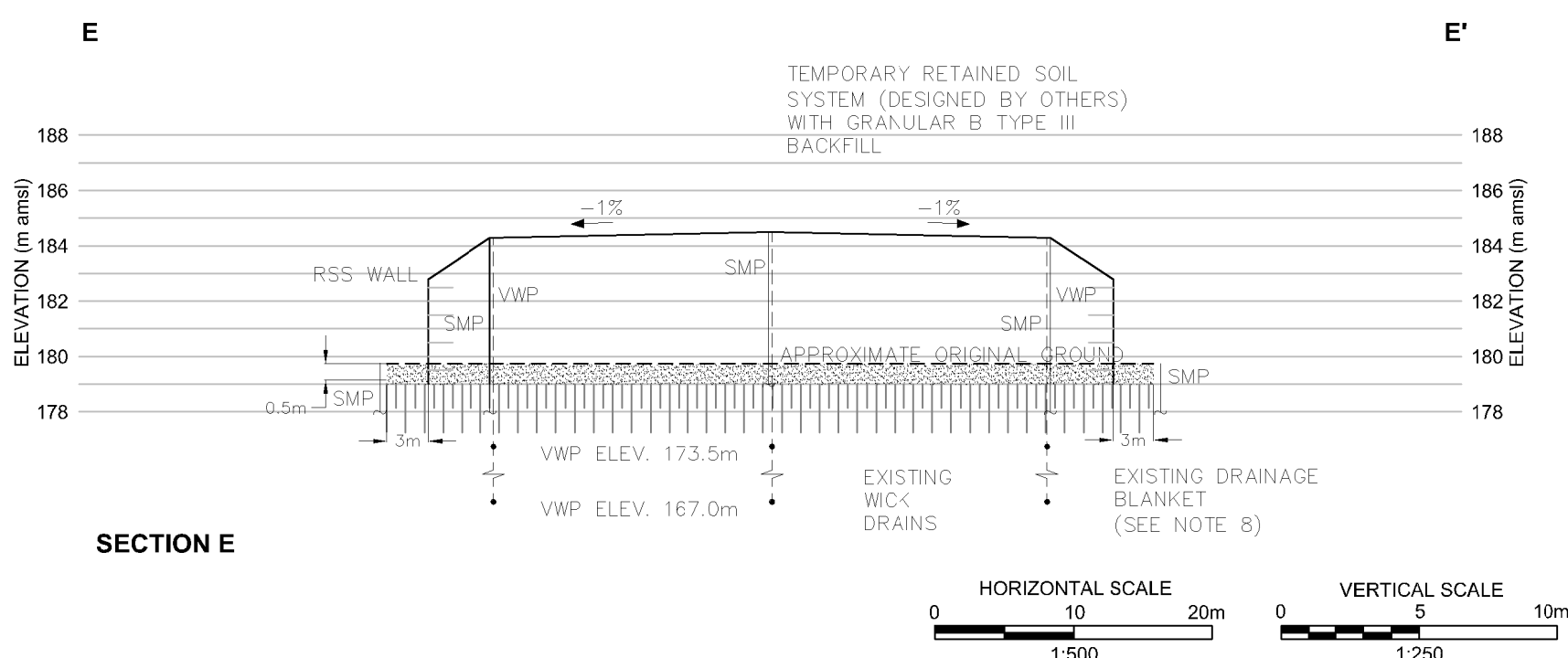
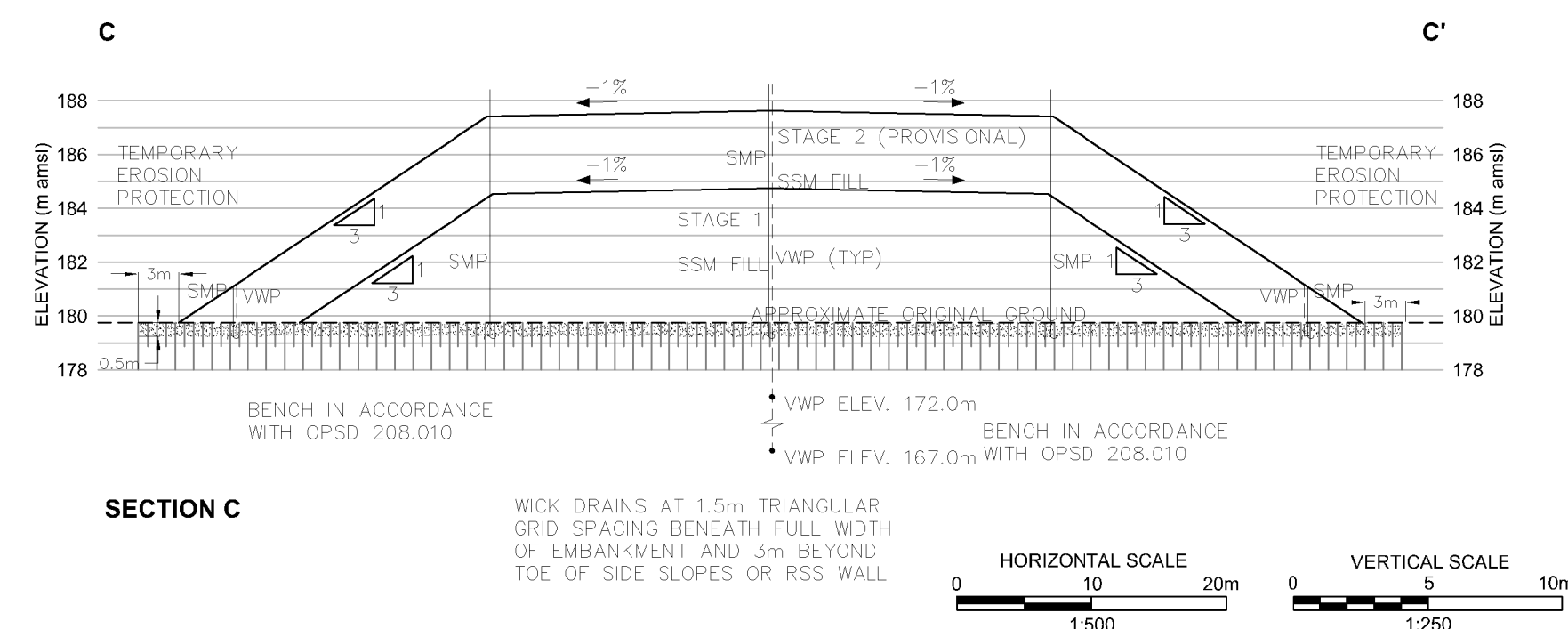
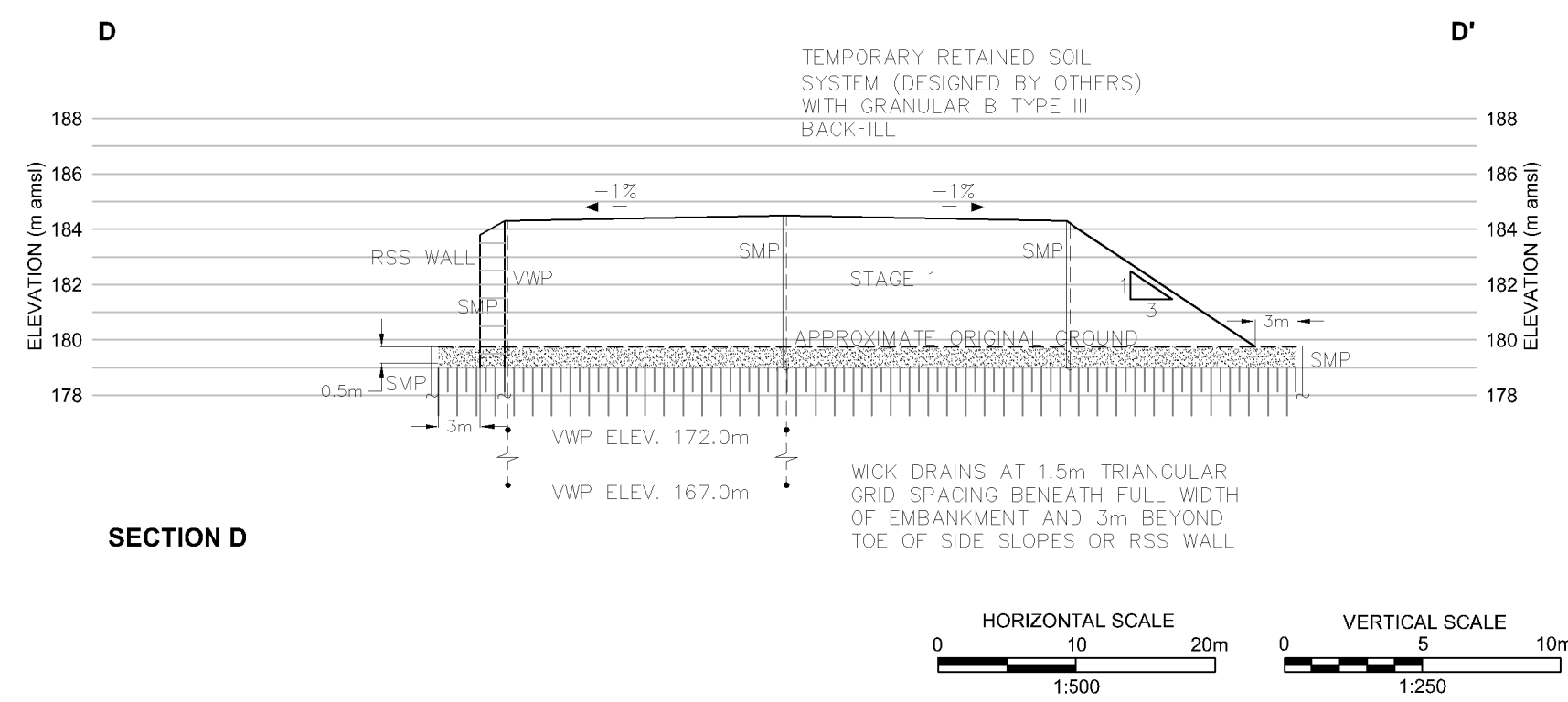
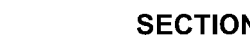
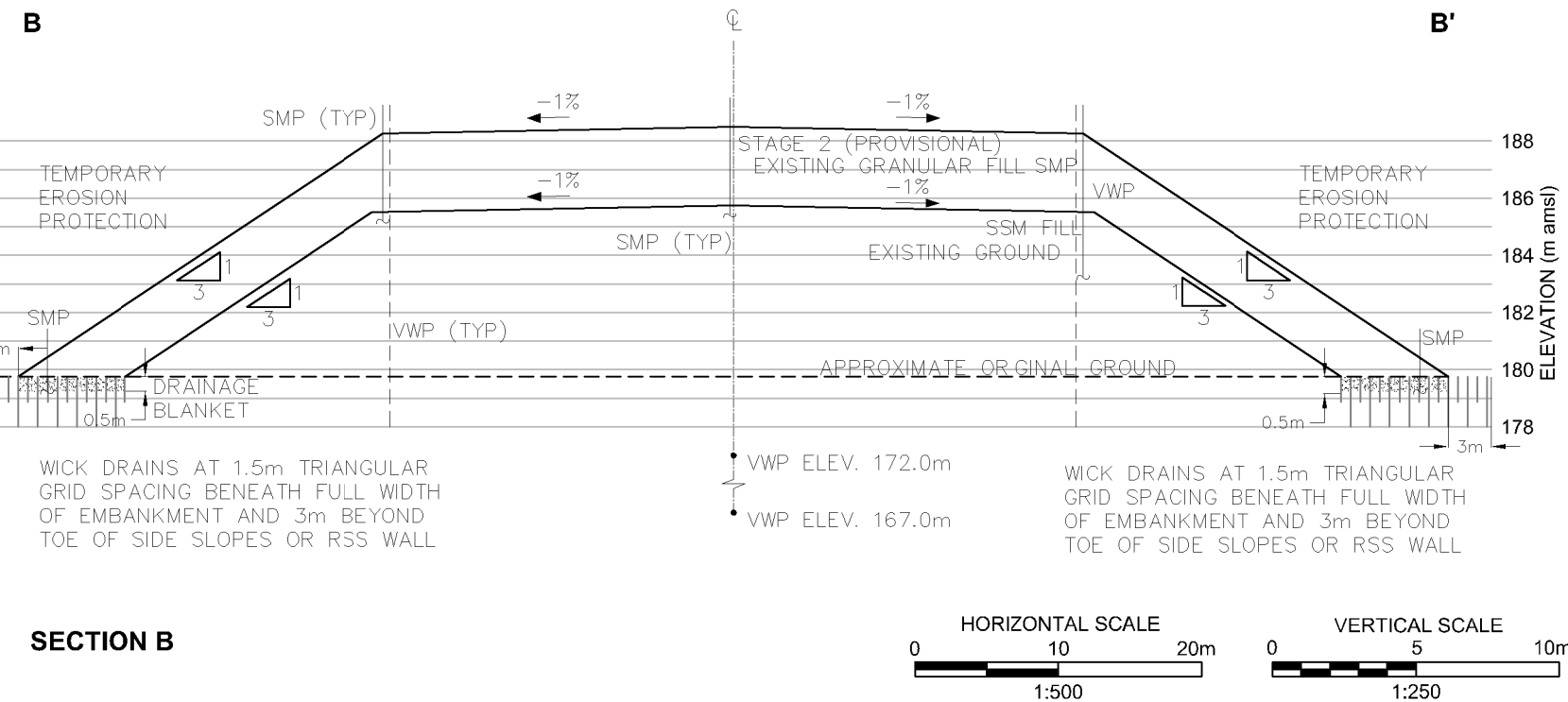
PROJECT
OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2
HIGHWAY 401 (RHHGP)
GWP 3028-14-00

TITLE

KEY PLAN



PROJECT No. 13-1132-0053			FILE No. 1311320053-1000-F01001	
CADD	DCH	May 20/16	SCALE AS SHOWN	REV. 0
CHECK			FIGURE 1	



- ## NOTES

1. WICKS DRAINS SHALL BE INSTALLED WITHIN THE AREA SHOWN AT UNIFORM 1.5m TRIANGULAR CENTRE-TO-CENTRE GRID DISTANCES.
2. WICK DRAIN SHALL BE INSTALLED TO A TIP ELEVATION OF 150 AND NO DEEPER.
3. THE WICK DRAIN TERMINATION LIMIT ELEVATION OF 158m WAS SELECTED TO MINIMIZE THE POTENTIAL FOR ENCOUNTERING GROUNDWATER UNDER THE ASSUMPTION THAT IT IS PRESENT AND TO PROVIDE AN ADDITIONAL 8m GRANULAR SLOPE IMMEDIATELY OVERLYING BEDROCK AND ALSO KNOWN TO CONTAIN HYDROGEN SULFIDE. IF ARTESIAN WATER FLOW IS ENCOUNTERED THAT PRODUCES TIME-WEIGHTED AVERAGE ATMOSPHERIC CONCENTRATIONS OF HYDRO SULFIDE OF 10PPM OR GREATER AT OR ABOVE AN ELEVATION OF 1m ABOVE THE WICK DRAIN SURFACE PROJECTION, THE CONTRACTOR SHALL UNDERTAKE NECESSARY MEASURES TO ADEQUATELY PROTECT WORKERS AND THE WICK DRAIN TERMINATION DEPTH SHALL INCREASED TO ELEVATION 160m FOR ALL WICK DRAINS WITHIN 30m OF THE LOCATION WHERE THE CONDITIONS WERE FIRST ENCOUNTERED.
4. A MINIMUM CONSOLIDATION PERIOD OF 30 CALENDAR DAYS SHALL ELAPSE ONCE FILL AND SURCHARGE GRADES REACH A HEIGHT OF 5m ABOVE THE EXISTING GROUND SURFACE ELEVATION. NO FILLING ABOVE THIS LEVEL SHALL OCCUR DURING THE CONSOLIDATION PERIOD. THIS IS APPLICABLE TO STAGE 1 AND 2.
5. WHERE THE FILL AND SURCHARGE SURFACE IS TO BE GREATER THAN 5m ABOVE EXISTING GRADE, FILL SHALL BE PLACED AT A VERTICAL RATE SUCH THAT THE VERTICAL SURFACE HEIGHT IS INCREASED NO FASTER THAN 1 cm PER 50 CALENDAR DAYS WITHOUT WRITTEN PERMISSION OF THE CONTRACT ADMINISTRATOR AND THE FILL PLACEMENT RATE SHALL BE SUCH THAT ALERT LEVELS FOR PNEUMATIC PRESSURE READINGS AS SPECIFIED IN THE SPECIAL SPECIFICATIONS ARE NOT EXCEEDED AT ANY LOCATION.
6. PLACEMENT OF STAGE 2 FILLS SHALL NOT COMMENCE UNTIL THE PRELOAD ENGINEERING CONSULTANT CONFIRMS THAT EXCESS POREWATER PRESSURES GENERATED DUE TO CONSTRUCTION OF THE STAGE 1 PRELOAD HAVE SUFFICIENTLY DISSIPATED.
7. EMBANKMENT SIDE SLOPES AND TEMPORARY SLOPES SHALL NOT BE STEEPER THAN 3:1 AT ANY TIME.
8. FIRST SETTLEMENT, CONFIRM INSTALLATION OF WICK DRAINS, AND FILLING OF THE DRAINAGE BLANKET SHALL BE PRIOR TO PLACEMENT OF PRELOAD AND BACKFILL FOR TEMPORARY RSS WALL. PRELOAD MATERIAL SHALL NOT BE PLACED IN THE ABSENCE OF WICK DRAINS AND A DRAINAGE BLANKET.

REFERENCE

DRAWING SUPPLIED BY AECOM, Herb Gray
Parkway_plan.dwg, Herb Gray Parkway_profile.dwg
SEPTEMBER 14, 2016.

NOTES

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

ALL LOCATIONS ARE APPROXIMATE ONLY

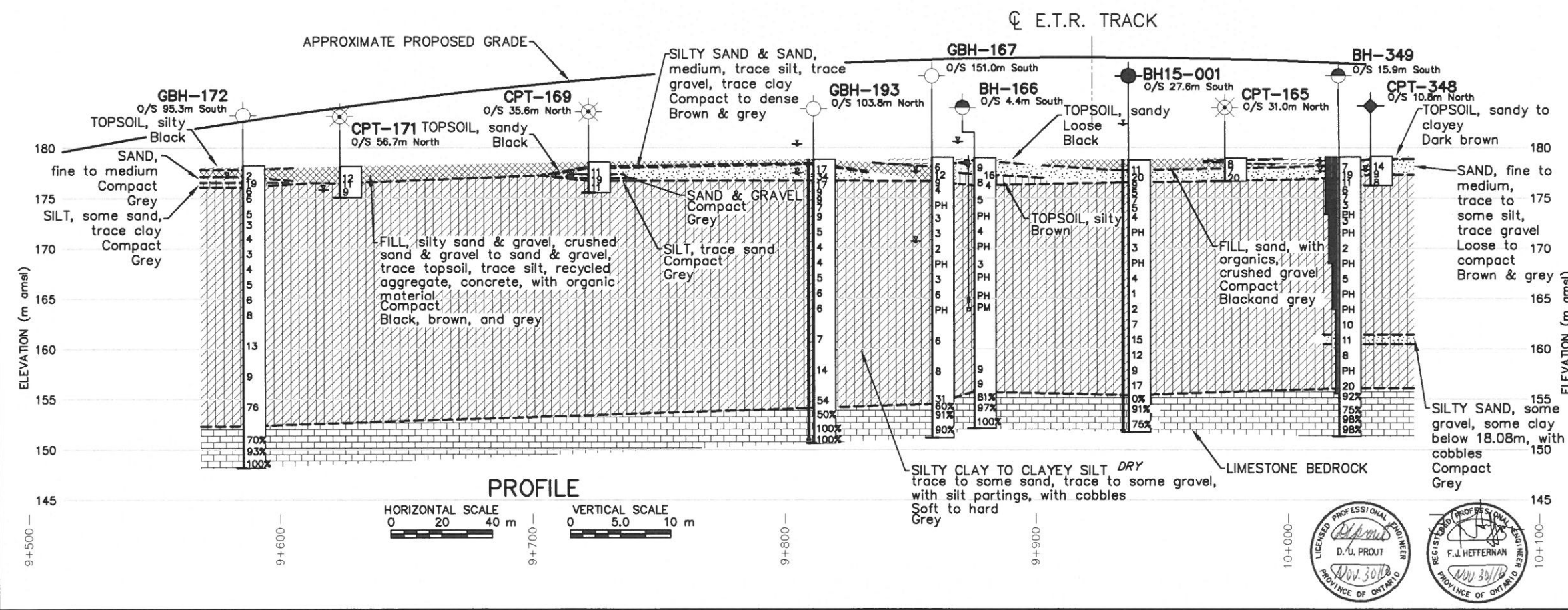
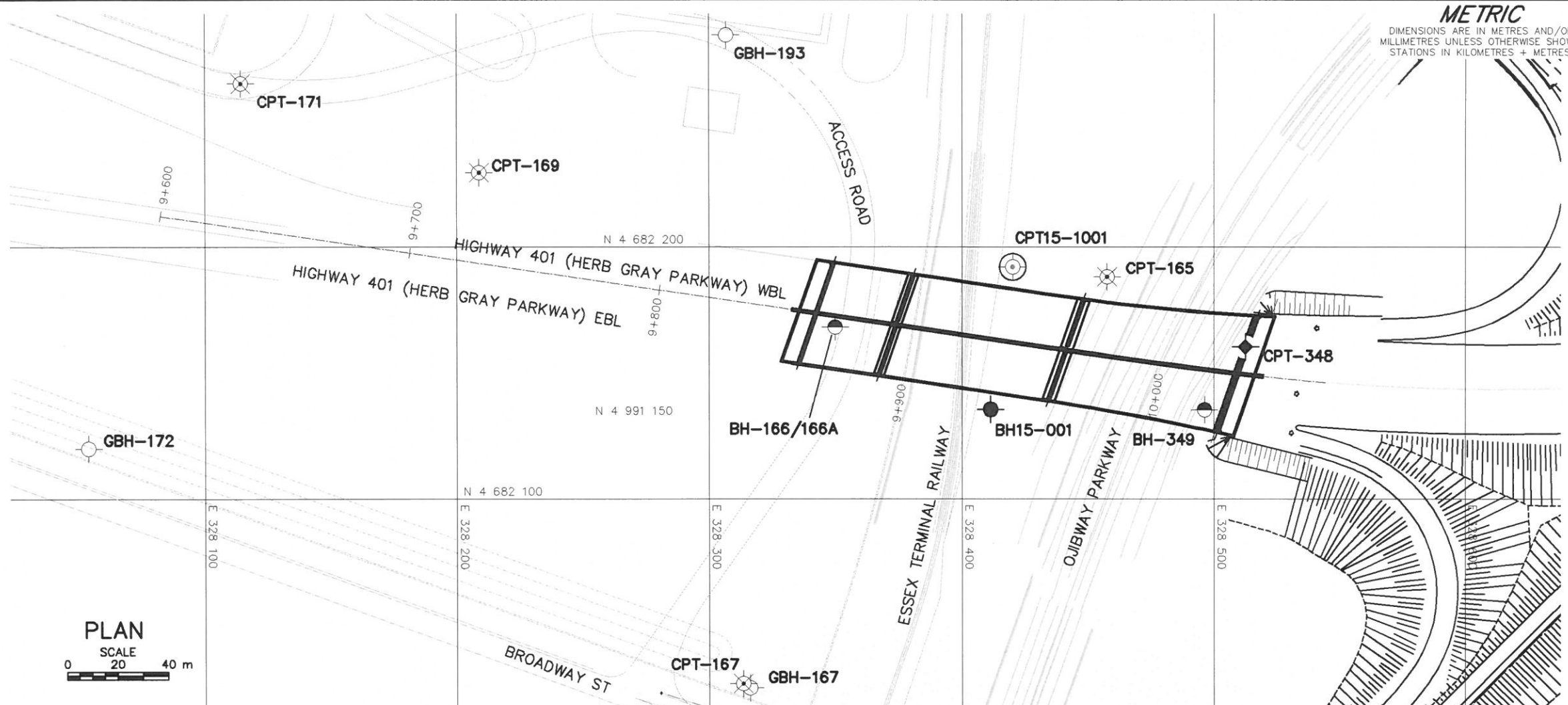
PROJECT
OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2
HIGHWAY 401 (RHHGP)
GWP 3028-14-00

PRELOAD FILL PLACEMENT AND SETTLEMENT MONITORING



PROJECT No. 13-1132-0053			FILE No. 1311320053-1000-F0100		
			SCALE AS SHOWN		REV.
CADD	LMK	Nov. 18/16	FIGURE 2		
CHECK					

FIGURE 2



CONT No.
WP No. 3028-14-00

OJIBWAY PARKWAY/ETR OVERPASS

HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
LONDON, ONTARIO, CANADA

KEY PLAN
SCALE IN KILOMETRES 0 2 4

LEGEND

- Borehole - Current Investigation
- CPT - Current Investigation
- Borehole - Previous Golder Project 0911320039-1000
- CPT - Previous Golder Project 0911320039-1000
- Borehole - GEOCRETS No. 40J6-27
- CPT - GEOCRETS No. 40J6-27
- Seal
- Standpipe/Vibrating wire
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Measured WL. (May 16, 2016)
- Encountered WL.

No.	ELEV.	CO-ORDINATES (UTM, Nad83, ZONE 17)	
		NORTHING	EASTING
BH15-001	178.81	4 68 2192.2	328 411.5
CPT15-1001	186.52	4 682 135.6	328 420.3
GOLDER REPORT 09-1132-0039			
GBH-167	179.03	4 682 025.1	328 316.1
CPT-167	178.91	4 682 026.8	328 313.4
CPT-169	178.57	4 682 229.5	328 208.8
CPT-171	178.14	4 682 264.8	328 114.3
GBH-172	178.23	4 682 120.1	328 054.0
GBH-193	178.85	4 682 284.0	328 306.8
GEOCRETS 40J6-27			
CPT-165	178.98	4 682 188.2	328 457.7
BH-166/166A	179.00	4 682 168.3	328 349.6
CPT-348	179.15	4 682 160.4	328 512.5
BH-349	179.08	4 682 135.5	328 496.2

NOTES

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

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

REFERENCE

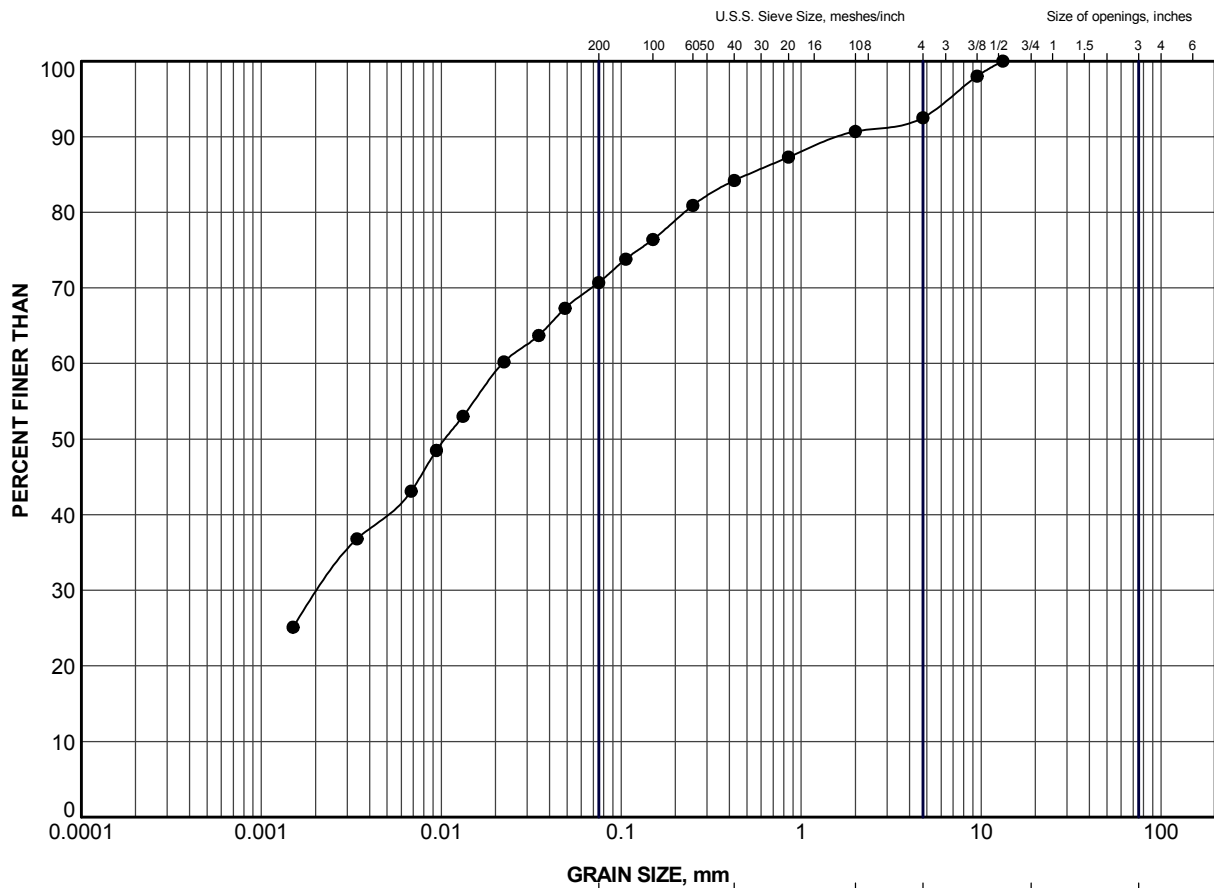
Base plans provided in digital format by AECOM
 Received December 10, 2015 and July 7, 2016.

NO.	DATE	BY	REVISION
Geocres No. 40J6-71			
HWY.	401	PROJECT NO.	13-1132-0053
SUBM'D. WH	CHKD. DUP	DATE:	June 7/16
DRAWN:	WDF/DCH	CHKD. SJB	APPD. FJH
DIST.		SITE: 6-500/1 & 2	
DWG.		1	




APPENDIX A

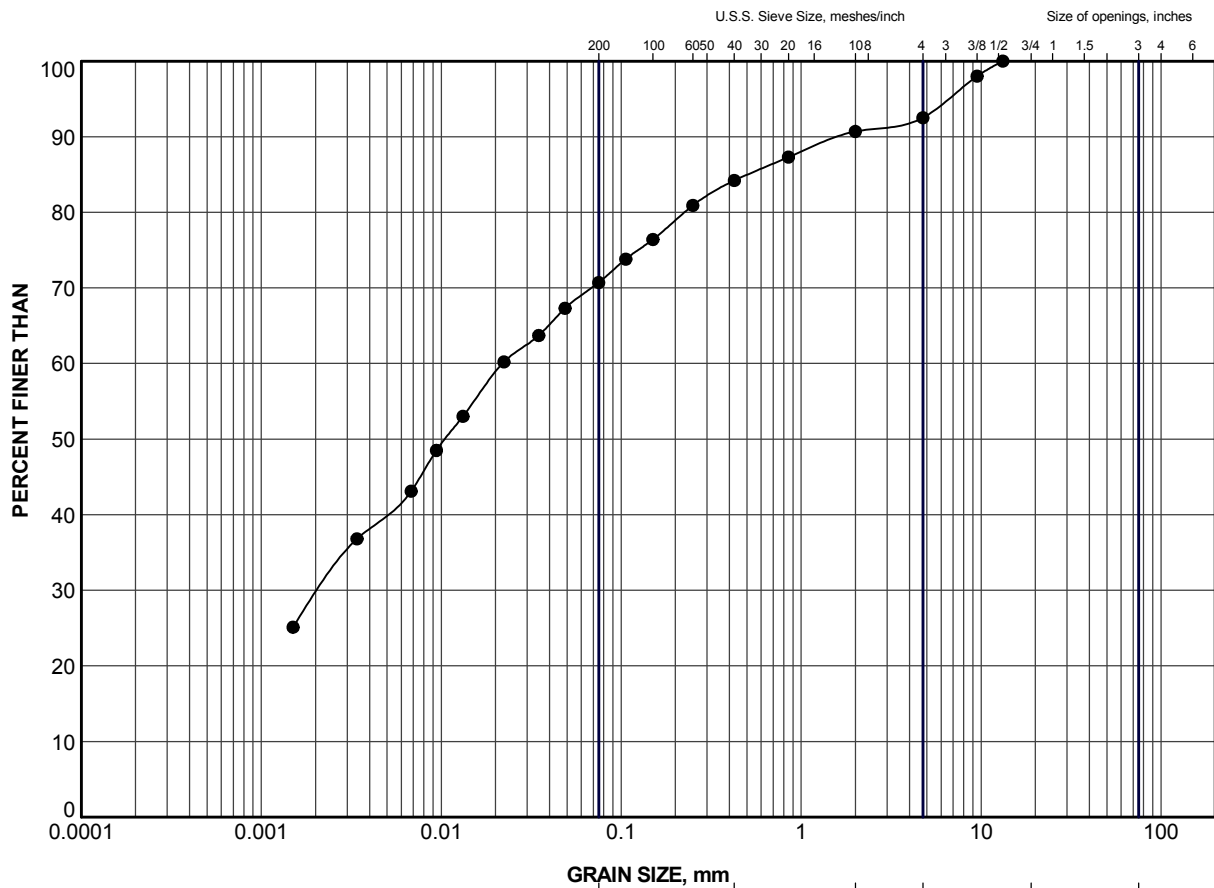
Laboratory Test Data - Soil



LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH15-001	14	162.4

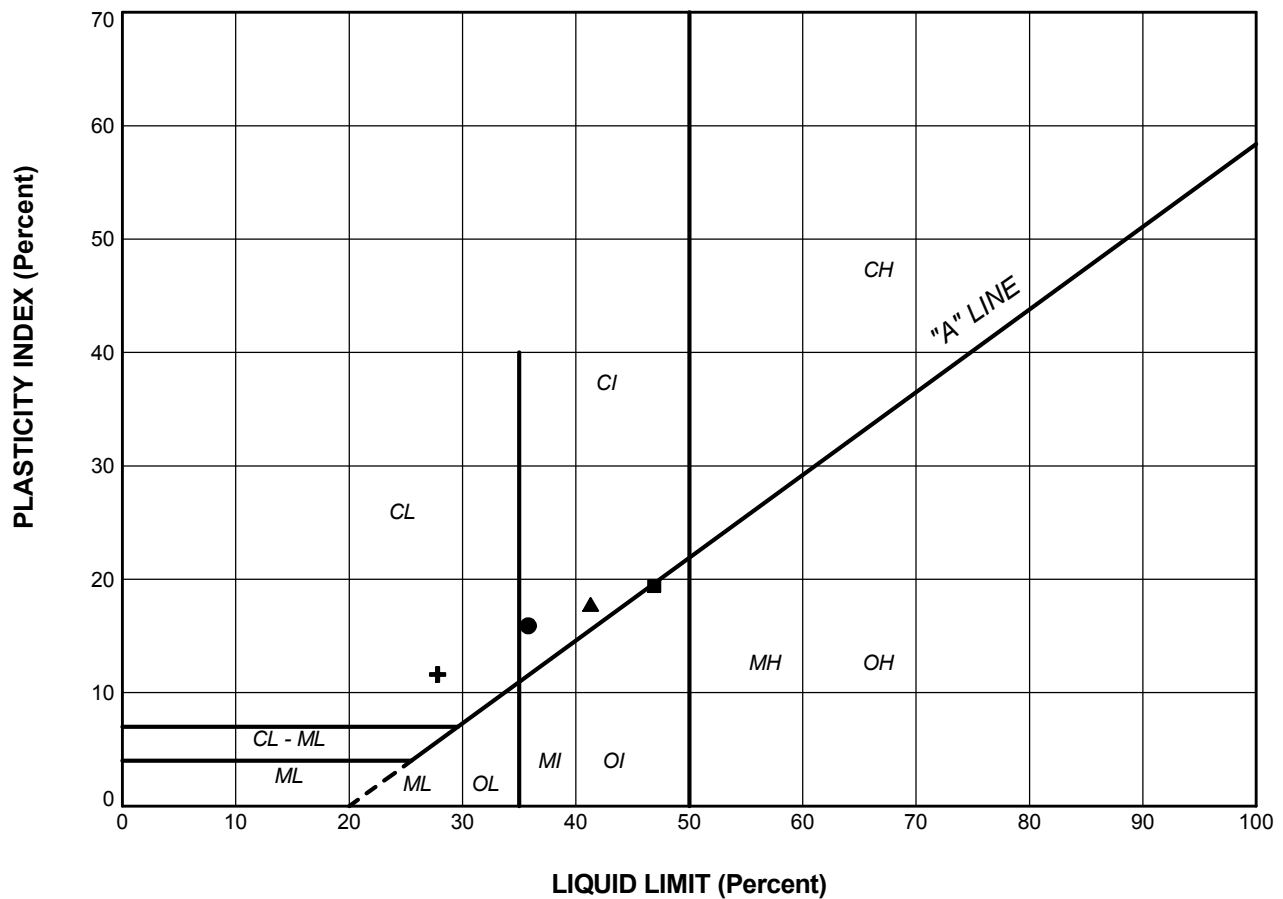
PROJECT			
OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2 HIGHWAY 401 (RHHGP) GWP 3028-14-00			
TITLE			
GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No. 13-1132-0053		FILE No. 1311320053-R010A1	
DRAWN	DCH	May 20/16	SCALE N/A REV.
CHECK			
			FIGURE A-1



LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH15-001	14	162.4

PROJECT				
OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2 HIGHWAY 401 (RHHGP) GWP 3028-14-00				
TITLE				
GRAIN SIZE DISTRIBUTION CLAYEY SILT				
PROJECT No.		13-1132-0053		FILE No.
				1311320053-R1002
DRAWN	DCH	May 20/16	SCALE	N/A
CHECK			REV.	
			FIGURE A-2	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
SILTY CLAY					
●	BH15-001	5	35.8	19.9	15.9
■	BH15-001	8	46.9	27.5	19.4
▲	BH15-001	10	41.3	23.5	17.8
CLAYEY SILT					
+	BH15-001	14	27.8	16.2	11.6

PROJECT OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2 HIGHWAY 401 (RHHGP) GWP 3028-14-00			
TITLE PLASTICITY CHART			
PROJECT No. 13-1132-0053		FILE No. 1311320053-R010A3	
DRAWN	DCH	May 20/16	SCALE N/A
CHECK			REV.
 Golder Associates			FIGURE A-3

CONSOLIDATION TEST SUMMARY**ASTM D2435/D2435M****FIGURE A-4A****SAMPLE IDENTIFICATION**

Project Number	13-1132-0053(1000)	Sample Number	8
Borehole Number	BH15-001	Sample Depth, m	7.01-7.47

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	11/27/2015		
Date Completed	12/11/2015		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	18.11
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	12.93
Area, cm ²	31.55	Specific Gravity, measured	2.78
Volume, cm ³	60.04	Solids Height, cm	0.903
Water Content, %	40.06	Volume of Solids, cm ³	28.47
Wet Mass, g	110.87	Volume of Voids, cm ³	31.56
Dry Mass, g	79.16	Degree of Saturation, %	100.5

TEST COMPUTATIONS

Stress	Corr. Height	Void	Average Height	t ₉₀	cv.	mv	k
kPa	cm	Ratio	cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.903	1.108	1.903				
6.51	1.903	1.108	1.903				
11.26	1.906	1.112	1.905				
21.00	1.899	1.104	1.903	317	2.42E-03	4.10E-04	9.73E-08
40.50	1.884	1.087	1.891	470	1.61E-03	4.02E-04	6.35E-08
79.53	1.860	1.060	1.872	591	1.26E-03	3.27E-04	4.03E-08
165.91	1.814	1.009	1.837	577	1.24E-03	2.79E-04	3.39E-08
311.83	1.701	0.885	1.757	1441	4.54E-04	4.06E-04	1.81E-08
622.72	1.573	0.743	1.637	1162	4.89E-04	2.16E-04	1.03E-08
1241.83	1.464	0.622	1.519	960	5.09E-04	9.25E-05	4.62E-09
2483.82	1.361	0.508	1.413	487	8.69E-04	4.35E-05	3.71E-09
1241.83	1.379	0.528	1.370				
311.83	1.446	0.602	1.412				
79.35	1.531	0.696	1.488				
21.23	1.606	0.779	1.568				
6.41	1.670	0.850	1.638				

Note:

Specimen taken 7-13cm from bottom of the tube

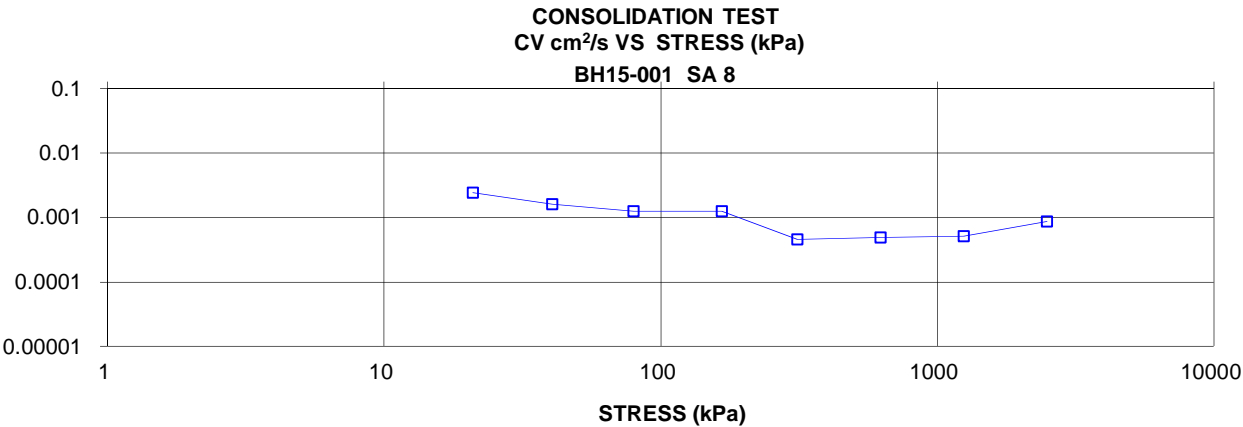
k calculated using cv based on λ_0 values.

Specimen swelled under 11.26 kPa.

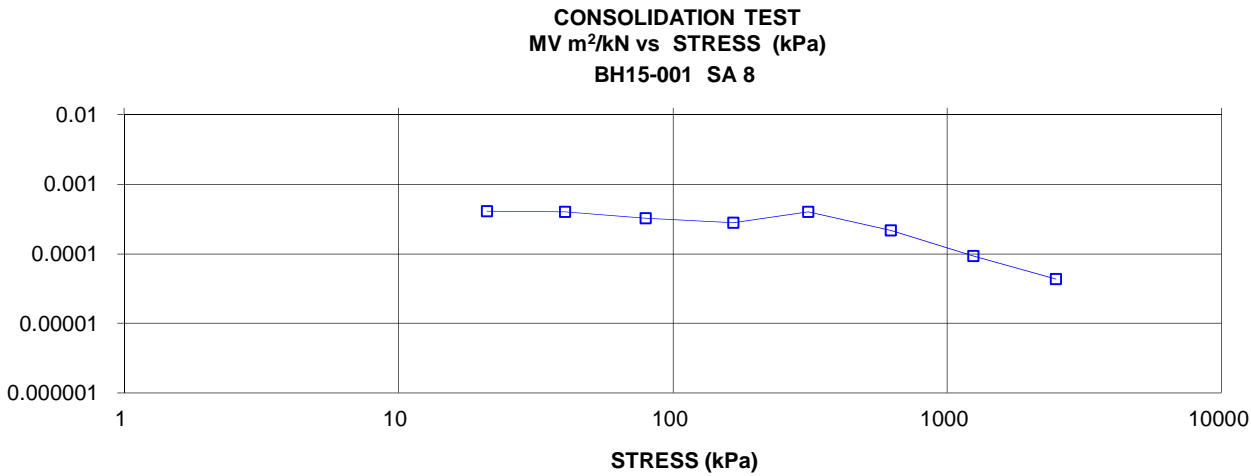
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.67	Unit Weight, kN/m ³	19.64
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	14.73
Area, cm ²	31.55	Specific Gravity, measured	2.78
Volume, cm ³	52.68	Solids Height, cm	0.903
Water Content, %	33.31	Volume of Solids, cm ³	28.47
Wet Mass, g	105.53	Volume of Voids, cm ³	24.21
Dry Mass, g	79.16		

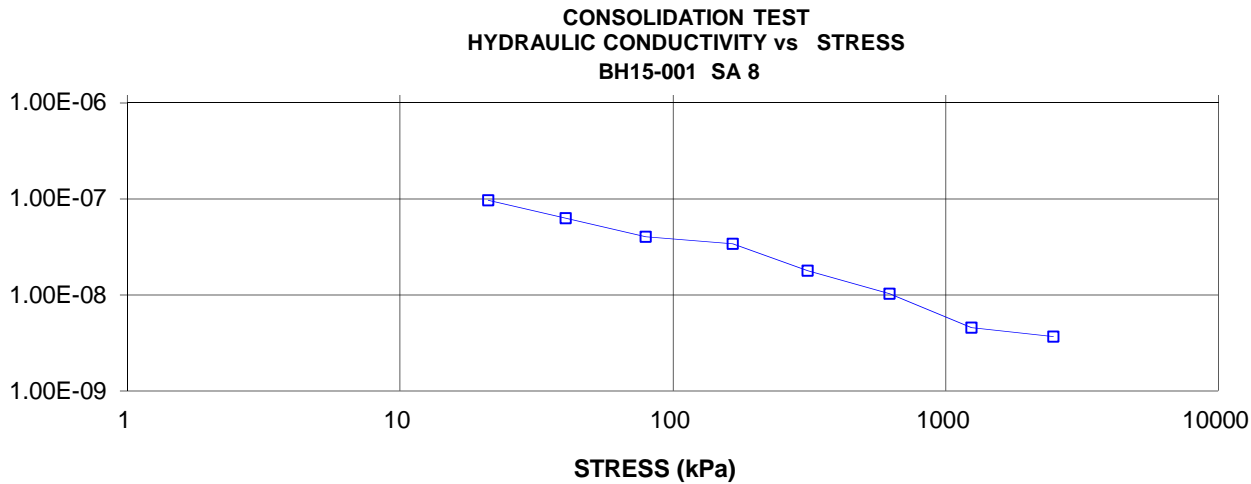
COEFFICIENT OF CONSOLIDATION,
cm²/s

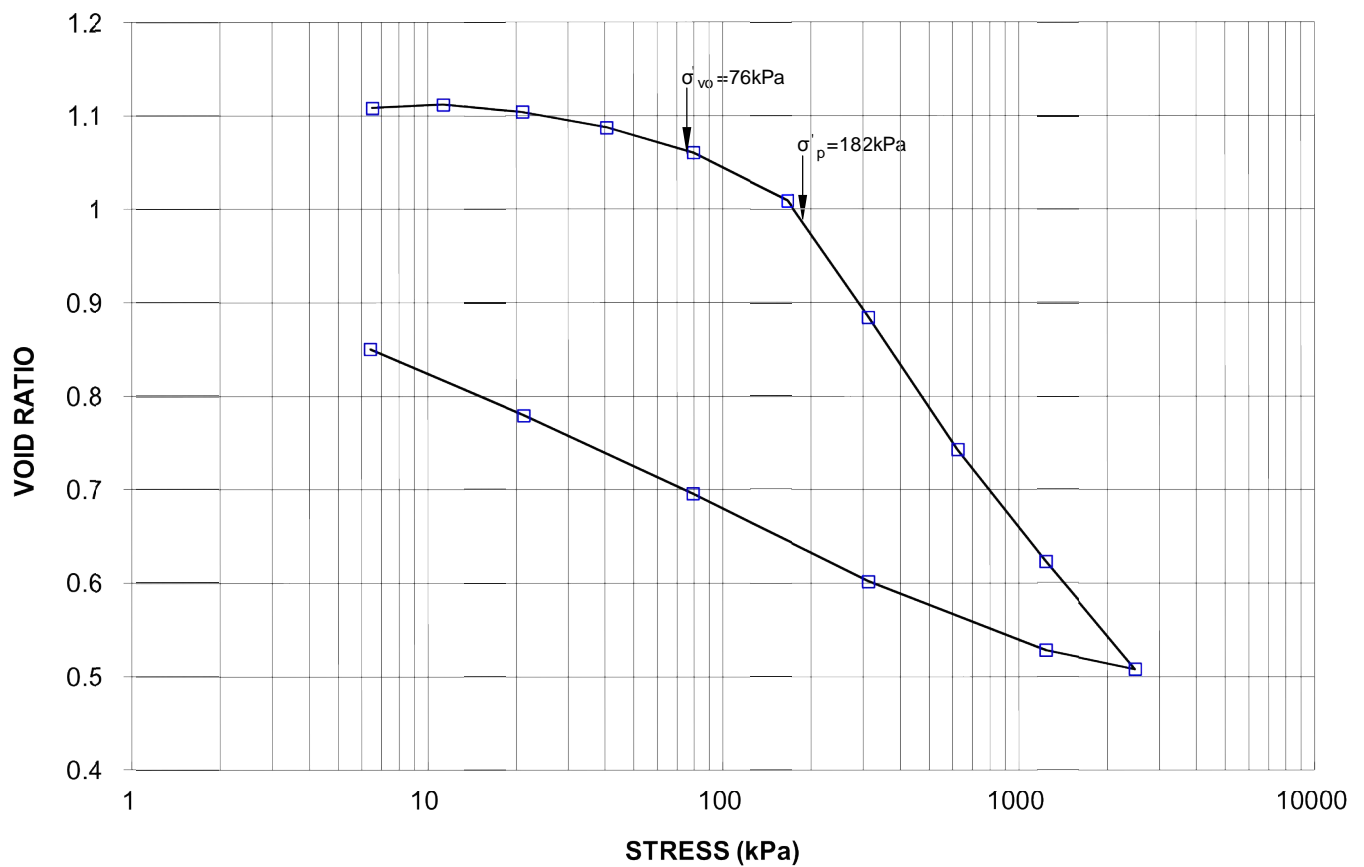


VOLUME COMPRESSIBILITY, m²/kN




HYDRAULIC CONDUCTIVITY,
cm/s



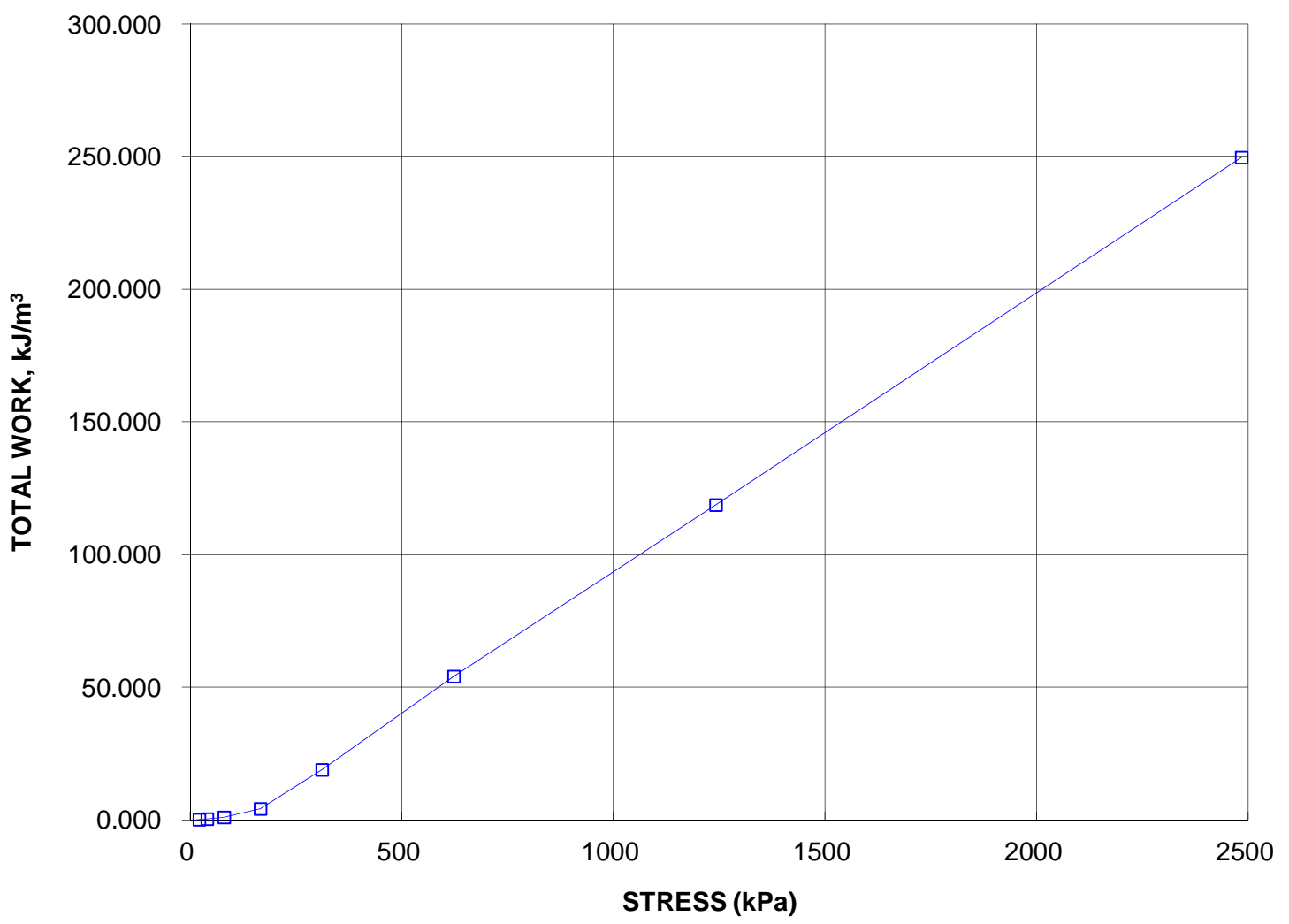


BOREHOLE 15-001 SA 8

PROJECT							
OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2 HIGHWAY 401 (RHHGP) GWP 3028-14-00							
TITLE							
CONSOLIDATION TEST VOID RATIO Vs. LOG PRESSURE							
 Golder Associates	PROJECT No.		13-1132-0053	FILE No. 1311320053-1000-F010A4C			
				SCALE	AS SHOWN	REV.	0
	CADD	DCH	May 20/16	FIGURE A-4C			
	CHECK						



CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs STRESS
BH15-001 SA 8



Project No. 13-1132-0053(1000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST SUMMARY**FIGURE A-5A****ASTM D2435/D2435M****SAMPLE IDENTIFICATION**

Project Number	13-1132-0053(1000)	Sample Number	10
Borehole Number	BH15-001	Sample Depth, m	10.06-10.52

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	11/27/2015		
Date Completed	12/10/2015		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m ³	17.67
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	12.29
Area, cm ²	31.62	Specific Gravity, measured	2.78
Volume, cm ³	59.82	Solids Height, cm	0.853
Water Content, %	43.80	Volume of Solids, cm ³	26.97
Wet Mass, g	107.82	Volume of Voids, cm ³	32.85
Dry Mass, g	74.98	Degree of Saturation, %	100.0

TEST COMPUTATIONS

Stress	Corr. Height	Void Ratio	Average Height	t ₉₀	cv.	mv	k
kPa	cm		cm	sec	cm ² /s	m ² /kN	cm/s
0.00	1.892	1.218	1.892				
5.93	1.884	1.208	1.888				
10.57	1.882	1.206	1.883				
20.53	1.865	1.187	1.873	118	6.30E-03	8.70E-04	5.38E-07
40.04	1.841	1.158	1.853	421	1.73E-03	6.56E-04	1.11E-07
78.72	1.811	1.123	1.826	485	1.46E-03	4.04E-04	5.78E-08
156.26	1.766	1.071	1.789	406	1.67E-03	3.07E-04	5.02E-08
311.09	1.655	0.940	1.711	866	7.16E-04	3.79E-04	2.66E-08
621.11	1.520	0.782	1.587	913	5.85E-04	2.31E-04	1.32E-08
1240.81	1.406	0.649	1.463	360	1.26E-03	9.67E-05	1.19E-08
2479.91	1.298	0.522	1.352	290	1.34E-03	4.61E-05	6.04E-09
1240.81	1.321	0.549	1.310				
311.09	1.375	0.612	1.348				
78.72	1.447	0.696	1.411				
20.41	1.513	0.773	1.480				
5.96	1.563	0.832	1.538				

Note:

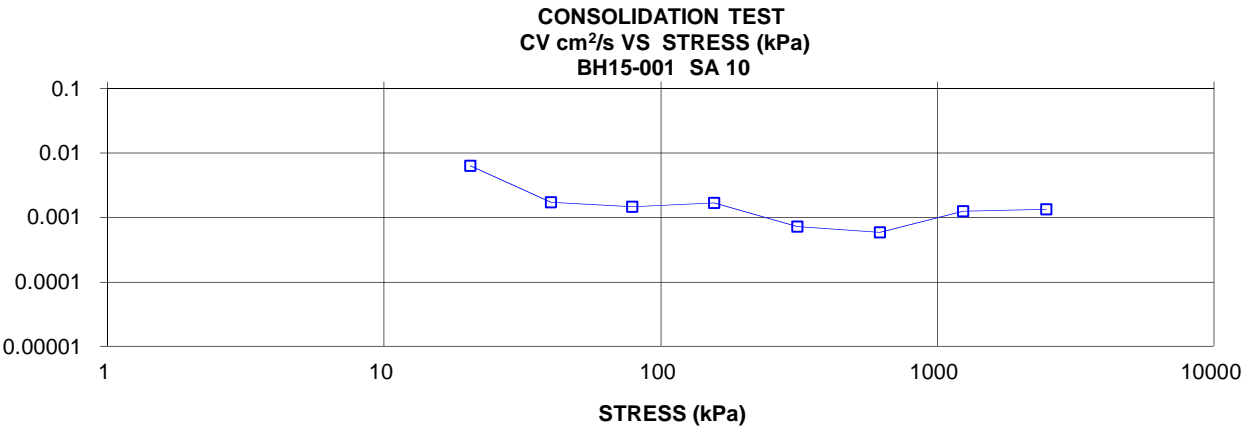
Specimen taken 4-10cm from bottom of the tube
k calculated using cv based on t₉₀ values.

Specimen swelled under 10.57 kPa.

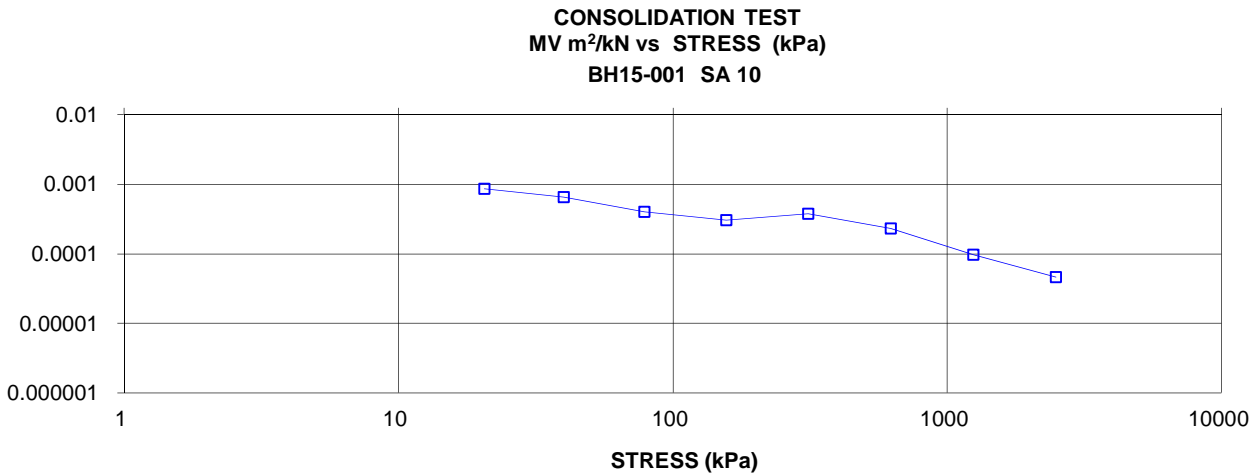
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.56	Unit Weight, kN/m ³	19.87
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	14.88
Area, cm ²	31.62	Specific Gravity, measured	2.78
Volume, cm ³	49.42	Solids Height, cm	0.853
Water Content, %	33.58	Volume of Solids, cm ³	26.97
Wet Mass, g	100.16	Volume of Voids, cm ³	22.45
Dry Mass, g	74.98		

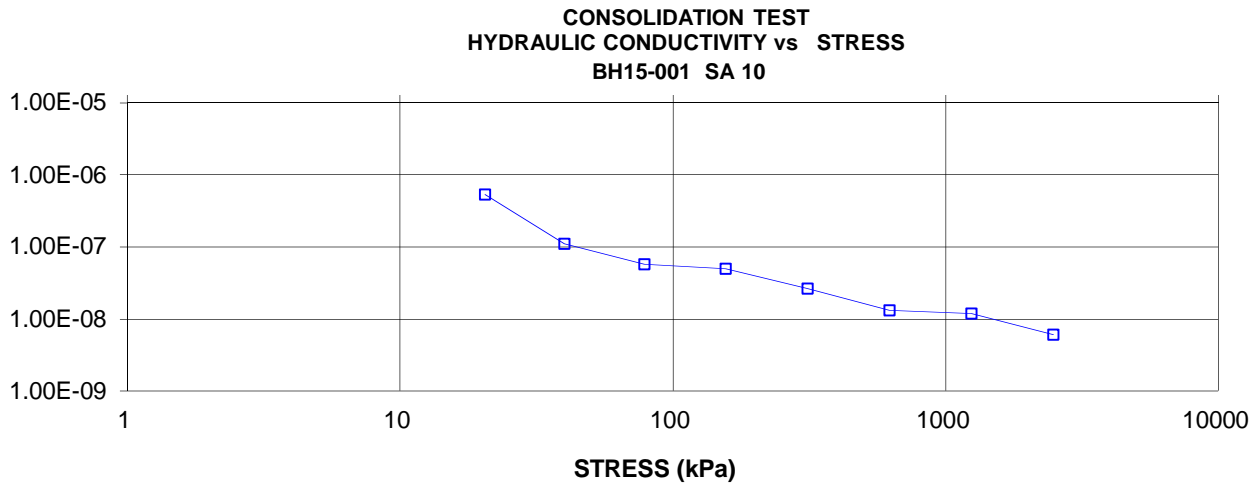
COEFFICIENT OF CONSOLIDATION,
cm²/s

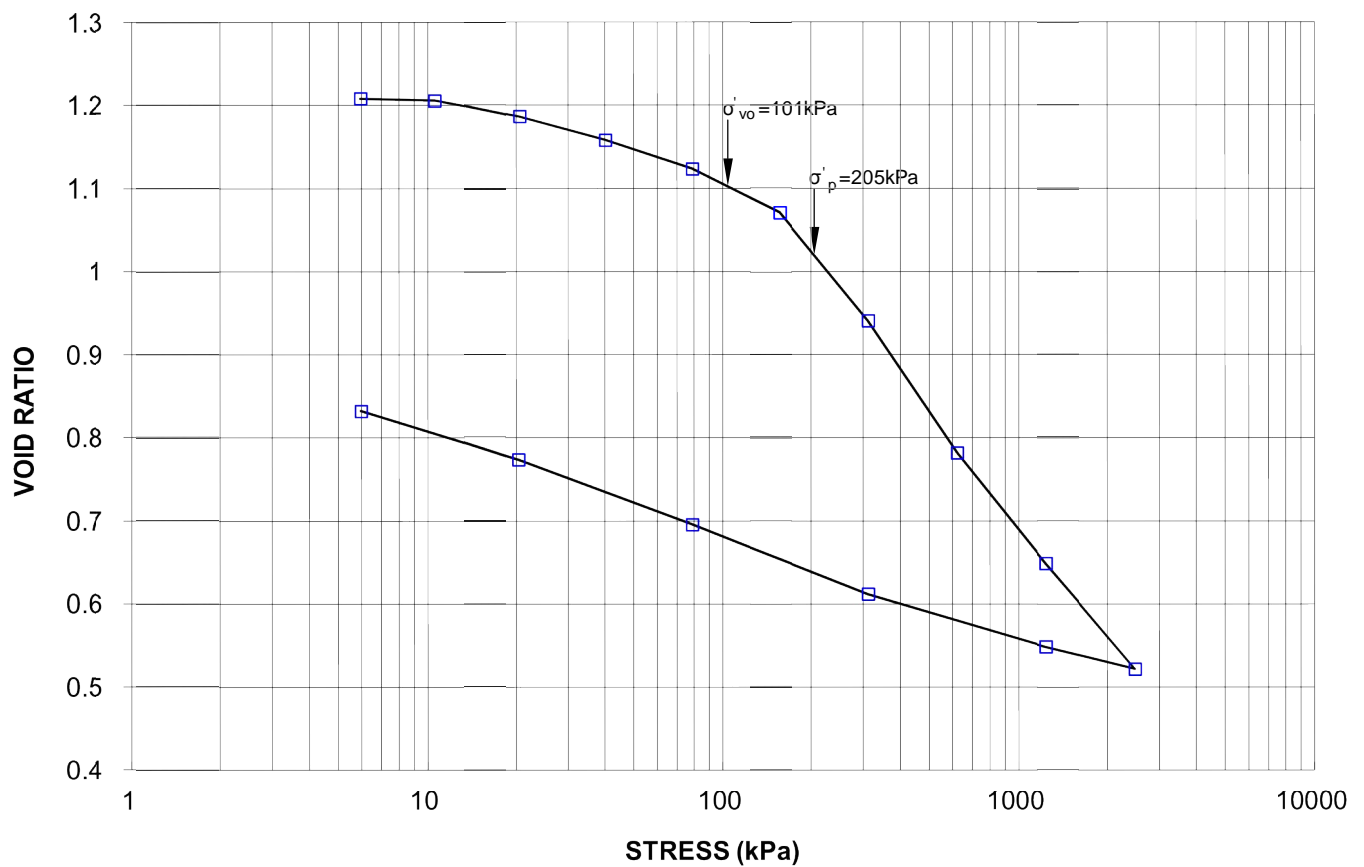


VOLUME COMPRESSIBILITY, m²/kN



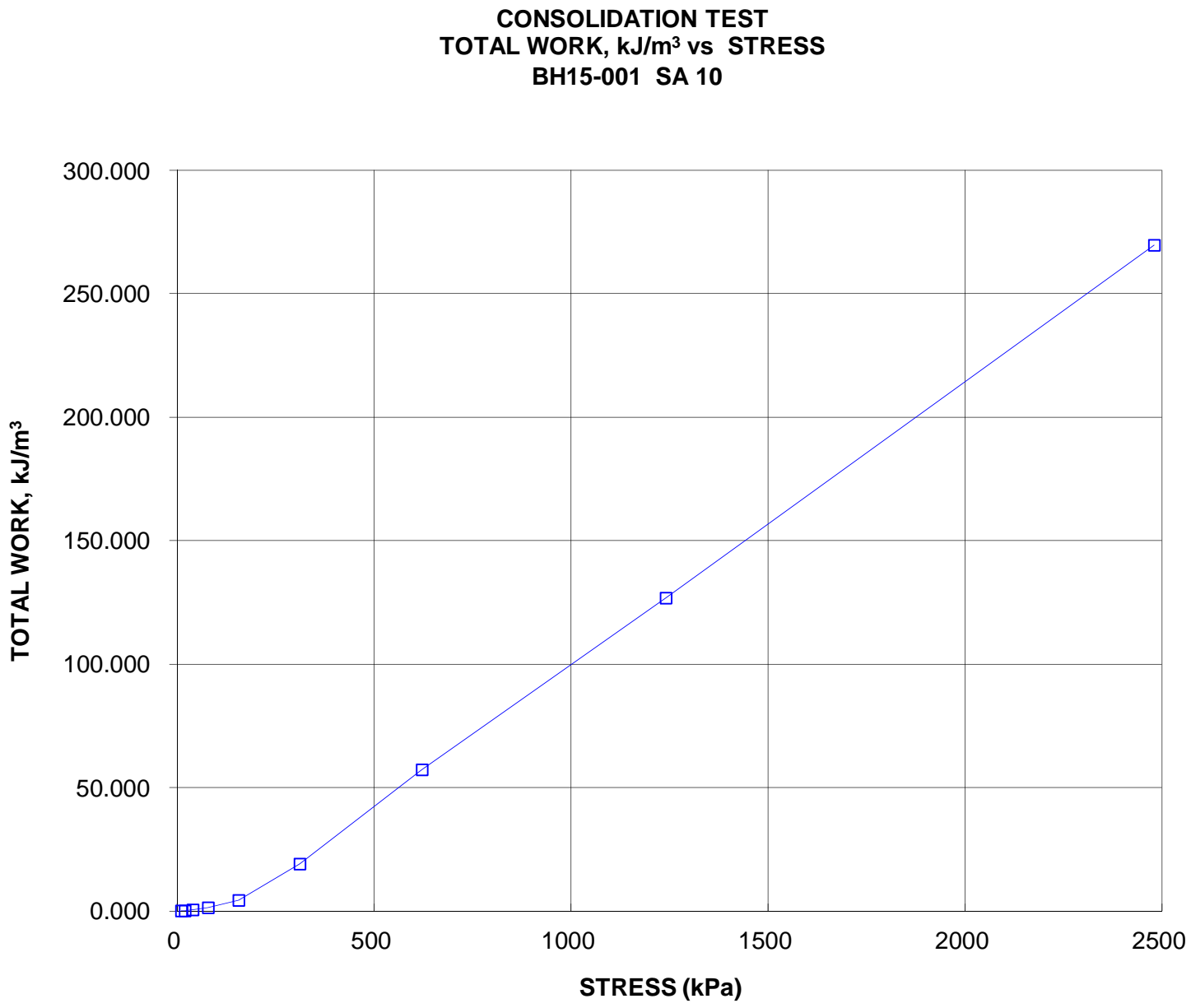
HYDRAULIC CONDUCTIVITY,
cm/s





PROJECT			
OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2 HIGHWAY 401 (RHHGP) GWP 3028-14-00			
TITLE			
CONSOLIDATION TEST VOID RATIO Vs. LOG PRESSURE			
PROJECT No.		13-1132-0053	
FILE No.		1311320053-1000-F010A5C	
CADD		DCH	May 20/16
CHECK			
SCALE		AS SHOWN	REV. 0
FIGURE A-5C			







APPENDIX B

Laboratory Test Data - Rock

FIGURE B-1A

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

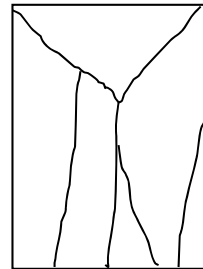
SAMPLE IDENTIFICATION			
PROJECT NUMBER	13-1132-0052	SAMPLE NUMBER	21
PROJECT NAME		SAMPLE DEPTH, m	26.62-26.74
BOREHOLE NUMBER	BH15-001	DATE:	12/16/2015

TEST CONDITIONS			
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.42	WATER CONTENT, (specimen) %	0.14
SAMPLE DIAMETER, cm	4.63	UNIT WEIGHT, kN/m ³	24.36
SAMPLE AREA, cm ²	16.82	DRY UNIT WT., kN/m ³	24.33
SAMPLE VOLUME, cm ³	175.35	SPECIFIC GRAVITY	-
WET WEIGHT, g	435.79	VOID RATIO	-
DRY WEIGHT, g	435.18		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS			
STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	31.2

REMARKS: -

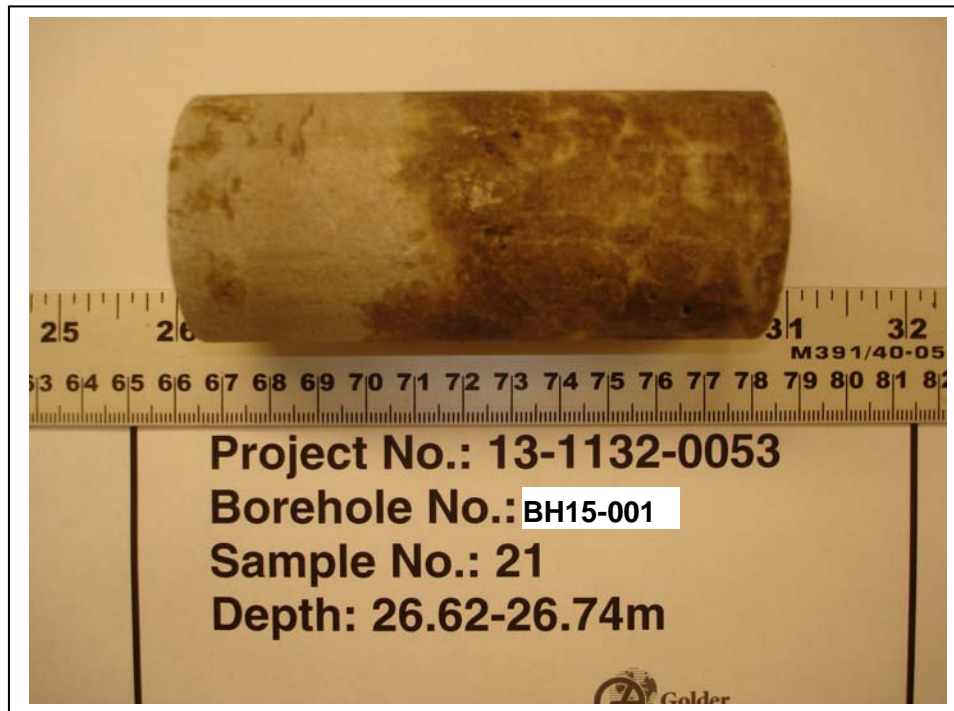
L/D Ratio not in accordance with ASTM Standard

Checked By:

Golder Associates

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

FIGURE B-1B



BEFORE COMPRESSION



AFTER COMPRESSION

Date Dec. 17, 2015
Project 13-1132-0053

Golder Associates

Drawn Frank
Chkd.

FIGURE B-2A

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

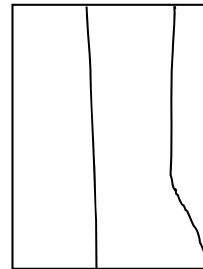
SAMPLE IDENTIFICATION			
PROJECT NUMBER	13-1132-0052	SAMPLE NUMBER	20
PROJECT NAME		SAMPLE DEPTH, m	25.02-25.15
BOREHOLE NUMBER	BH15-001	DATE:	12/16/2015

TEST CONDITIONS			
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.26

SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	10.46	WATER CONTENT, (specimen) %	0.09
SAMPLE DIAMETER, cm	4.63	UNIT WEIGHT, kN/m ³	25.21
SAMPLE AREA, cm ²	16.80	DRY UNIT WT., kN/m ³	25.19
SAMPLE VOLUME, cm ³	175.75	SPECIFIC GRAVITY	-
WET WEIGHT, g	451.95	VOID RATIO	-
DRY WEIGHT, g	451.57		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS			
STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	53.5

REMARKS: -

Checked By:

Golder Associates

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

FIGURE B-2B



BEFORE COMPRESSION



AFTER COMPRESSION

Date Dec. 17, 2015
Project 13-1132-0053

Golder Associates

Drawn Frank
Chkd.



APPENDIX C

Photograph of Rock Core



APPENDIX C

Photograph of Rock Core



n:\active\2013\1132-geo\1132-0000\13-1132-0053 aecom_herb gray parkway_3012-e-0029\ph 1000-fdns\2-correspondence\5-rpts\1311320053-1000-r01 (final) nov 18 16 app c rock core photo.docx



APPENDIX D

Site Photographs



APPENDIX D

Site Photograph



Photograph 1: Bridge Site B-1 - Construction of east approach embankment.



Photograph 2: Plaza construction area.



APPENDIX E

Record of Previous Boreholes and Laboratory Testing Geocres No. 40J6-27

RECORD OF BOREHOLE No CPT-165

1 OF 1

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4682188.2 :E 328457.7

ORIGINATED BY CC

DIST WEST HWY 401/3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY BRS

DATUM GEODETIC

DATE

August 13, 2008

CHECKED BY *SJS*

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			20 40 60 80 100										SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		
178.98	GROUND SURFACE																			
0.00	FILL, crushed gravel Grey																			
0.30	TOPSOIL, sandy																			
0.46	Black		1	SS	8															
177.91	FILL, silty sand topsoil with silty sand layers, pockets of gravel and wood						178													
1.07	Loose		2	SS	7															
177.30	Black																			
1.68	SILTY SAND AND GRAVEL						177													
176.69	Loose		3	SS	20															
2.29	Brown																			
	SAND, trace gravel																			
	Compact																			
	Grey																			
	END OF BOREHOLE																			
	Water level in borehole at about elev. 177.31m during drilling on August 13, 2008.																			

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-165

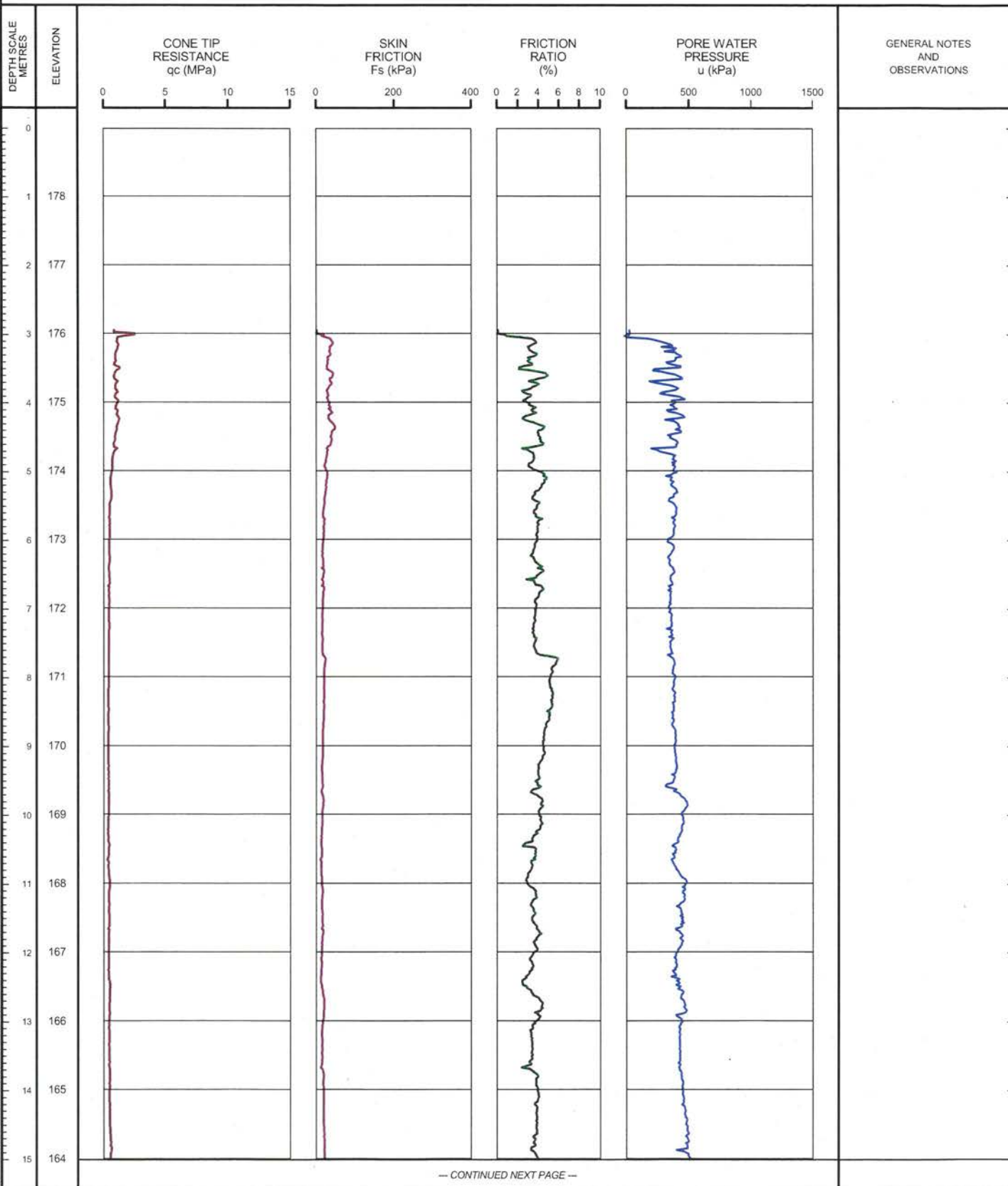
SHEET 1 OF 2

LOCATION: N 4682188.2 E 328457.7

TEST DATE: August 13, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.95m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LDN CPT 01 07-1130-207-0-CPT.GPJ GLDR LON.GDT 6/18/09 DATA INPUT:

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SSS

PROJECT: 07-1130-207-0

RECORD OF CONE PENETRATION TEST CPT-165

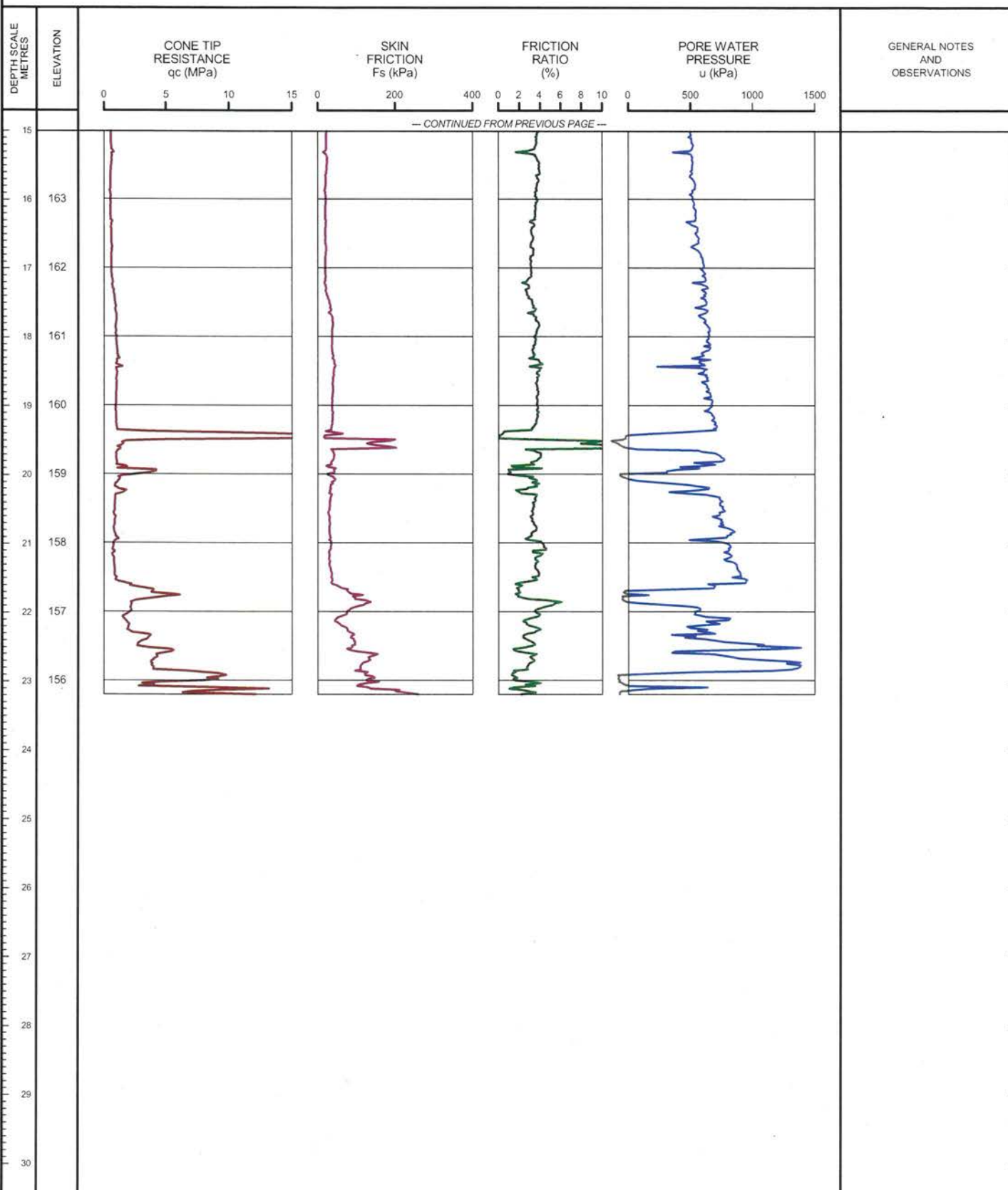
SHEET 2 OF 2

LOCATION: N 4682188.2 ; E 328457.7

TEST DATE: August 13, 2008

DATUM: GEODETIC

GROUND SURFACE ELEVATION: PREDRILL DEPTH: 2.95m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LDN CPT 01 07-1130-207-0-CPT.GPJ GLDR LON.GDT 6/18/09 DATA INPUT:

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SJB

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4682168.3 :E 328349.6

ORIGINATED BY CC

DIST WEST HWY 401/3

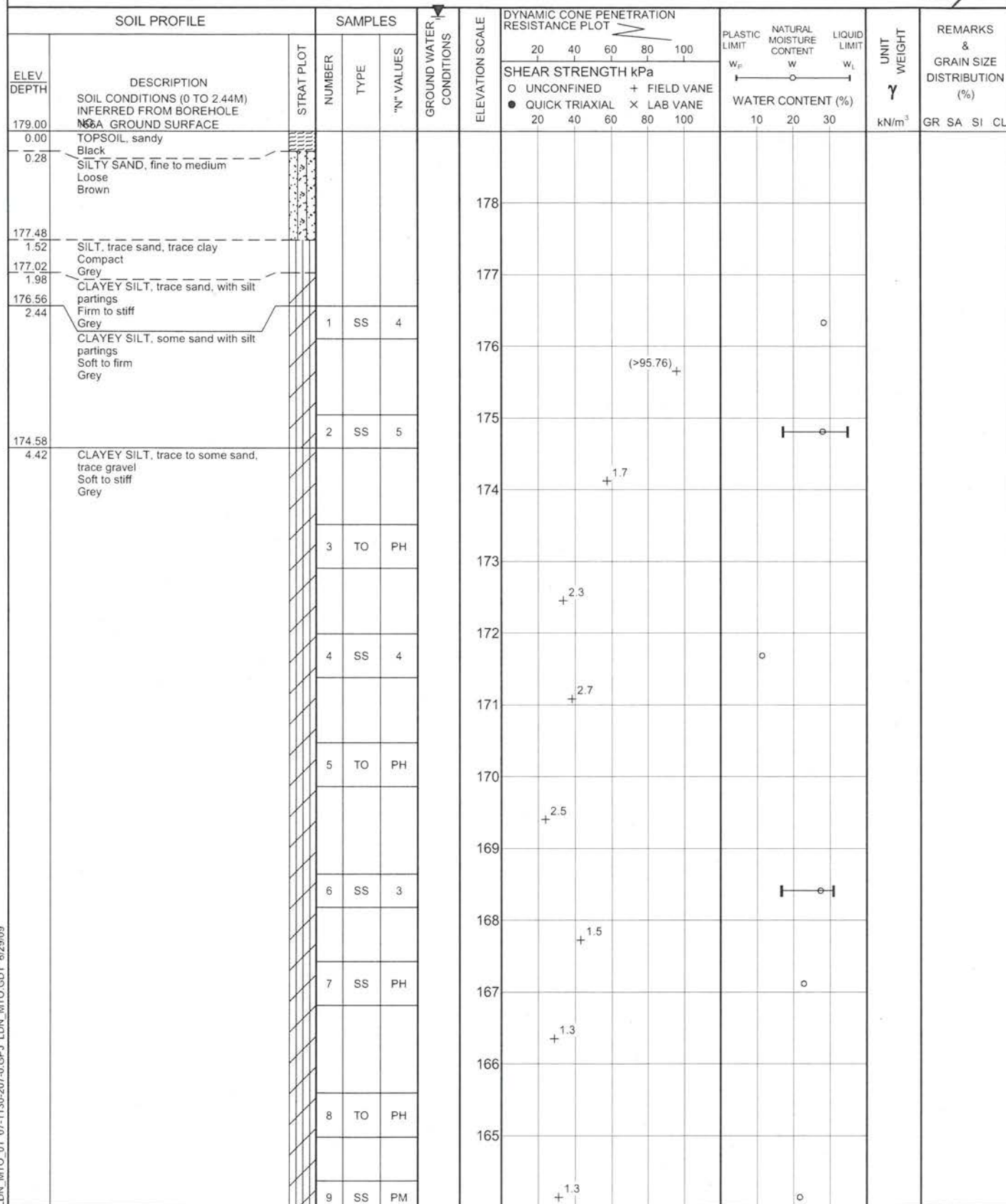
BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY LMK

DATUM GEODETIC

DATE September 11, 2008 - September 17, 2008

CHECKED BY SIB



DN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/29/09

Continued Next Page

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

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4682168.3 :E 328349.6

ORIGINATED BY CC

DIST WEST HWY 401/3

BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY LMK

DATUM GEODETIC

DATE September 11, 2008 - September 17, 2008

CHECKED BY **SJR**

[illegible]

DN_MTO_01 07-1130-207-0.GPJ LDN_MTO.GDT 6/30/09

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

PROJECT: 07-1130-207-0

RECORD OF DRILLHOLE: 166

SHEET 3 OF 3

LOCATION: N 4682168.3 ; E 328349.6

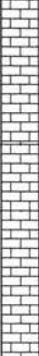
DRILLING DATE: September 11, 2008 - September 17, 2008

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: --

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: AARDVARK DRILLING INC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	ELEVATION	JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular PO- Polished K - Slickensided SM- Smooth Ro - Rough Br - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)					RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION			
									TOTAL CORE %	SOLID CORE %										
																	80 60 40 20	80 60 40 20		
		ROCK SURFACE		155.73																
	MUD ROTARY NO ROCK CORE	LIMESTONE, fresh, medium strong, weakly laminated, fine grained, porous to pitted with occasional vugs, fossiliferous, hydrocarbon staining, mottled brown and grey		23.27				155							JN,PL,Ro CI					
24																				
		LIMESTONE, fresh, medium strong, weakly laminated, fine grained, faintly porous, hydrocarbon staining, brown, mottled brown and grey zone at 25.3m		154.29					154											
25					24.71															
		LIMESTONE, fresh, medium strong, thinly laminated, very fine grained to fine grained, faintly porous, stylolitic, occasional fossils, grey with light grey inclusions		153.63				153							JN,PL,Ro CI					
26				25.37											JN,UN,Ro CI					
27		END OF DRILLHOLE		152.08																
				26.92																
28																				
29																				
30																				
31																				
32																				
33																				
34																				
35																				
36																				
37																				
38																				

DEPTH SCALE

1 : 75



LOGGED: SG

CHECKED: SJB

RECORD OF BOREHOLE No 166A

1 OF 2

METRIC

PROJECT 07-1130-207-0

W.P.

LOCATION

N 4682168.3 :E 328349.6

ORIGINATED BY CC

DIST

WEST

HWY 401/3

BOREHOLE TYPE

POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE

September 17, 2008

CHECKED BY *SB*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				
179.00	SOIL CONDITIONS (2.75 TO 15.39M) INFERRED FROM BOREHOLE NO. 166											
0.00	TOPSOIL, sandy											
0.28	Black SILTY SAND, fine to medium Loose Brown		1	SS	9							
177.48												
1.52	SILT, trace sand, trace clay Compact Grey		2	SS	16							0 3 89 8
177.02												
1.98	CLAYEY SILT, trace sand, with silt partings Firm to stiff Grey		3	SS	8							
176.25												
2.75	CLAYEY SILT, some sand with silt partings Soft to firm Grey											

RECORD OF BOREHOLE No 166A

2 OF 2

METRIC

PROJECT 07-1130-207-0

W.P. LOCATION N 4682168.3 :E 328349.6

ORIGINATED BY CC

DIST WEST HWY 401/3 BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC DATE September 17, 2008

CHECKED BY *SB*

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			20 40 60 80 100	10 20 30					
163.61 15.39	END OF BOREHOLE Water level measured in shallow piezometer at elev. 163.76m on September 19, 2008. Water level measured in shallow piezometer at elev. 165.19m on September 22, 2008. Water level measured in shallow piezometer at elev. 178.43m on January 28, 2009.						Piezometer							

RECORD OF BOREHOLE No CPT-348

1 OF 1

METRIC

PROJECT 09-1132-0080-7000

W.P.

LOCATION

N 4682160.4 :E 328512.5

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, SOLID STEM

COMPILED BY AG

DATUM GEODETTIC

DATE April 27, 2010

CHECKED BY SJB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
179.15	GROUND SURFACE					▽	179								
0.00	TOPSOIL, sandy Dark brown						178								
0.27	SAND, fine to medium Loose to compact Brown		1	SS	14										
177.32			2	SS	9										
1.83	CLAYEY SILT, some sand, trace gravel Stiff Grey						177								
176.25			3	SS	8										
2.90	END OF BOREHOLE														
	Groundwater encountered at about elev. 177.8m during drilling on April 27, 2010.														

LDN_MTO_01 09-1132-0080-7000.GPJ LDN_MTO.GDT 04/06/10

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-348

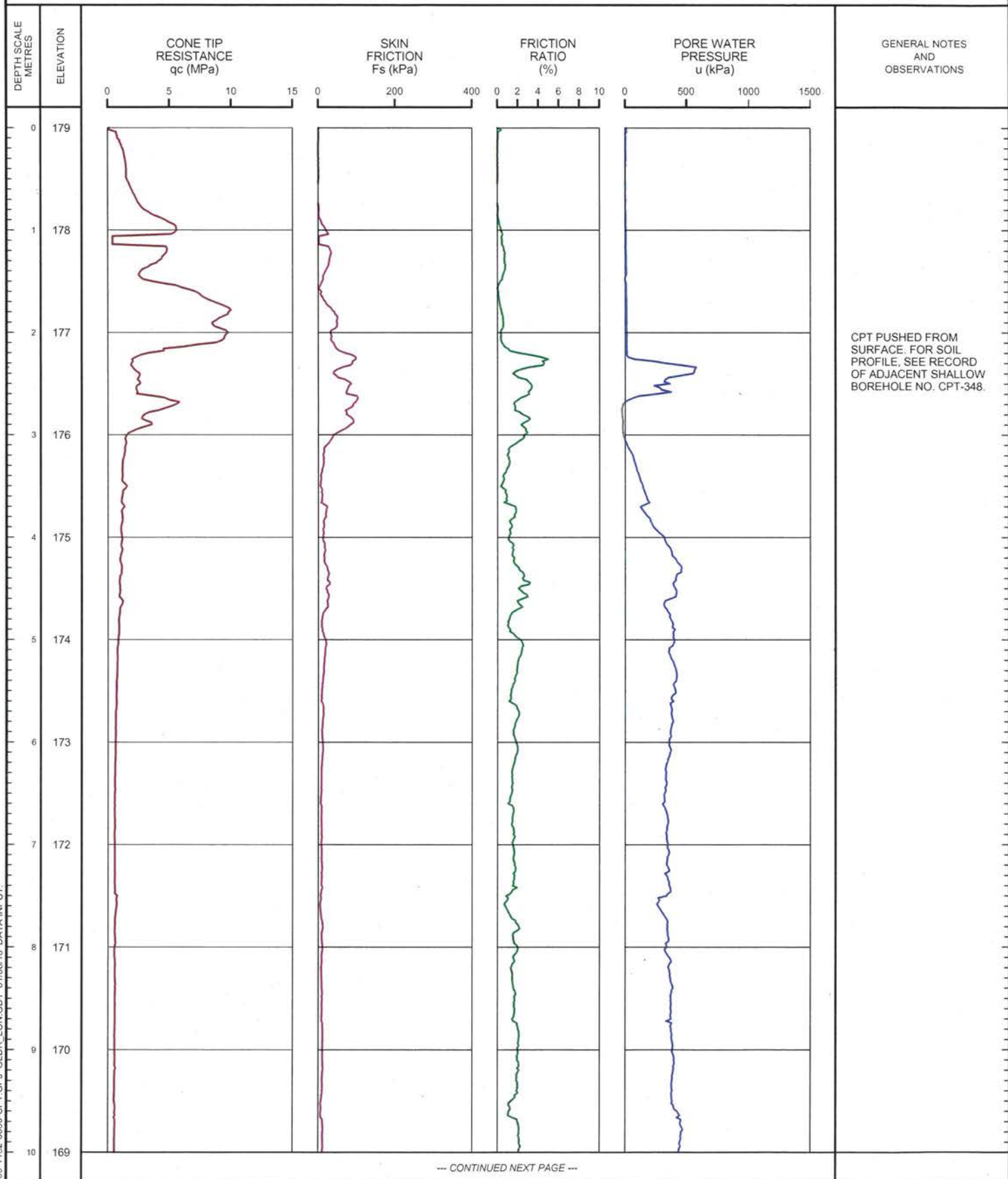
SHEET 1 OF 3

LOCATION: N 4682160.4 ; E 328512.5

TEST DATE: April 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.15m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



CPT PUSHED FROM
SURFACE. FOR SOIL
PROFILE, SEE RECORD
OF ADJACENT SHALLOW
BOREHOLE NO. CPT-348.

LDN CPT-01 09-1132-0080-CPT.GPJ GLDR_LON.GDT 01/06/10 DATA INPUT:

DEPTH SCALE

1:50



OPERATOR: TA

CHECKED: *503*

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-348

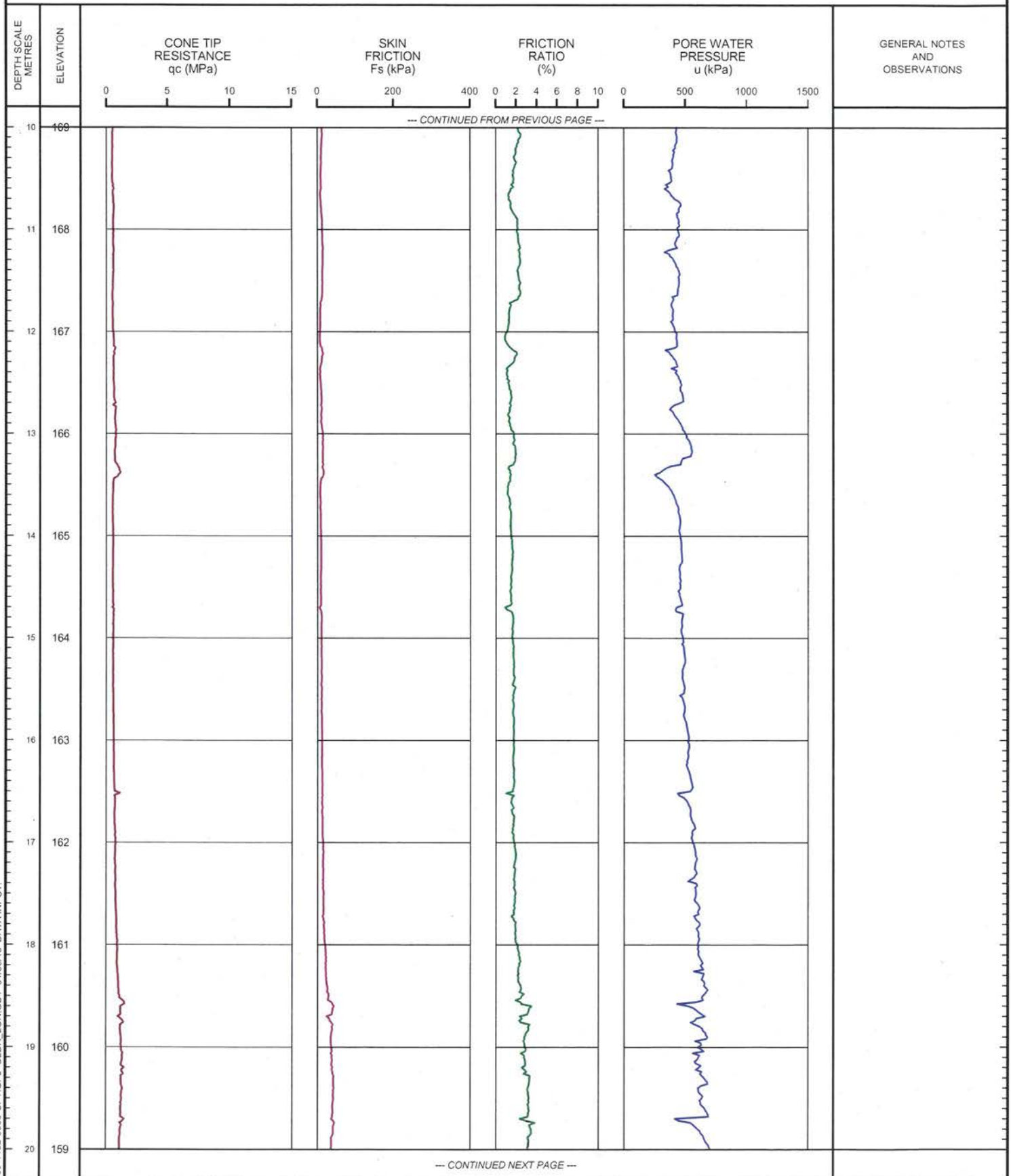
SHEET 2 OF 3

LOCATION: N 4682160.4 ; E 328512.5

TEST DATE: April 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.15m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LDN CPT 01 09-1132-0080-CPT GPJ GLDR LON GDT 01/06/10 DATA INPUT:

DEPTH SCALE

1:50



OPERATOR: TA

CHECKED: SJB

PROJECT: 09-1132-0080

RECORD OF CONE PENETRATION TEST CPT-348

SHEET 3 OF 3

LOCATION: N 4682160.4 ; E 328512.5

TEST DATE: April 26, 2010

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 179.15m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LDN_CPT_01_09-1132-0080-CPT.GPJ GLDR LON GDT 01/06/10 DATA INPUT:

DEPTH SCALE

1 : 50

Golder
Associates

OPERATOR: TA

CHECKED: *SJS*

METRIC

PROJECT 09-1132-0080-7000

W.P.

LOCATION

N 4682135.5 :E 328496.2

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY AG

DATUM GEODETIC

DATE April 22, 2010 - April 23, 2010

CHECKED BY CJK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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		ELEVATION SCALE	SHEAR STRENGTH kPa						
								○ UNCONFINED						+ FIELD VANE
								● QUICK TRIAXIAL						× LAB VANE
								WATER CONTENT (%)						
								20 40 60 80 100	10 20 30					

179.08	GROUND SURFACE						180						
0.00	TOPSOIL, sandy to clayey Dark brown						WL - VWP4						
178.17							179						
0.91	SAND, fine to medium, trace to some silt Loose to compact Brown		1	SS	7		WL - VWP3 WL - VWP2						
176.95			2	SS	19		178						
2.13	CLAYEY SILT, trace sand, with silt partings Soft to stiff Grey		3	SS	11		177						1 94 (5)
			4	SS	6		176						
			5	SS	7		175						0 3 74 23
			6	SS	3		174						
173.59			7	TO	PH		173						
5.49	SILTY CLAY, trace sand Soft to firm Grey		8	SS	3		172						0 7 28 65 Oedometer
			9	TO	PH		171						
			10	SS	2		170						
169.63			11	TO	PH		169						
9.45	CLAYEY SILT, trace sand Soft to stiff Grey		12	SS	5		168						2 12 52 34 Oedometer
			13	TO	PH		167						
							166						

Continued Next Page

 $+^3 \times 3$

+ 3, × 3: Numbers refer to Sensitivity

○ 3%

○ 3% STRAIN AT FAILURE

METRIC

PROJECT 09-1132-0080-7000

W.P.

LOCATION

N 4682135.5 ;E 328496.2

ORIGINATED BY TA

DIST WEST HWY 401 / 3

BOREHOLE TYPE POWER AUGER, MUD ROTARY WITH HQ TRICONE, NQRC

COMPILED BY AG

DATUM GEODETIC

DATE April 22, 2010 - April 23, 2010

CHECKED BY SJB

[illegible]

DN_MTO_06 09-1132-0080-7000.GPJ LDN_MTO.GDT 04/06/10

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

PROJECT: 09-1132-0080

RECORD OF DRILLHOLE: 349

SHEET 3 OF 3

LOCATION: N 4682135.5 ;E 328496.2

DRILLING DATE: April 22, 2010 - April 23, 2010

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: MOBILE B-57

DRILLING CONTRACTOR: LANTECH DRILLING SERVICES INC.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (mm/min)	COLOUR % RETURN	FLUSH	ELEVATION	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough Br - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.										HYDRAULIC CONDUCTIVITY k, cm/sec				DIAMETRAL POINT LOAD INDEX (MPa)				NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
				DEPTH (m)	RECOVERY						SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION					2	4	6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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LDN ROCK 03 09-1132-0080-7000-ROCK.GPJ GLDR LDN.GDT 01/06/10 DATA INPUT: AG

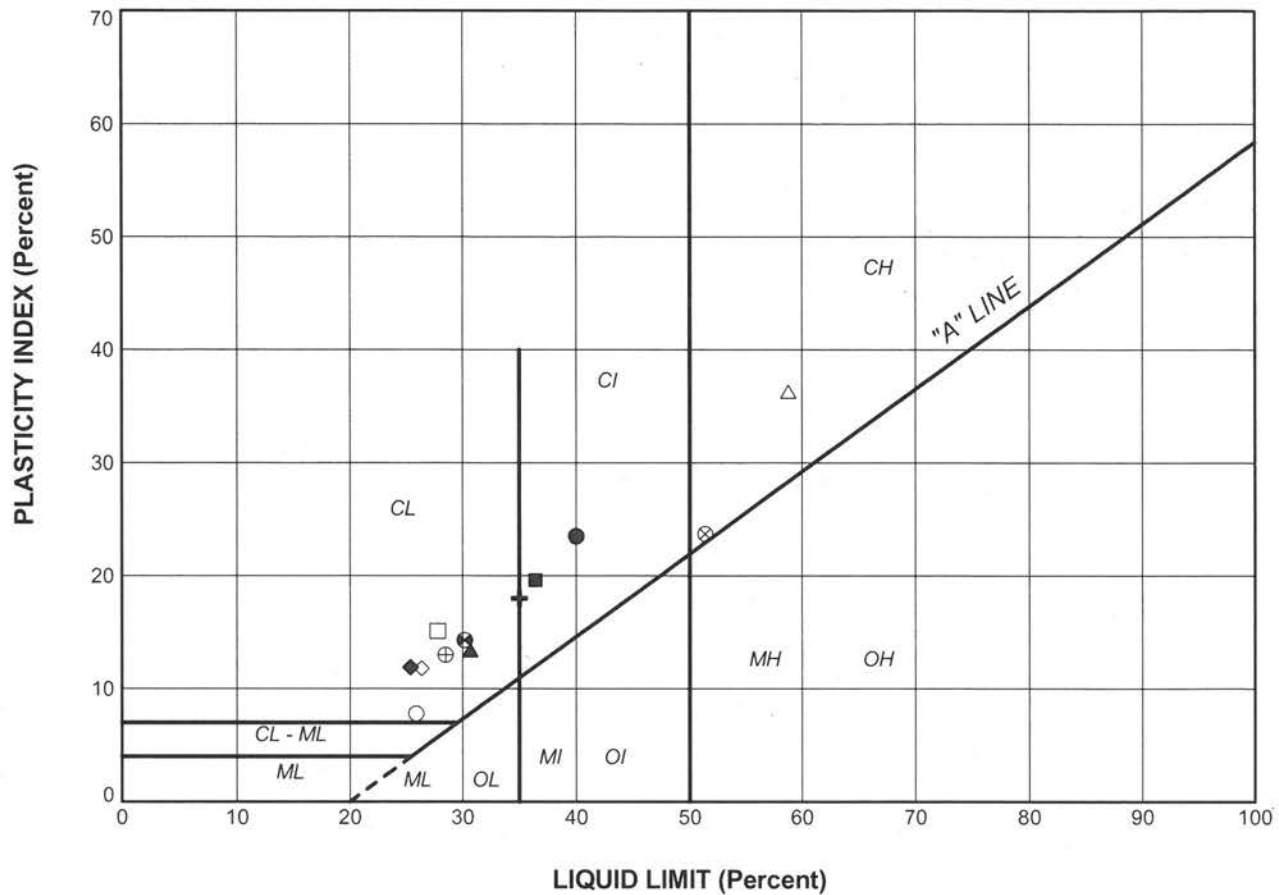
DEPTH SCALE

1:75



LOGGED: TA

CHECKED: 518

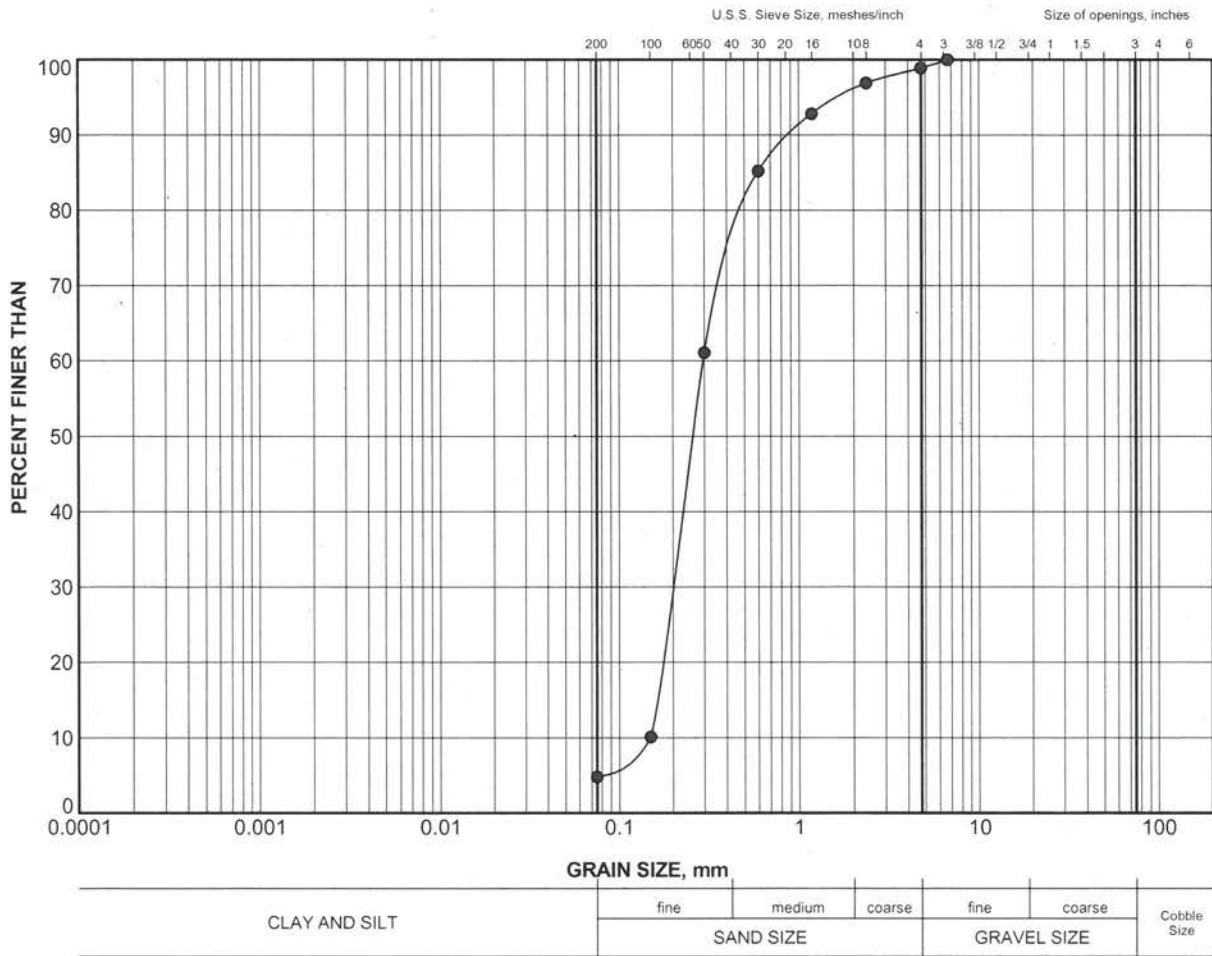


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	346	5	40.0	16.5	23.5
■	346	9	36.4	16.8	19.6
▲	346	10	30.7	17.3	13.4
⊕	346	11	35.0	17.0	18.0
◆	346	13	25.4	13.5	11.9
◇	346	18	26.4	14.6	11.8
○	349	4	25.9	18.1	7.8
△	349	7	58.8	22.5	36.3
⊗	349	10	51.4	27.7	23.7
⊕	349	11	28.5	15.5	13.0
□	349	14	27.8	12.7	15.1
⊙	349	17	30.2	15.9	14.3

PROJECT			
GEOTECHNICAL DATA REPORT - ADDENDUM NO. 7			
WINDSOR-ESSEX PARKWAY			
WINDSOR, ONTARIO			
TITLE			
PLASTICITY CHART			
PROJECT No. 09-1132-0080		FILE No 0911320080-7000-R020D1	
DRAWN AG	June 8/10	SCALE N/A	REV.
CHECK 5215		FIGURE D-1	

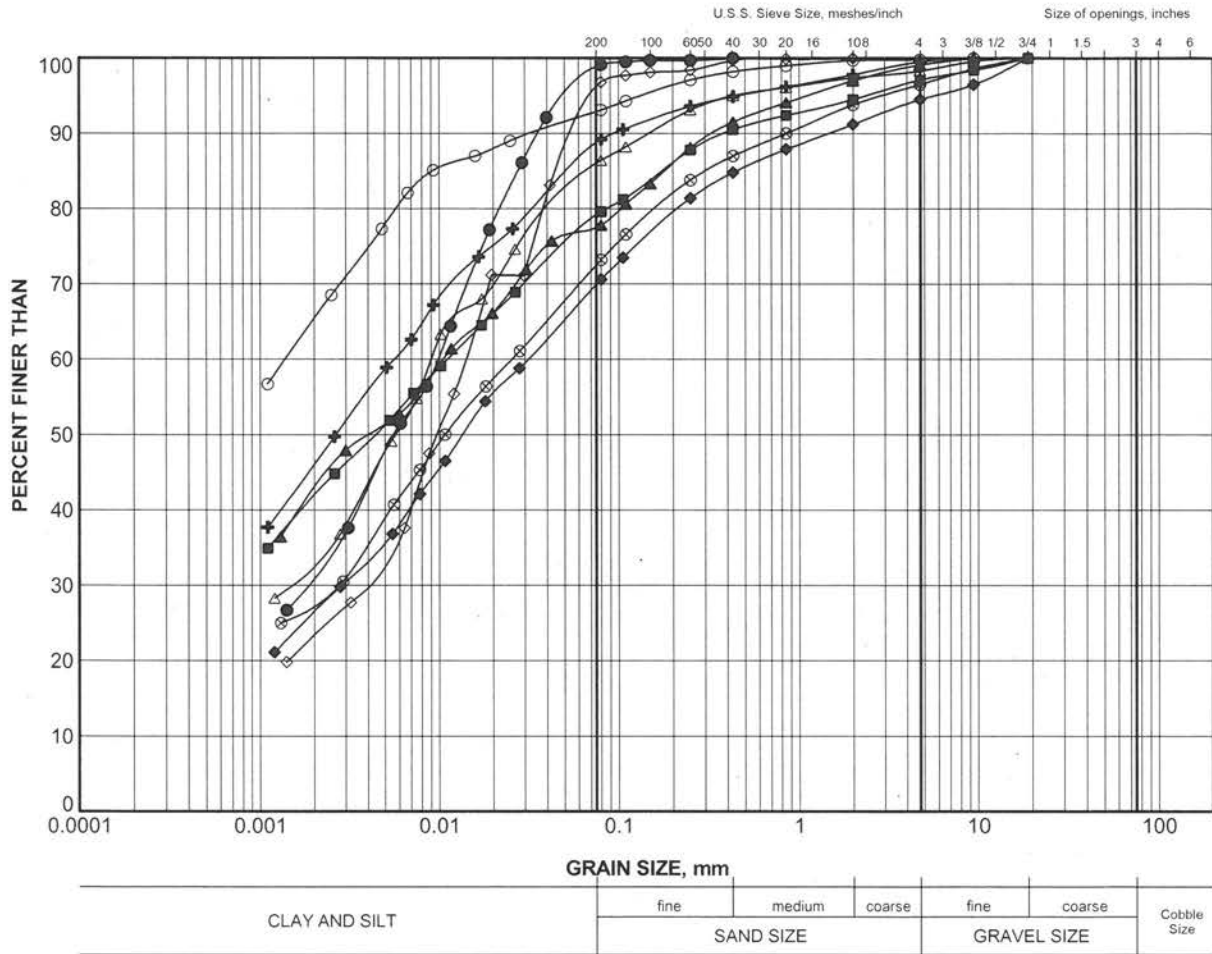




LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	349	2	177.6

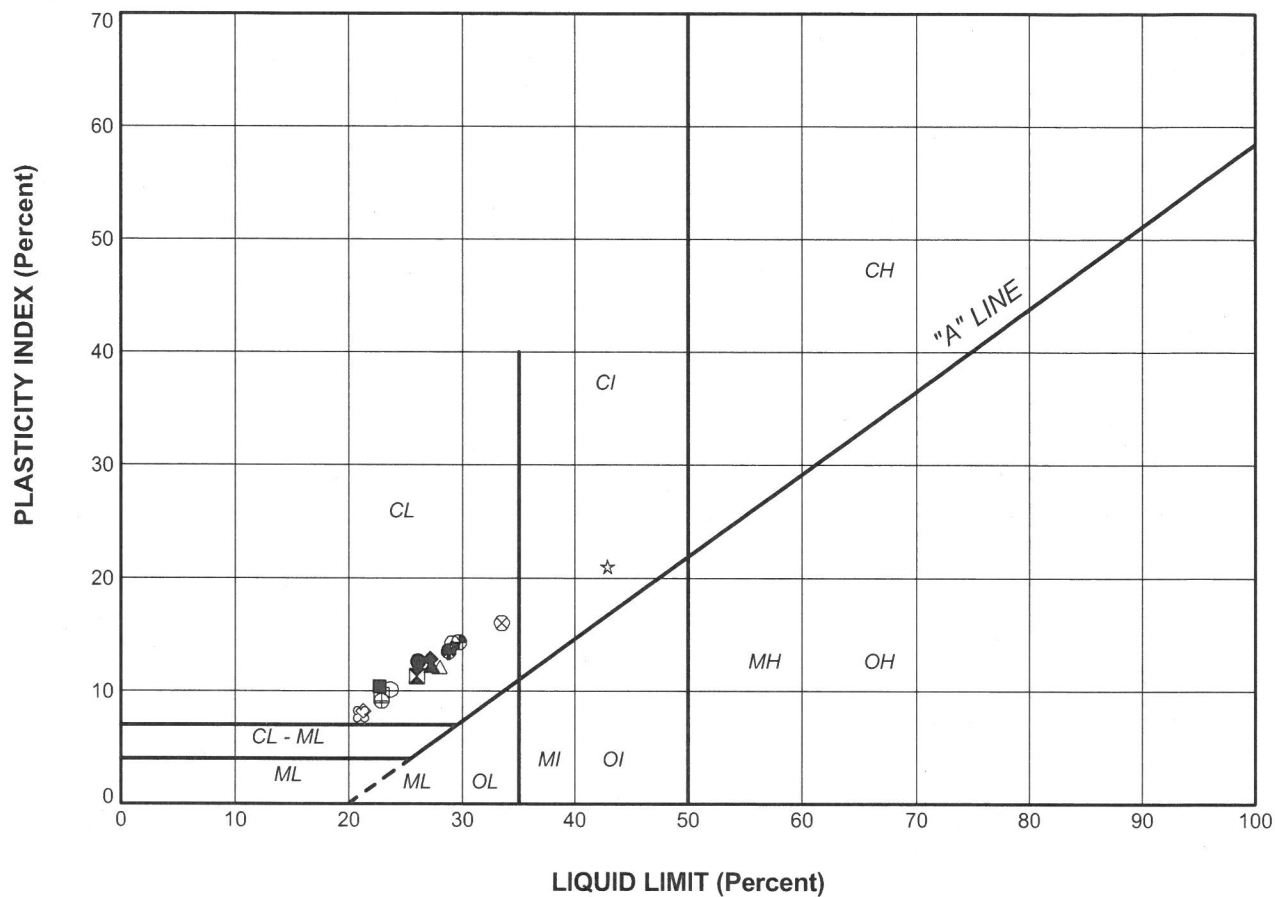
PROJECT			
GEOTECHNICAL DATA REPORT - ADDENDUM NO. 7 WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE			
GRAIN SIZE DISTRIBUTION UPPER GRANULAR DEPOSITS			
PROJECT No: 09-1132-0080		FILE No 0911320080-7000-R020D2	
DRAWN AG		June 8/10	
CHECK		5/5/2010	
Golder Associates LONDON, ONTARIO		SCALE N/A REV.	



LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	346	5	175.4
■	346	9	171.9
▲	346	10	170.4
✦	346	11	168.9
◆	346	13	165.8
◇	349	4	176.0
○	349	7	173.6
△	349	11	168.7
⊗	349	14	164.1

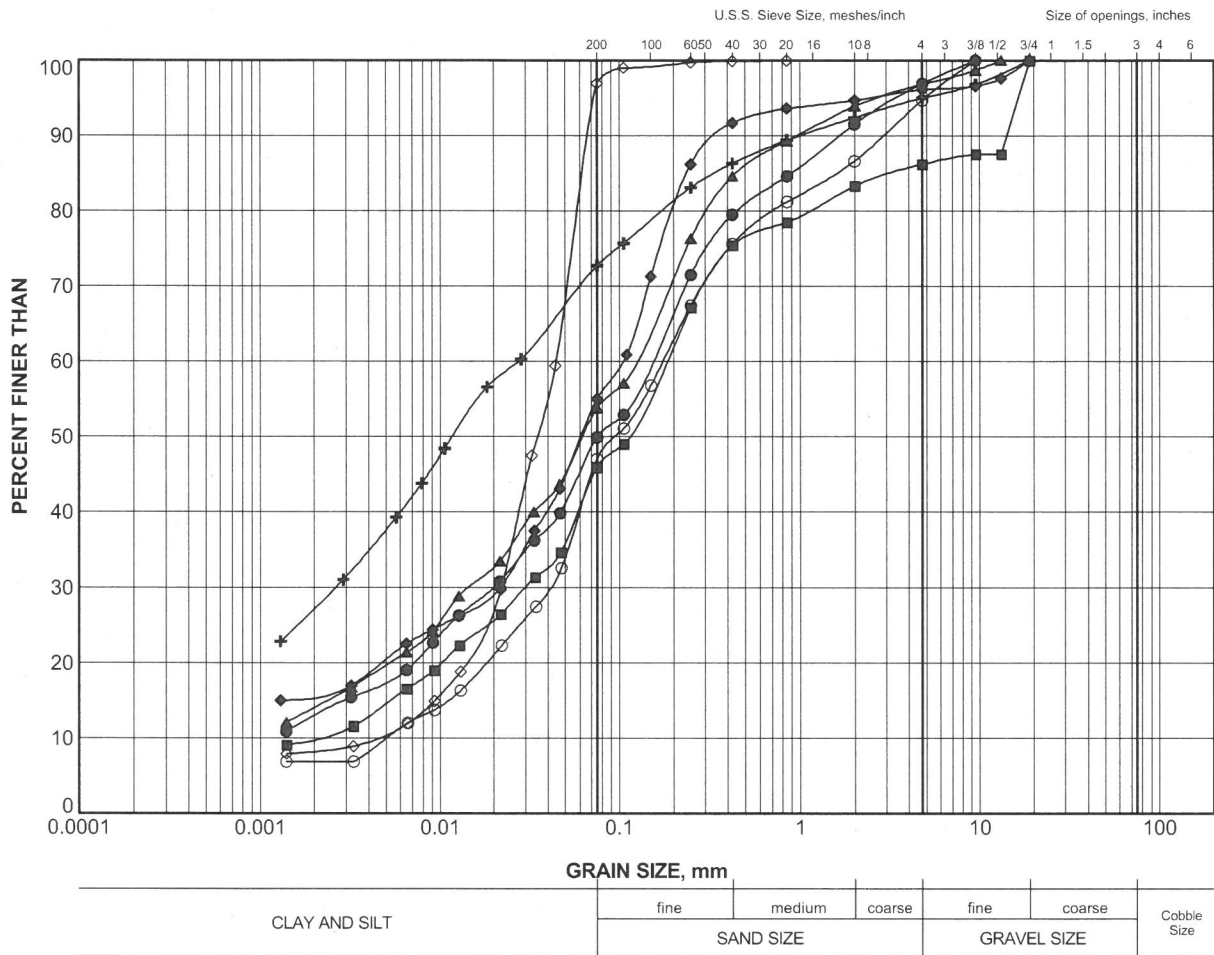
PROJECT			
GEOTECHNICAL DATA REPORT - ADDENDUM NO. 7 WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE			
GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY DEPOSIT			
PROJECT No.		09-1132-0080	
FILE No.		0911320080-7000-R020D3	
SCALE		N/A	
REV.			
DRAWN	AG	June 8/10	
CHECK	SR	June 8/10	
Golder Associates LONDON, ONTARIO		FIGURE D-3	



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	1	5	26.1	13.5	12.6
■	1	7	22.7	12.3	10.4
▲	1	10	27.3	15.1	12.2
+	1	12	28.8	15.4	13.4
◆	1	14	27.2	14.4	12.8
◇	1	16	21.3	13.1	8.2
○	1	18	23.7	13.6	10.1
△	1	20	28.0	15.9	12.1
⊗	1	23	33.5	17.5	16.0
⊕	7	5	22.9	13.8	9.1
□	7	7	22.9	13.3	9.6
⊗	7	9	28.8	15.3	13.5
⊕	7	11	29.7	15.4	14.3
☆	7	12	42.9	21.9	21.0
⊗	7	15	21.1	13.2	7.9
⊕	7	17	26.0	14.7	11.3
⊗	7	20	29.1	14.9	14.2

PROJECT				GEOTECHNICAL DATA REPORT WINDSOR-ESSEX PARKWAY WINDSOR, ONTARIO			
TITLE							
PLASTICITY CHART							
PROJECT No.		07-1130-207-0		FILE No.		0711302070-R0100F1	
DRAWN		WDF		SCALE		N/A	
CHECK		SSB		REV.			
		May 11/09					
 Golder Associates LONDON, ONTARIO				FIGURE F.1A			



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	104	10	177.5
■	107	13a	172.1
▲	115	10	173.8
+	135	17	162.0
◆	203	10	178.8
◇	166A	2	177.3
○	230N	6	182.5

PROJECT

GEOTECHNICAL DATA REPORT
WINDSOR-ESSEX PARKWAY
WINDSOR, ONTARIO

TITLE

GRAIN SIZE DISTRIBUTION UPPER GRANULAR DEPOSITS



PROJECT No.	07-1130-207-0	FILE No.	0711302070-R0100F2
DRAWN	WDF	May 11/09	SCALE N/A REV.
CHECK	WDF	WDF	FIGURE F.2

CONSOLIDATION TEST SUMMARY

FIGURE BH 349 SA 7 OED A

SAMPLE IDENTIFICATION

Project Number	09-1132-0080	Sample Number	7
Borehole Number	349	Sample Depth, m	5.5-5.9

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	5/4/2010		
Date Completed	5/23/2010		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	17.02
Sample Diameter, cm	6.31	Dry Unit Weight, kN/m ³	11.21
Area, cm ²	31.27	Specific Gravity, measured	2.74
Volume, cm ³	79.43	Solids Height, cm	1.060
Water Content, %	51.77	Volume of Solids, cm ³	33.14
Wet Mass, g	137.82	Volume of Voids, cm ³	46.29
Dry Mass, g	90.81	Degree of Saturation, %	101.6

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	c _v cm ² /s	mv m ² /kN	k cm/s
0.00	2.540	1.397	2.540				
4.89	2.539	1.396	2.539	1	1.37E+00	8.86E-05	1.19E-05
9.57	2.538	1.395	2.538	43	3.18E-02	7.57E-05	2.36E-07
19.52	2.534	1.391	2.536	163	8.36E-03	1.62E-04	1.33E-07
39.17	2.504	1.363	2.519	205	6.56E-03	6.01E-04	3.87E-07
19.50	2.513	1.371	2.508				
9.57	2.519	1.377	2.516				
4.91	2.521	1.379	2.520				
9.57	2.520	1.378	2.521	216	6.24E-03	5.07E-05	3.10E-08
19.74	2.516	1.374	2.518	368	3.65E-03	1.74E-04	6.24E-08
39.38	2.504	1.363	2.510	305	4.38E-03	2.41E-04	1.03E-07
78.38	2.478	1.338	2.491	645	2.04E-03	2.63E-04	5.27E-08
156.54	2.423	1.286	2.450	693	1.84E-03	2.78E-04	5.00E-08
313.17	2.283	1.154	2.353	1135	1.03E-03	3.52E-04	3.56E-08
626.50	2.087	0.969	2.185	1984	5.10E-04	2.46E-04	1.23E-08
1252.53	1.916	0.808	2.001	1185	7.17E-04	1.07E-04	7.55E-09
2506.45	1.759	0.660	1.837	1070	6.69E-04	4.92E-05	3.23E-09
1252.53	1.787	0.686	1.773				
313.17	1.880	0.773	1.833				
78.38	1.993	0.880	1.936				
19.74	2.105	0.986	2.049				
4.89	2.182	1.059	2.144				

Note:

k calculated using cv based on t₉₀ values.

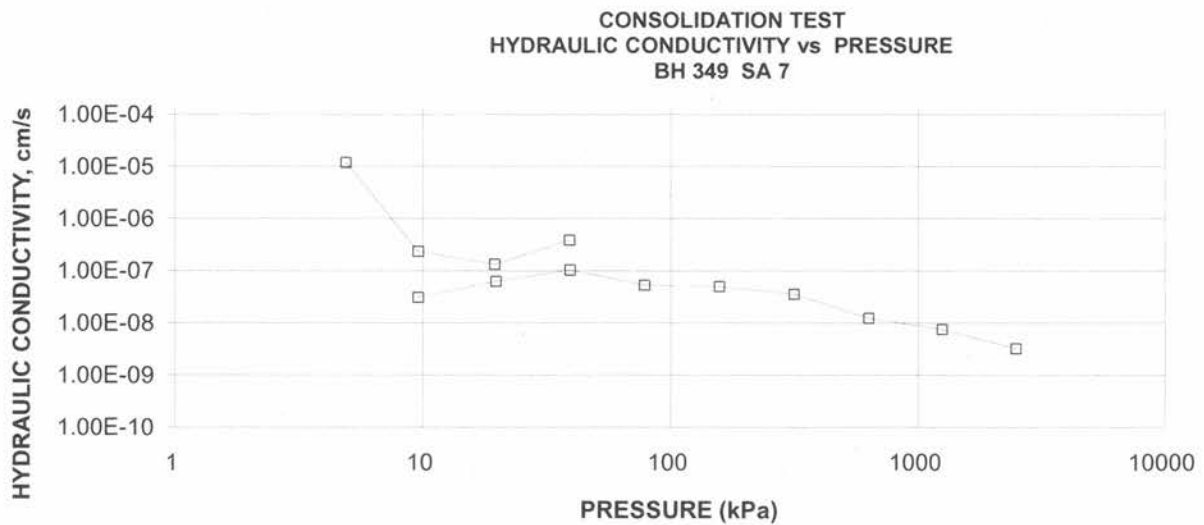
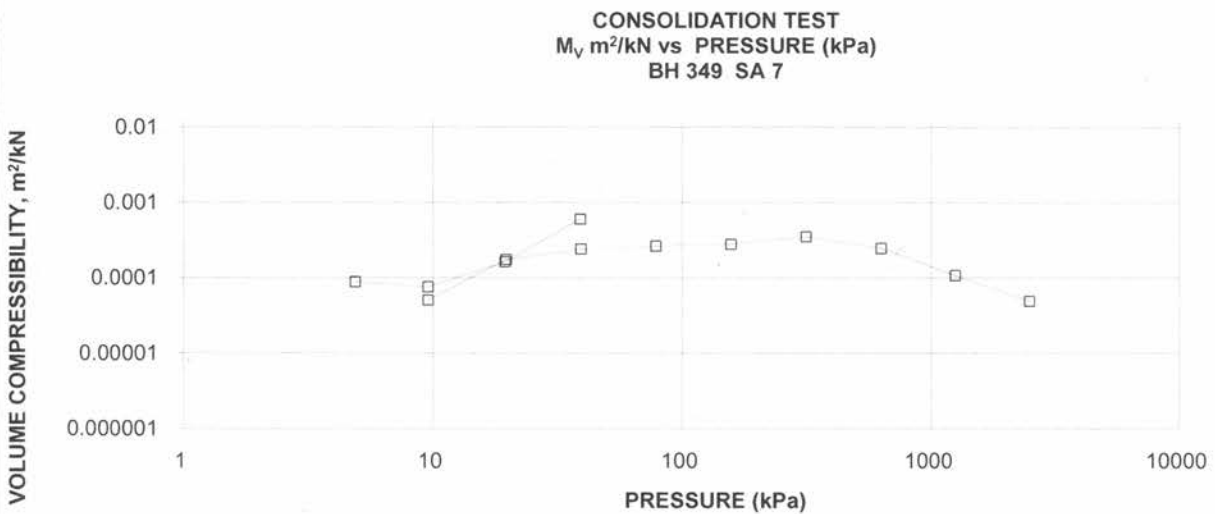
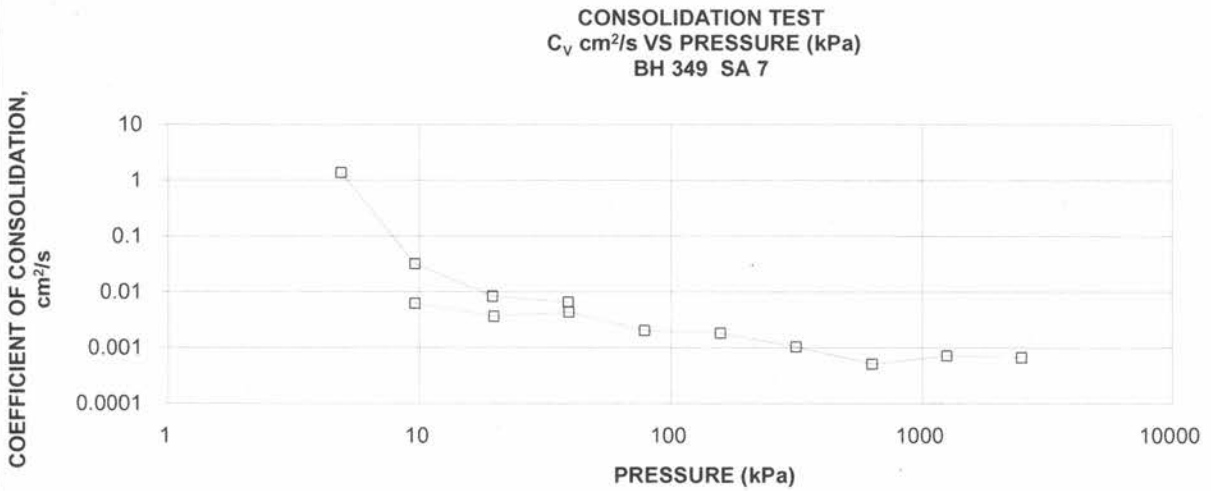
Specimen swelled under 10kPa

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.18	Unit Weight, kN/m ³	18.46
Sample Diameter, cm	6.31	Dry Unit Weight, kN/m ³	13.05
Area, cm ²	31.27	Specific Gravity, measured	2.74
Volume, cm ³	68.23	Solids Height, cm	1.060
Water Content, %	41.45	Volume of Solids, cm ³	33.14
Wet Mass, g	128.45	Volume of Voids, cm ³	35.09
Dry Mass, g	90.81		

CONSOLIDATION TEST SUMMARY

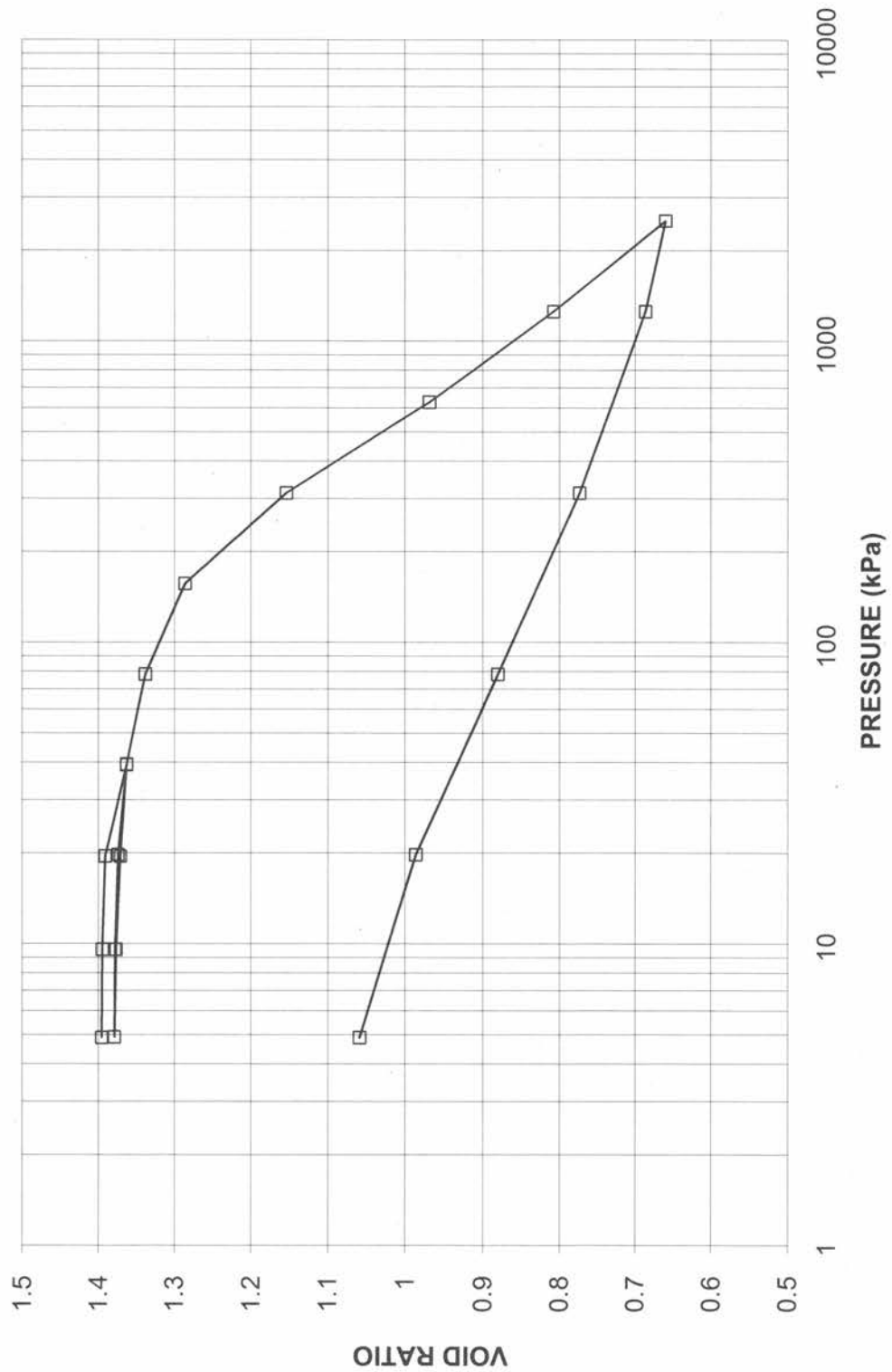
FIGURE BH 349 SA 7 OED B



**CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE**

FIGURE BH 349 SA 7 OED C

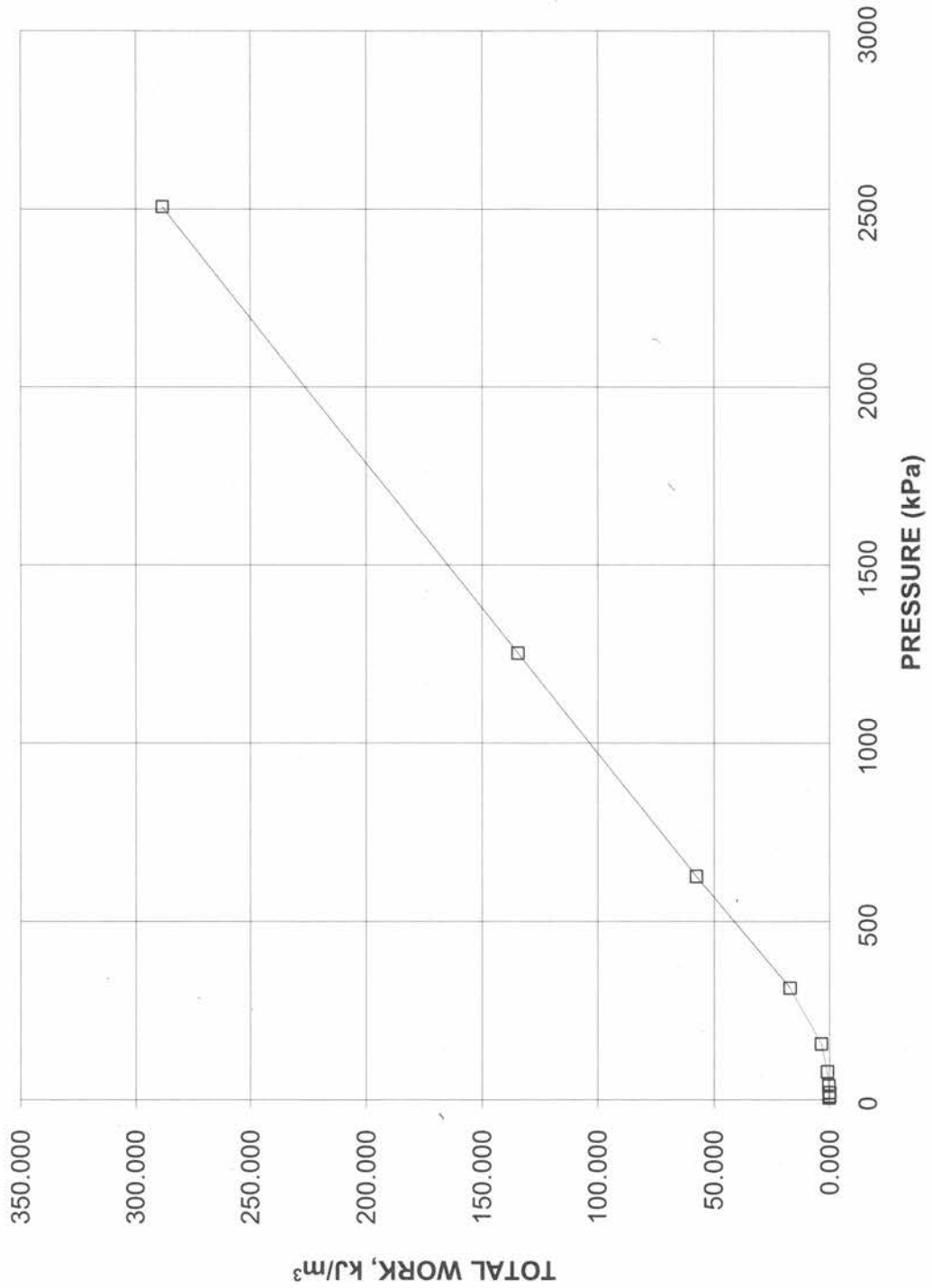
**CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 349 SA 7**



**CONSOLIDATION TEST
TOTAL WORK VS PRESSURE**

FIGURE BH 349 SA 7 OED D

**CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs PRESSURE
BH 349 SA 7**



CONSOLIDATION TEST SUMMARY

FIGURE BH 349 SA 11 OED A

SAMPLE IDENTIFICATION

Project Number	09-1132-0080	Sample Number	11
Borehole Number	349	Sample Depth, m	10.4-10.8

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	5/6/2010		
Date Completed	5/27/2010		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m ³	19.08
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	15.22
Area, cm ²	31.52	Specific Gravity, measured	2.74
Volume, cm ³	59.89	Solids Height, cm	1.076
Water Content, %	25.35	Volume of Solids, cm ³	33.93
Wet Mass, g	116.53	Volume of Voids, cm ³	25.96
Dry Mass, g	92.96	Degree of Saturation, %	90.8

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	c _v cm ² /s	mv m ² /kN	k cm/s
0.00	1.900	0.765	1.900				
4.78	1.880	0.747	1.890	375	2.02E-03	2.21E-03	4.38E-07
9.58	1.869	0.736	1.874	346	2.15E-03	1.21E-03	2.54E-07
19.37	1.853	0.721	1.861	311	2.36E-03	8.66E-04	2.00E-07
38.86	1.831	0.701	1.842	205	3.51E-03	5.94E-04	2.04E-07
77.80	1.803	0.675	1.817	145	4.83E-03	3.80E-04	1.80E-07
38.82	1.806	0.678	1.804				
9.59	1.823	0.693	1.814				
4.78	1.826	0.696	1.824				
9.59	1.824	0.694	1.825	178	3.97E-03	2.19E-04	8.51E-08
19.14	1.820	0.691	1.822	217	3.24E-03	2.26E-04	7.18E-08
38.82	1.811	0.682	1.815	190	3.68E-03	2.41E-04	8.67E-08
77.88	1.799	0.671	1.805	202	3.42E-03	1.63E-04	5.46E-08
155.42	1.759	0.634	1.779	267	2.51E-03	2.71E-04	6.67E-08
310.02	1.689	0.569	1.724	304	2.07E-03	2.39E-04	4.85E-08
620.41	1.616	0.501	1.652	217	2.67E-03	1.24E-04	3.23E-08
1241.70	1.547	0.437	1.581	167	3.17E-03	5.85E-05	1.82E-08
2482.39	1.476	0.371	1.511	135	3.59E-03	3.01E-05	1.06E-08
1241.70	1.489	0.383	1.482				
310.02	1.516	0.408	1.502				
77.88	1.555	0.445	1.536				
19.14	1.598	0.484	1.576				
4.78	1.633	0.517	1.615				

Note:

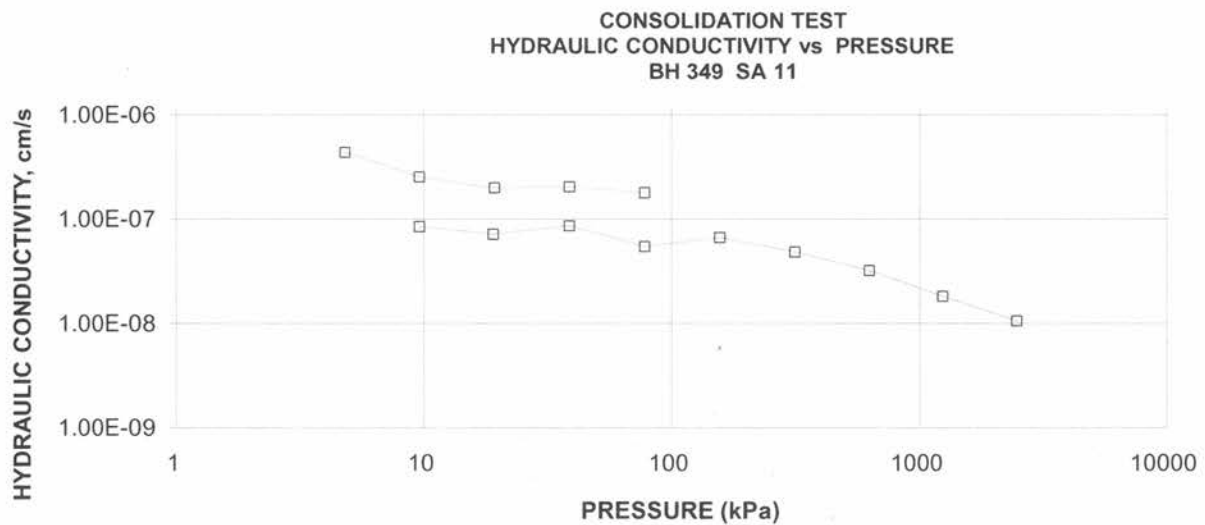
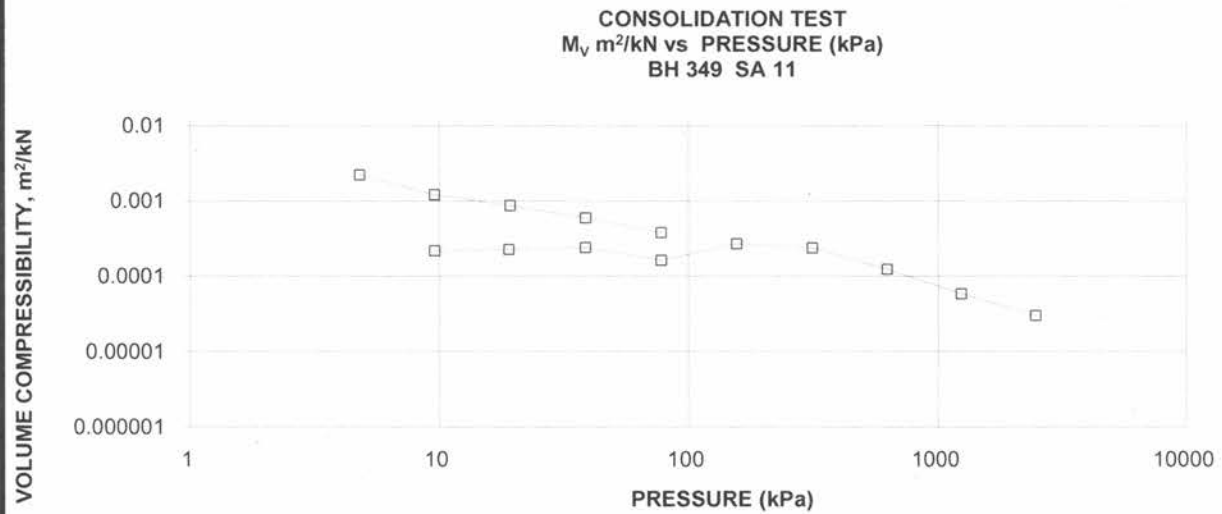
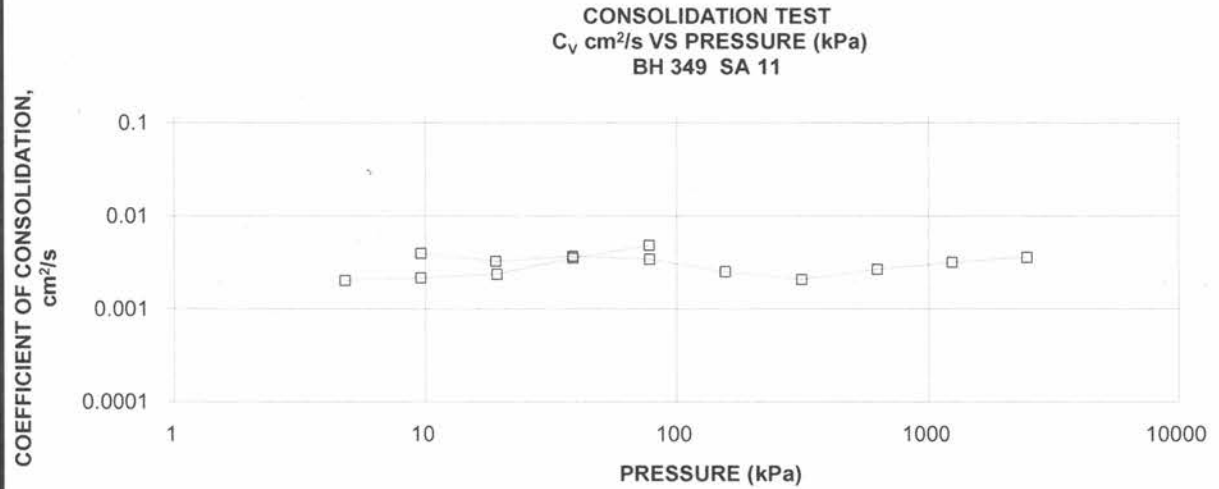
k calculated using cv based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.63	Unit Weight, kN/m ³	21.88
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	17.71
Area, cm ²	31.52	Specific Gravity, measured	2.74
Volume, cm ³	51.47	Solids Height, cm	1.076
Water Content, %	23.55	Volume of Solids, cm ³	33.93
Wet Mass, g	114.85	Volume of Voids, cm ³	17.54
Dry Mass, g	92.96		

CONSOLIDATION TEST SUMMARY

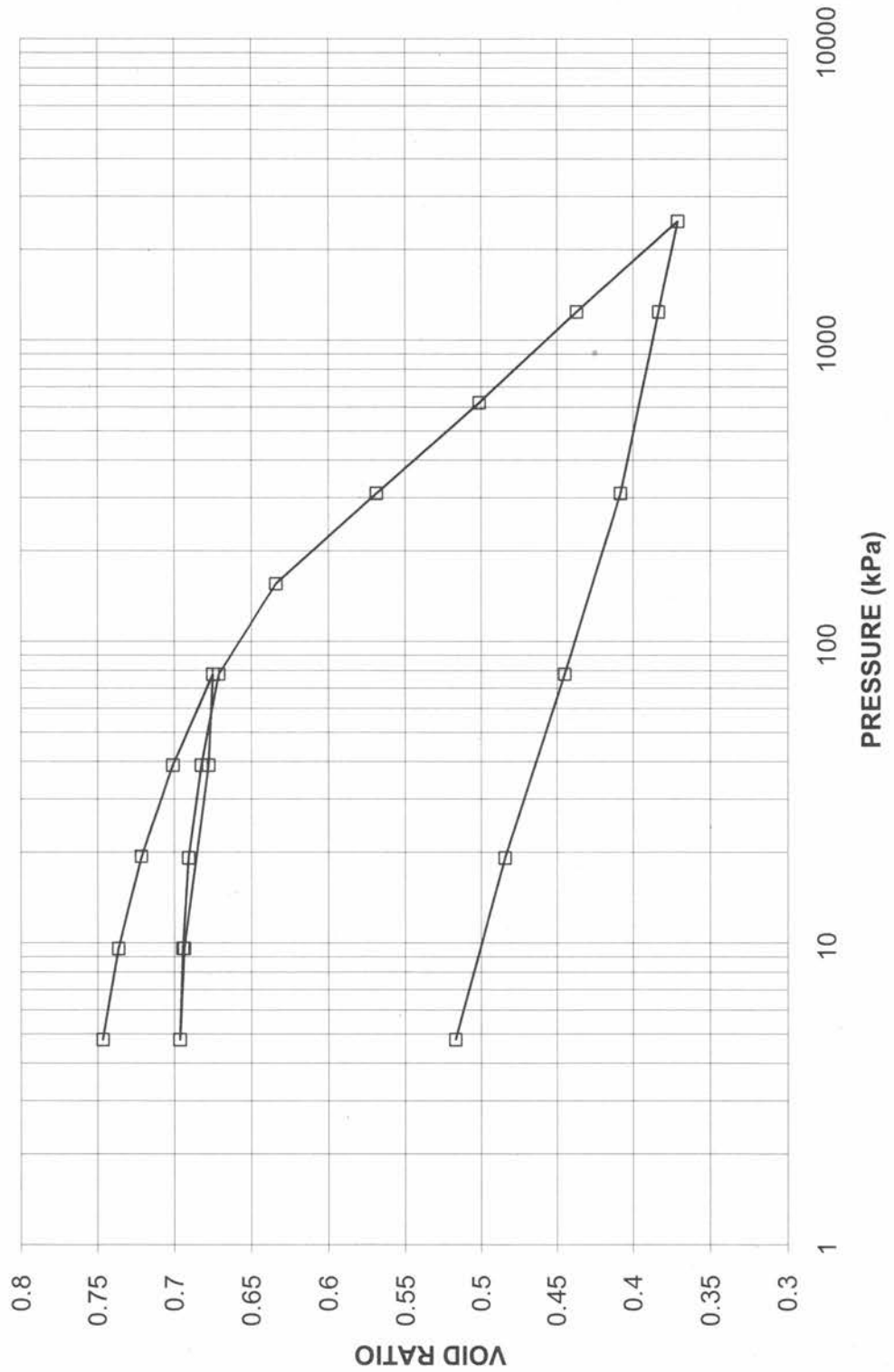
FIGURE BH 349 SA 11 OED B



CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE

FIGURE BH 349 SA 11 OED C

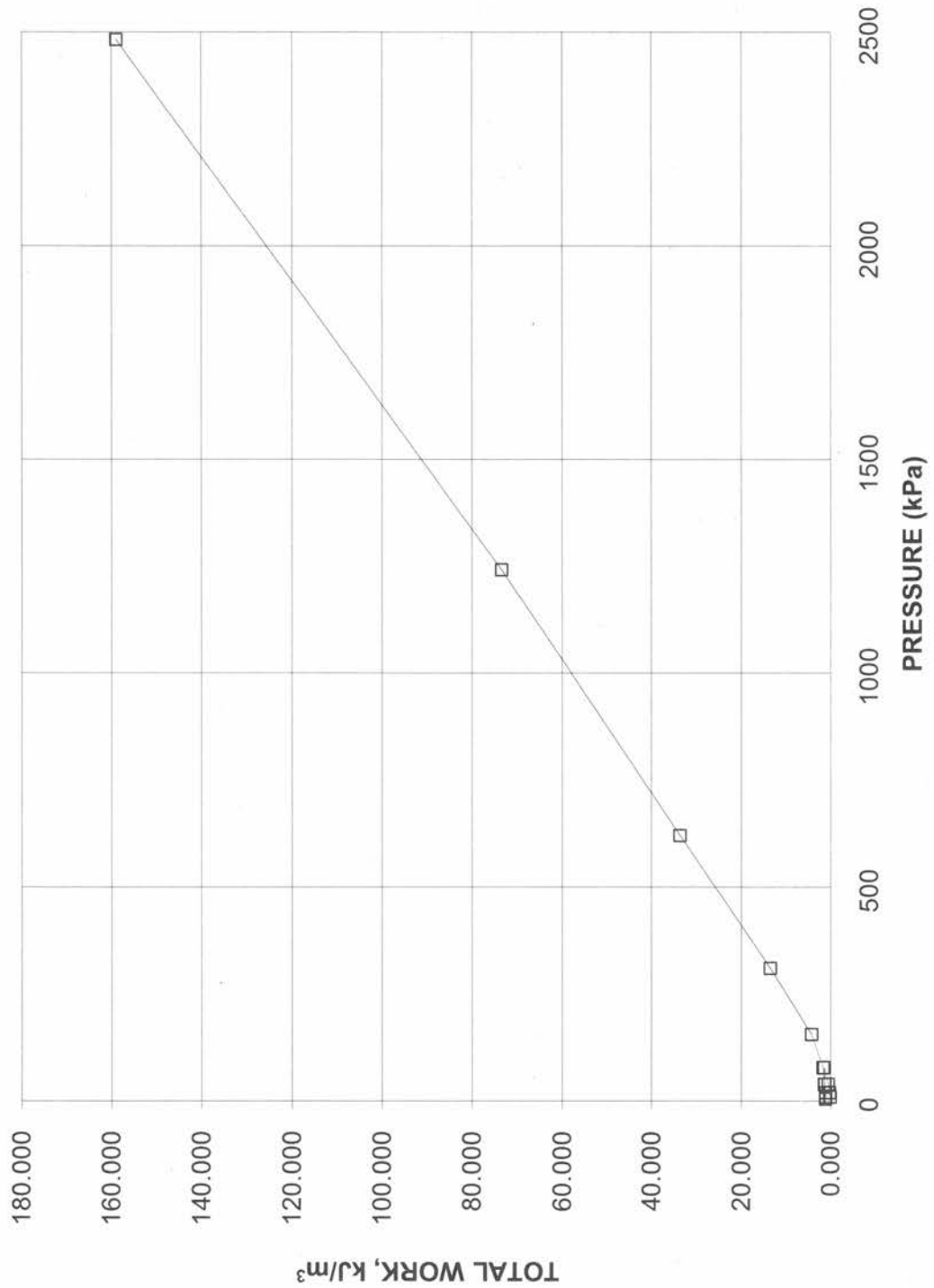
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 349 SA 11



CONSOLIDATION TEST TOTAL WORK VS PRESSURE

FIGURE BH 349 SA 11 OED D

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 349 SA 11



Project No. 09-1132-0080

Prepared By: LFG

Golder Associates

Checked By: *SJB*

CONSOLIDATION TEST SUMMARY

FIGURE BH 349 SA 14 OED A

SAMPLE IDENTIFICATION

Project Number	09-1132-0080	Sample Number	14
Borehole Number	349	Sample Depth, m	14.9-15.4

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	4		
Date Started	5/7/2010		
Date Completed	5/27/2010		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m ³	20.50
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	16.95
Area, cm ²	31.57	Specific Gravity, measured	2.73
Volume, cm ³	80.19	Solids Height, cm	1.608
Water Content, %	20.93	Volume of Solids, cm ³	50.77
Wet Mass, g	167.61	Volume of Voids, cm ³	29.42
Dry Mass, g	138.6	Degree of Saturation, %	98.6

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	c _v cm ² /s	mv m ² /kN	k cm/s
0.00	2.540	0.579	2.540				
4.84	2.504	0.557	2.522	1561	8.64E-04	2.94E-03	2.49E-07
9.58	2.494	0.551	2.499	1470	9.01E-04	8.39E-04	7.40E-08
19.47	2.477	0.540	2.485	1135	1.15E-03	6.65E-04	7.52E-08
38.76	2.450	0.524	2.464	1307	9.85E-04	5.45E-04	5.26E-08
77.76	2.418	0.504	2.434	821	1.53E-03	3.28E-04	4.92E-08
155.17	2.369	0.473	2.393	623	1.95E-03	2.49E-04	4.75E-08
77.76	2.376	0.478	2.373				
19.47	2.391	0.487	2.384				
4.77	2.411	0.499	2.401				
9.56	2.405	0.495	2.408	866	1.42E-03	4.68E-04	6.52E-08
19.34	2.400	0.492	2.402	602	2.03E-03	1.89E-04	3.77E-08
38.81	2.391	0.487	2.396	540	2.25E-03	1.76E-04	3.88E-08
77.62	2.380	0.480	2.386	482	2.50E-03	1.17E-04	2.86E-08
155.14	2.360	0.468	2.370	290	4.11E-03	1.01E-04	4.07E-08
310.14	2.309	0.436	2.335	653	1.77E-03	1.29E-04	2.24E-08
621.08	2.245	0.396	2.277	560	1.96E-03	8.13E-05	1.56E-08
1239.69	2.179	0.355	2.212	317	3.27E-03	4.18E-05	1.34E-08
2479.89	2.098	0.305	2.139	581	1.67E-03	2.58E-05	4.22E-09
1239.69	2.108	0.311	2.103				
310.14	2.138	0.330	2.123				
77.62	2.178	0.355	2.158				
19.47	2.218	0.379	2.198				
4.84	2.255	0.402	2.236				

Note:

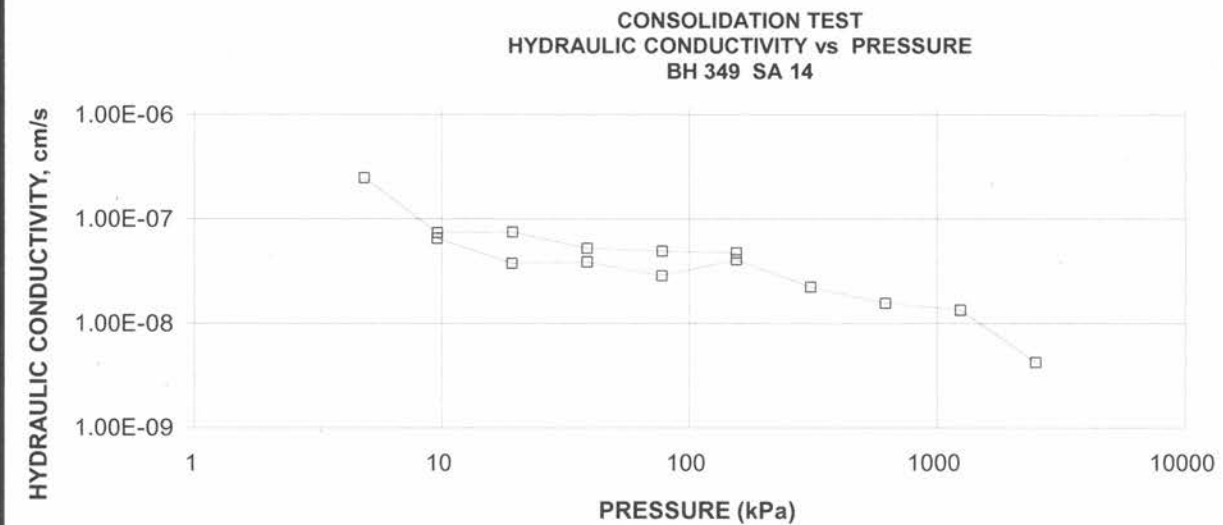
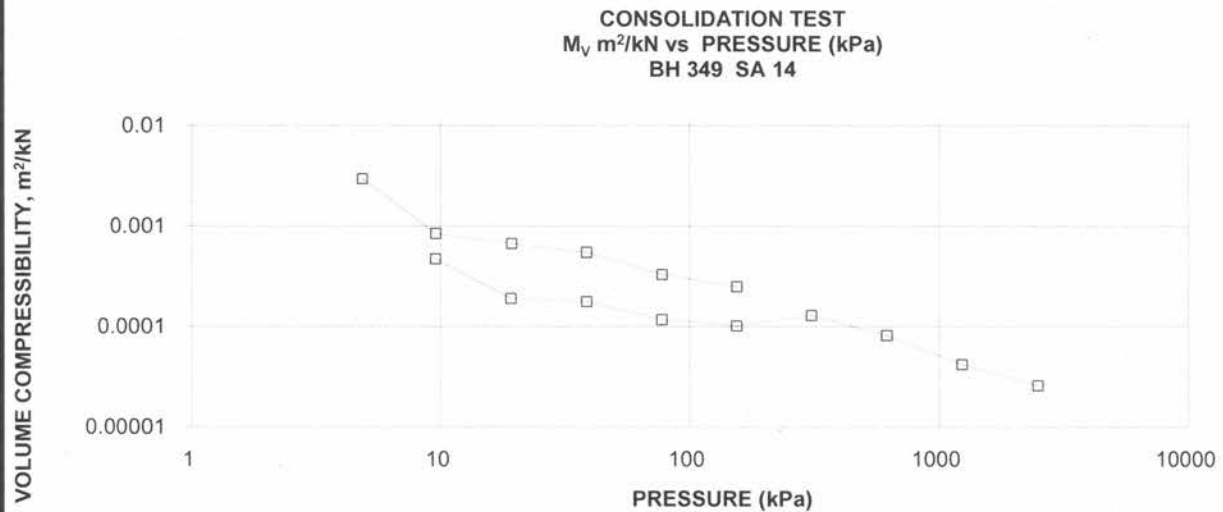
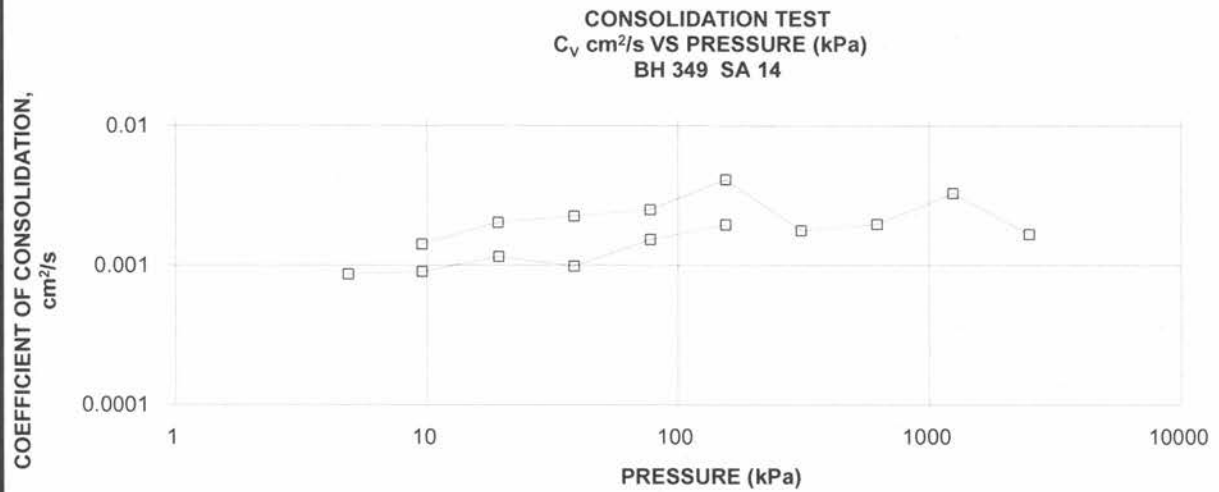
k calculated using c_v based on t₉₀ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.25	Unit Weight, kN/m ³	22.09
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m ³	19.10
Area, cm ²	31.57	Specific Gravity, measured	2.73
Volume, cm ³	71.17	Solids Height, cm	1.608
Water Content, %	15.66	Volume of Solids, cm ³	50.77
Wet Mass, g	160.30	Volume of Voids, cm ³	20.40
Dry Mass, g	138.6		

CONSOLIDATION TEST SUMMARY

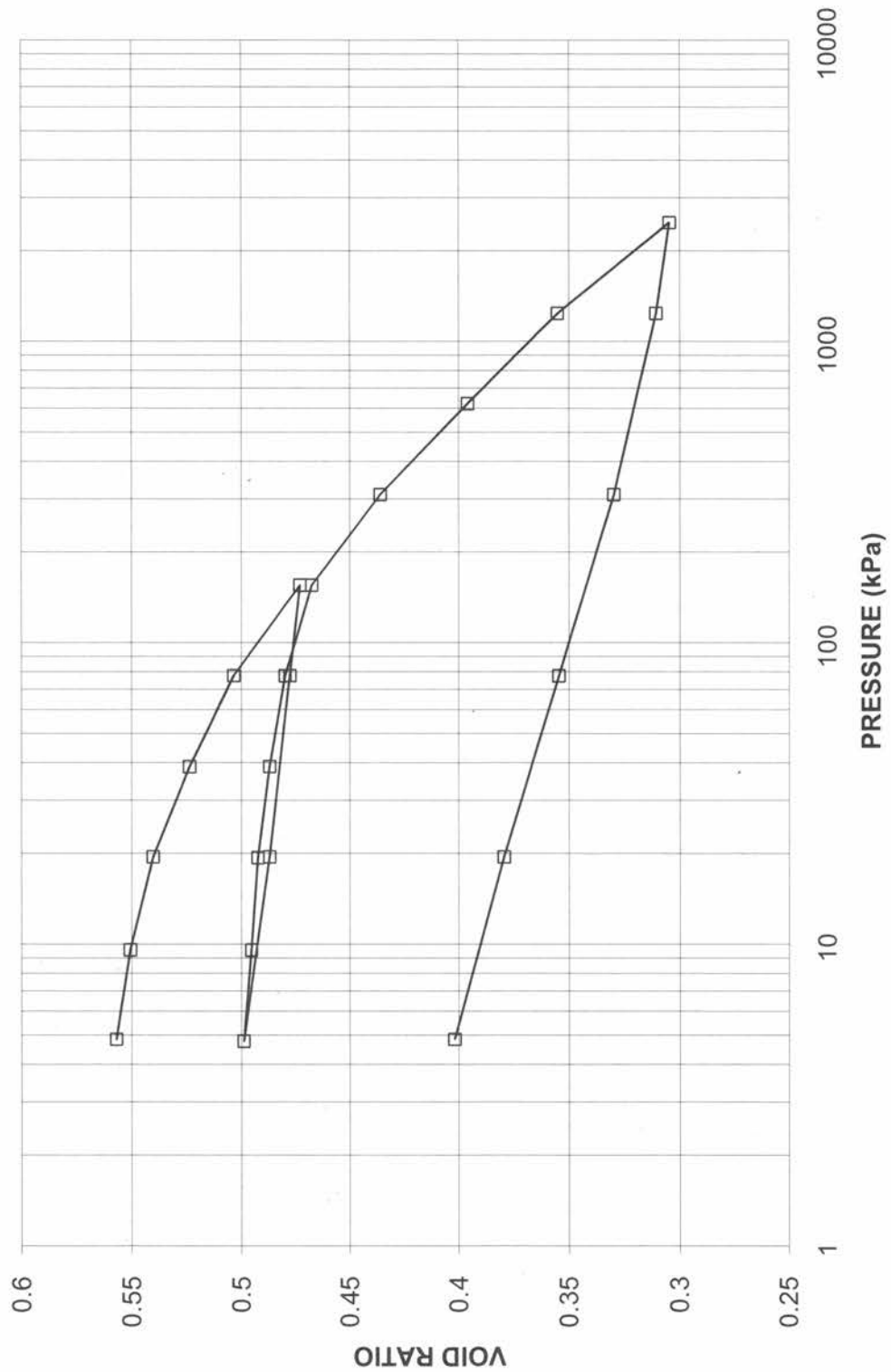
FIGURE BH 349 SA 14 OED B



CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE BH 349 SA 14 OED C

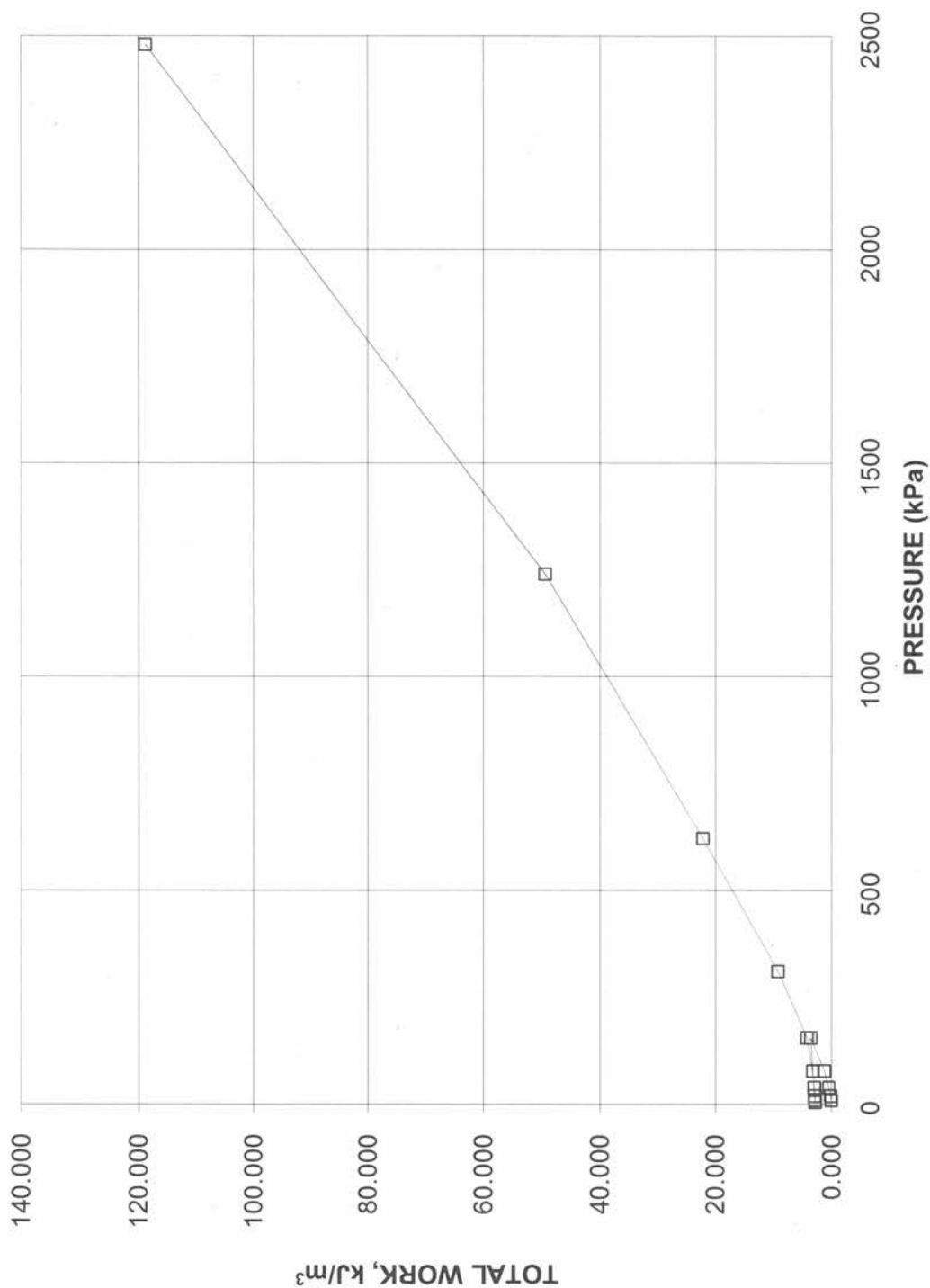
CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 349 SA 14



CONSOLIDATION TEST TOTAL WORK VS PRESSURE

FIGURE BH 349 SA 14 OED D

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 349 SA 14





APPENDIX F

**Record of Previous Boreholes and Laboratory Testing
Golder Associates Project No. 09-1132-0039-1000**

PROJECT 09-1132-0039

RECORD OF BOREHOLE No GBH-167

1 OF 3

METRIC

W.P.

LOCATION

N 4682025.1 : E 328316.1

ORIGINATED BY NG

DIST WEST HWY

BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY WITH NQRC

COMPILED BY DMB

DATUM GEODETIC

DATE

May 25, 2009 - May 28, 2009

CHECKED BY *SJB*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
179.03	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.00	TOPSOIL, sandy Loose Black							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
178.12														
0.91	FILL, sand, trace silt, with topsoil and organic material Loose to compact Brown		1	SS	6		178							
			2	SS	12		177							
176.67														
2.36	CLAYEY SILT, trace sand, with silt layers and partings Firm to stiff Grey		3	SS	9		176							
			4	SS	4									0 3 82 15
174.99							175							
4.04	SILTY CLAY TO CLAYEY SILT, some sand, trace gravel Soft to Firm Grey		5	TO	PH		174							0 2 31 67 Oedometer
			6	SS	3		173							
							172							
			7	SS	3		171							0 19 44 37
							170							
			8	SS	2		169							
169.18							168							
9.85	CLAYEY SILT TO SILTY CLAY, some sand, trace gravel Soft to stiff Grey		9	TO	PH		167							0 22 46 32 Oedometer
			10	SS	3		166							
							165							
			11	SS	6									3 23 50 24

LDN.MTO.06 09-1132-0039.GPJ LDN.MTO.GDT 09/04/10

Continued Next Page

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

PROJECT: 09-1132-0039

RECORD OF DRILLHOLE: **GBH-167**

SHEET 3 OF 3

LOCATION: N 4682025.1 ; E 328316.1

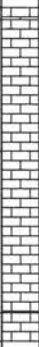
DRILLING DATE: May 25, 2009 - May 28, 2009

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: LANTECH DRILLING SERVICES INC.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		PENETRATION RATE (mm/min)	FLUSH	ELEVATION	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth Ro - Rough Br - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.										DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)	RUN No.				RECOVERY		R.Q.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec					
									TOTAL CORE %	SOLID CORE %			DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION						
															0 5 10 15 20 25 30 35 40 45 50	0 5 10 15 20 25 30 35 40 45 50	0 5 10 15 20 25 30 35 40 45 50	0 5 10 15 20 25 30 35 40 45 50		
		ROCK SURFACE		154.49																
25	MUD ROTARY NO ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, fine to medium grained, faintly porous, grey - Broken core from 24.54m to 24.66m depth.		24.54 24.66	1			154												
26		LIMESTONE, fresh, medium strong, thinly laminated, very fine to medium grained, faintly porous to porous with pits, mottled dark brown-grey to light grey, fossiliferous		153.06 25.97	2			153												
27		LIMESTONE, fresh, medium strong, thinly laminated, very fine to medium grained, faintly porous to porous with pits, grey, stylolitic, fossiliferous			3			152												
28		LIMESTONE, fresh, medium strong, thinly laminated, fine to medium grained, faintly porous to porous with pits, brown to tannish-grey, fossiliferous END OF DRILLHOLE		151.45 27.58 151.17 27.86																
29																				
30																				
31																				
32																				
33																				
34																				
35																				
36																				
37																				
38																				
39																				

DEPTH SCALE

1:75


 LOGGED: *ug*
 CHECKED: *SB*

PROJECT: 09-1132-0039

RECORD OF CONE PENETRATION TEST CPT 167

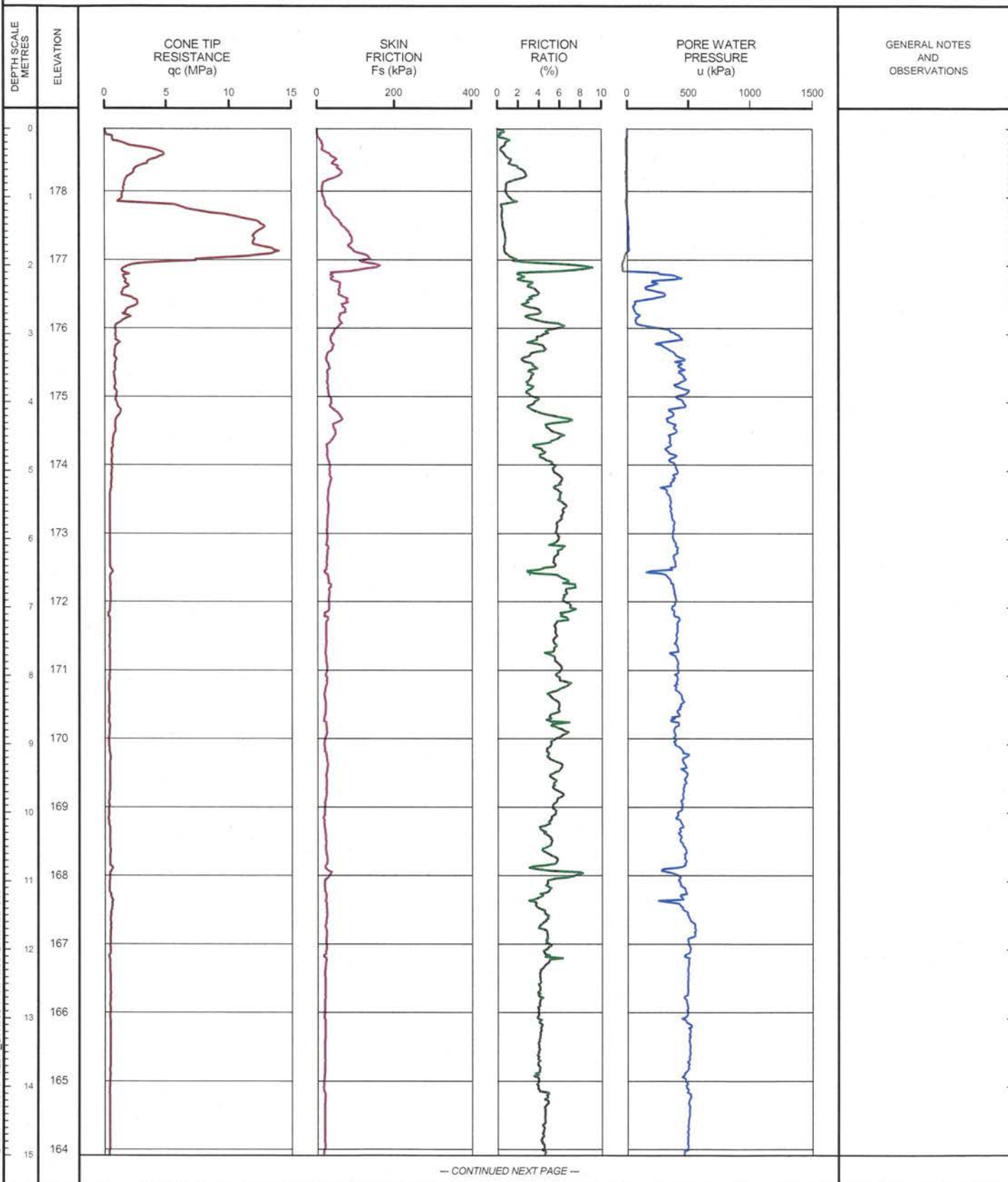
SHEET 1 OF 2

LOCATION: N 4682026.8 ; E 328313.4

TEST DATE: June 3, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.91m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LON_CPT_01 0911320039-CPT.GPJ GLDR_LON.GDT 11/19/09 DATA INPUT:

DEPTH SCALE

1 : 75

OPERATOR: *cu*CHECKED: *SSB*

PROJECT: 09-1132-0039

RECORD OF CONE PENETRATION TEST CPT 167

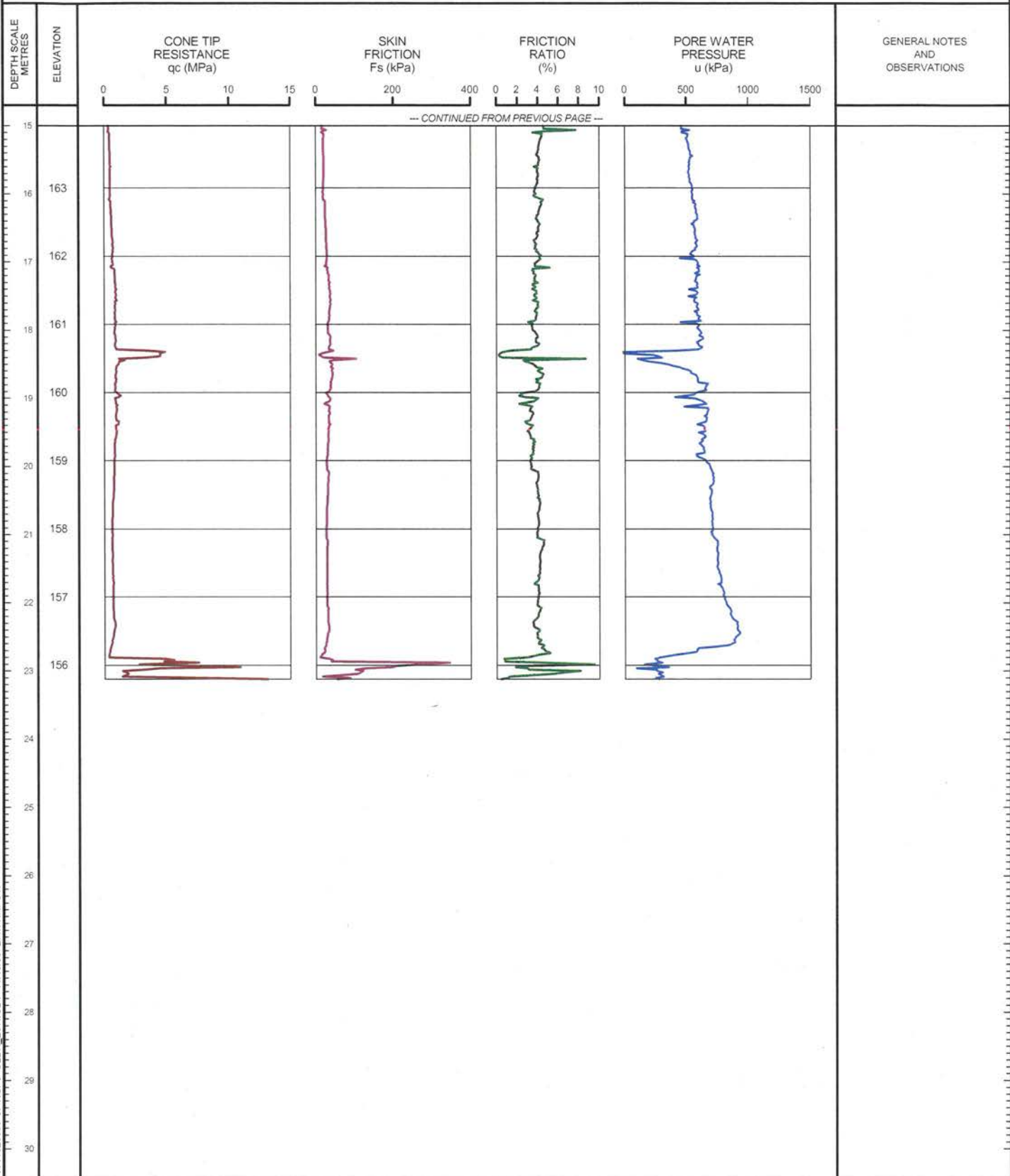
SHEET 2 OF 2

LOCATION: N 4682026.8 ; E 328313.4

TEST DATE: June 3, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.91m PREDRILL DEPTH: 0.00m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LDN_CPT_01 0911320039-CPT.GPJ GLDR_LON.GDT 11/19/09 DATA INPUT:

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SDB

RECORD OF BOREHOLE No CPT-169

1 OF 1

METRIC

PROJECT 09-1132-0039

W.P.

LOCATION

N 4682229.5 ; E 328208.8

ORIGINATED BY CC

DIST

WEST HWY

BOREHOLE TYPE

POWER AUGER, SOLID STEM

COMPILED BY LMK

DATUM GEODETIC

DATE

June 4, 2009

CHECKED BY *SJB*

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			20	40	60	80	100			
178.57	GROUND SURFACE														
0.00	FILL, crushed sand and gravel, Grey														
0.17	FILL, sand and gravel, Brown														
0.46	TOPSOIL, sandy, Black														
0.61	SILTY SAND, trace gravel, trace clay		1	SS	11		178								
177.35	Compact Brown														
1.22	SAND AND GRAVEL		2	SS	19		177								
176.97	Compact, Brown														
1.60	SILT, trace sand														
1.86	Compact, Grey														
	CLAYEY SILT, trace sand, with silt partings		3	SS	11		176								
	Stiff Grey														
175.52	END OF BOREHOLE														
3.05	Groundwater encountered at about elev. 177.37m during drilling on June 4, 2009.														

PROJECT: 09-1132-0039

RECORD OF CONE PENETRATION TEST CPT 169

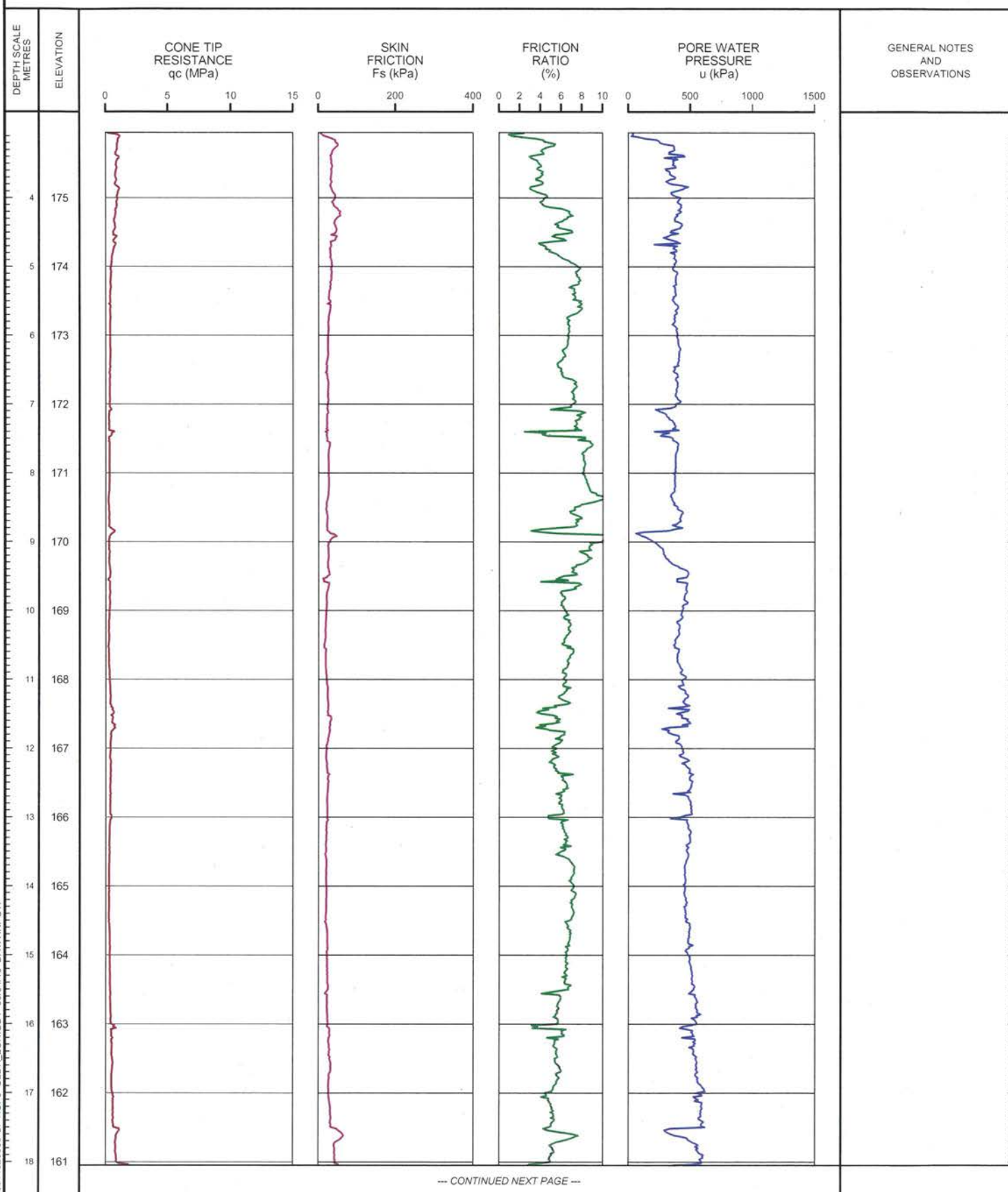
SHEET 1 OF 2

LOCATION: N 4682229.5 ; E 328208.8

TEST DATE: June 4, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.57m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



LDN CPT 01 0911320039 CPT GPJ GLDR LON GDT 09/04/10 DATA INPUT

DEPTH SCALE

1 : 75

**Golder
Associates**OPERATOR: *CL*CHECKED: *SJB*

PROJECT: 09-1132-0039

RECORD OF CONE PENETRATION TEST CPT 169

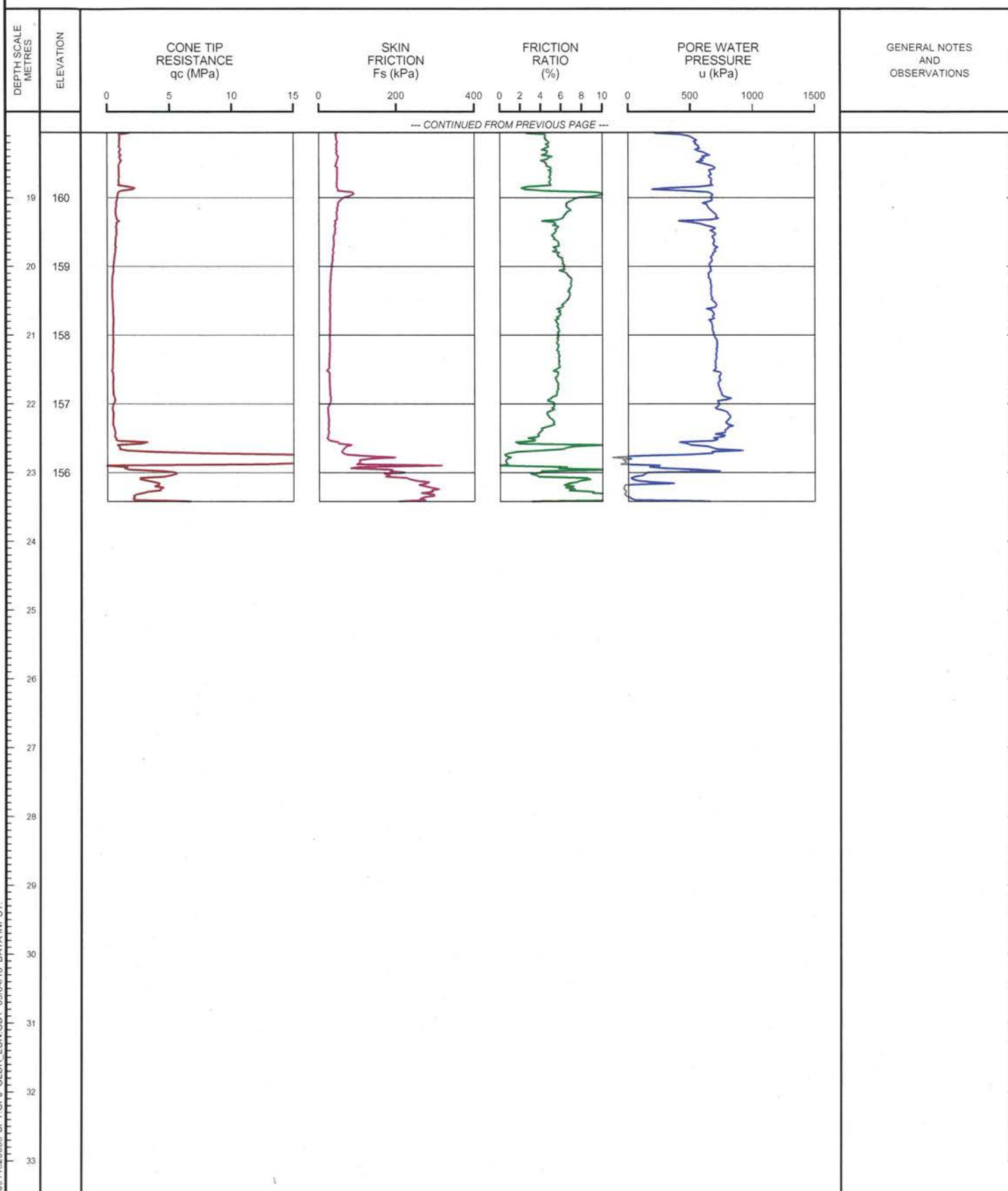
SHEET 2 OF 2

LOCATION: N 4682229.5 ;E 328208.8

TEST DATE: June 4, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.57m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.6 CORRECTION FACTOR B: 0.013



DEPTH SCALE

1: 75

OPERATOR: *CC*CHECKED: *SSB*

LDN_CPT_01_0911320039-CPT.GPJ GLDR_LON_GDT_090410_DATA.INPUT

PROJECT <u>09-1132-0039</u>		RECORD OF BOREHOLE No CPT-171		1 OF 1	METRIC
W.P. _____		LOCATION <u>N 4682264.8 ; E 328114.3</u>		ORIGINATED BY <u>CC</u>	
DIST <u>WEST</u> HWY _____		BOREHOLE TYPE <u>POWER AUGER, SOLID STEM</u>		COMPILED BY <u>LMK</u>	
DATUM <u>GEODETIC</u>		DATE <u>June 4, 2009</u>		CHECKED BY <u>SB</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								<div><div></div><div>20406080100</div></div> <div>○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE</div>										
178.14 0.00	GROUND SURFACE FILL, recycled aggregate	<div></div>				▽	178											
177.53 0.61	FILL, concrete	<div></div>					177											
0.91	FILL, silty sand and gravel, with topsoil and concrete	<div></div>	1	SS	12		176											
176.46 1.68	Compact Brown and grey	<div></div>	2	SS	11													
	CLAYEY SILT, trace sand, with silt partings Stiff Grey	<div></div>	3	SS	9													
175.09 3.05	END OF BOREHOLE																	
	Groundwater encountered at about elev. 175.84m during drilling on June 4, 2009.																	

PROJECT: 09-1132-0039

RECORD OF CONE PENETRATION TEST CPT 171

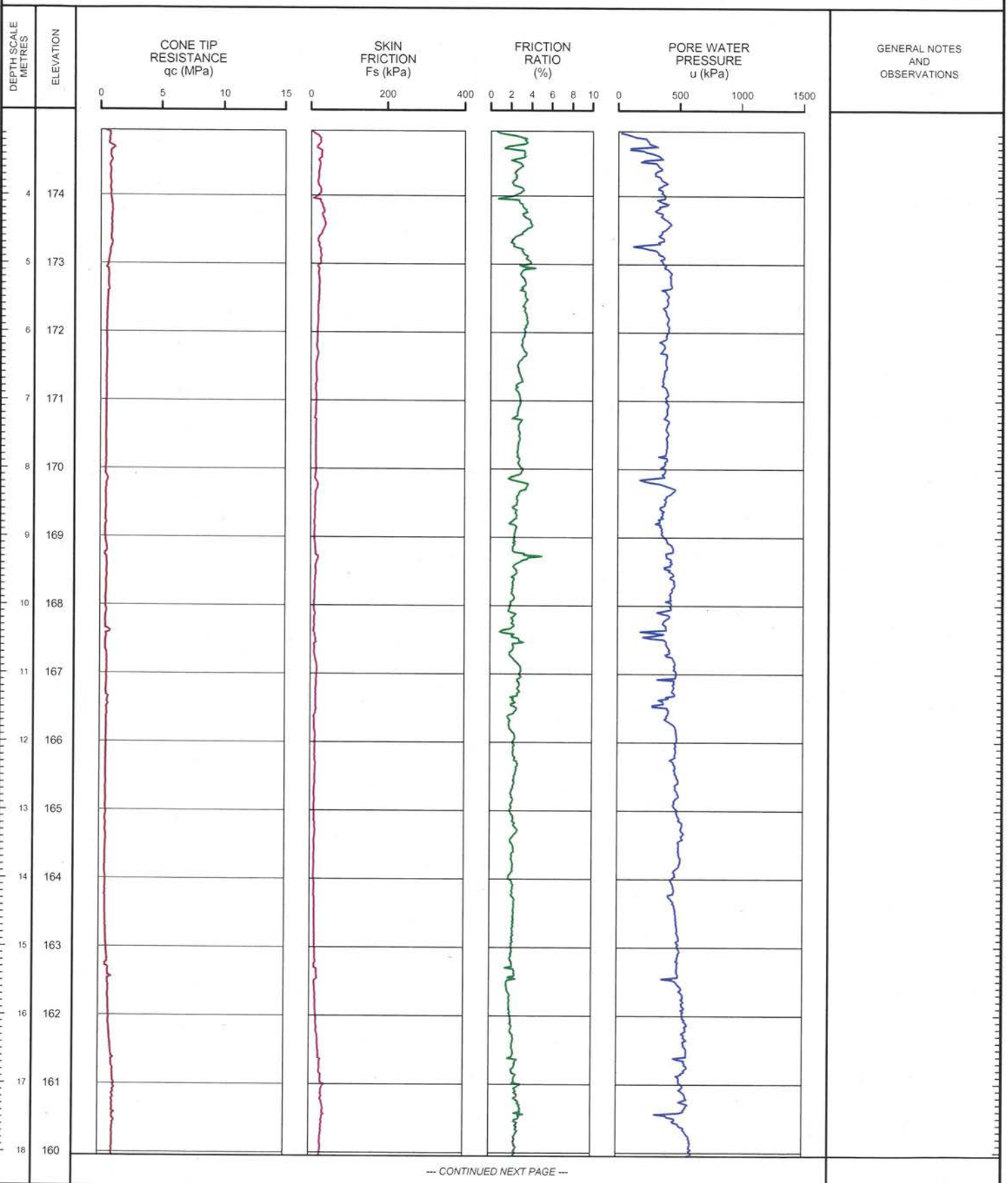
SHEET 1 OF 2

LOCATION: N 4682264.8 ; E 328114.3

TEST DATE: June 4, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.14m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



LDN_CPT_01_0911320039-CPT.GPJ GLDR LON GDT 09/04/10 DATA INPUT:

DEPTH SCALE

1 : 75



OPERATOR: CC

CHECKED: SJB

PROJECT: 09-1132-0039

RECORD OF CONE PENETRATION TEST CPT 171

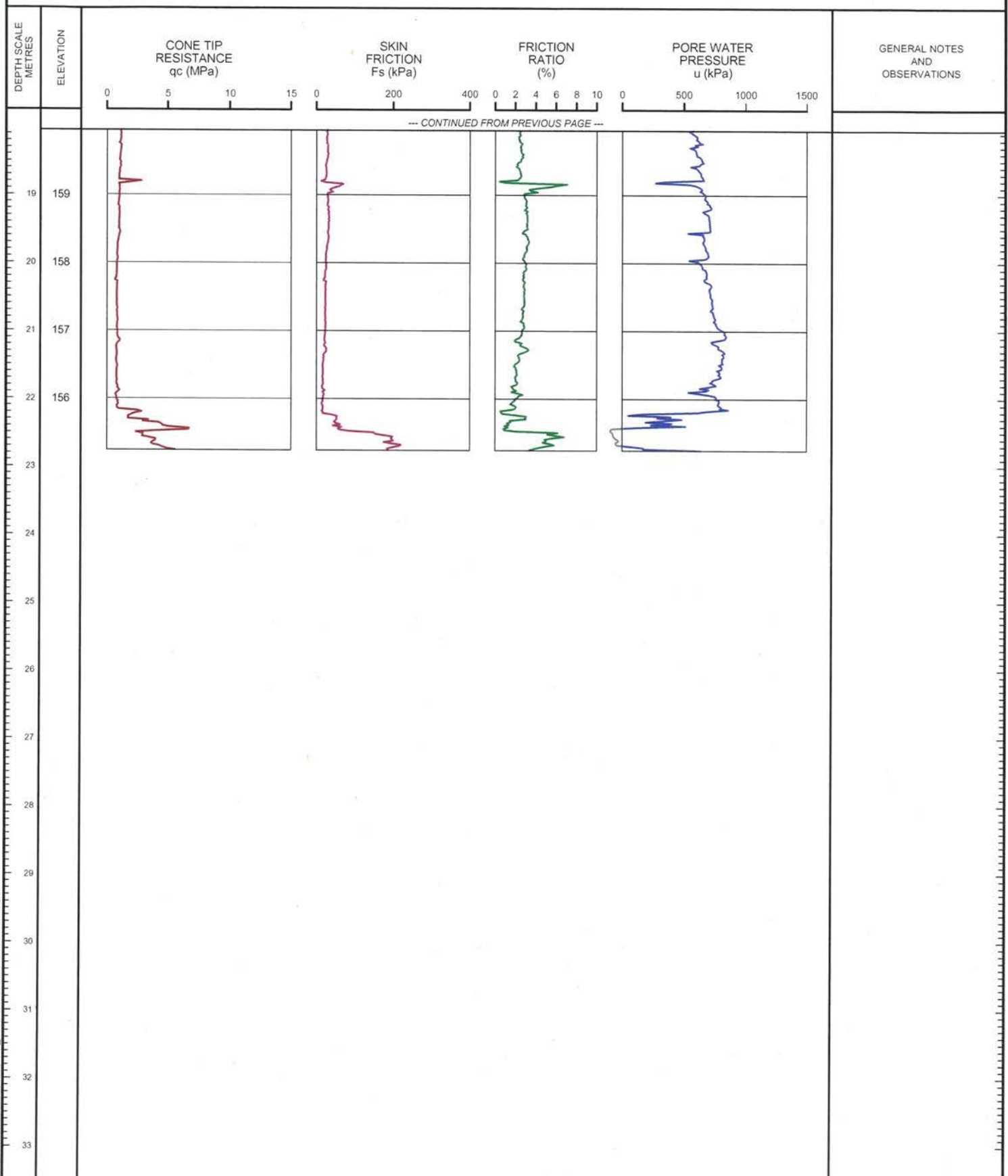
SHEET 2 OF 2

LOCATION: N 4682264.8 ; E 328114.3

TEST DATE: June 4, 2009

DATUM: GEODETIC

GROUND SURFACE ELEVATION: 178.14m PREDRILL DEPTH: 3.05m CORRECTION FACTOR A: 0.584 CORRECTION FACTOR B: 0.012



DEPTH SCALE

1 : 75

OPERATOR: CC
CHECKED: SSB

LDN_CPT_01_0911320039-CPT.GPJ GLDR_LON.GDT 09/04/10 DATA INPUT:

PROJECT 09-1132-0039

RECORD OF BOREHOLE No GBH-172

1 OF 4

METRIC

W.P. LOCATION N 4682120.1 E 328054.0

ORIGINATED BY MR

DIST WEST HWY BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY WITH NQRC

COMPILED BY DMB

DATUM GEODETIC DATE May 19, 2009 - May 21, 2009

CHECKED BY **SJS**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							20 40 60 80 100							
178.23	GROUND SURFACE													
0.00	TOPSOIL, silty													
177.83	Black													
0.40	FILL, silty sand, some gravel, trace topsoil													
177.16	Very loose, Brown		1	SS	2									
1.07	SAND, some gravel, trace silt													
1.37	Very loose, Brown													
176.55	SAND, fine to medium													
1.68	Compact, Grey		2	SS	19									
176.10	SILT, some sand, trace clay													
2.13	Compact, Grey													
	CLAYEY SILT, with silt partings		3	SS	6									0 2 76 22
	Firm													
	Grey		4	SS	6									
174.12														
4.11	SILTY CLAY													
	Firm		5	SS	5									
	Grey													
172.90														
5.33	SILTY CLAY TO CLAYEY SILT, some sand, trace gravel													
	Soft to stiff		6	SS	3									1 14 33 52
	Grey													
			7	SS	4									
			8	SS	3									
168.48														
9.75	CLAYEY SILT, some sand, trace gravel													
	Firm to stiff		9	SS	4									
	Grey													
			10	SS	5									
			11	SS	6									4 25 47 27

Continued Next Page

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

RECORD OF BOREHOLE No GBH-172

2 OF 4

METRIC

PROJECT 09-1132-0039

W.P.

LOCATION

N 4682120.1 ; E 328054.0

ORIGINATED BY MR

DIST WEST HWY

BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY WITH NQRC

COMPILED BY DMB

DATUM GEODETIC

DATE

May 19, 2009 - May 21, 2009

CHECKED BY *SB*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey						163					
							162					
							161					
			13	SS	13		160					
							159					
							158					
			14	SS	9		157					
							156					
155.37							155					
22.86	CLAYEY SILT, some sand, trace to some gravel, with cobbles Hard Grey						154					
			15	SS	76		153					
							152					
152.32							151					
25.91	LIMESTONE, fresh medium strong, weakly laminated to laminated, very fine to medium grained, faintly porous to porous, with occasional pits Dark brown to grey (FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)		16	WS			150					
			17	SS			149					
			18	NQ RC								
			19	NQ RC								
			20	NQ RC								

Continued Next Page

+ 3, x 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

LDN_MTO_01 09-1132-0039.GPJ LDN_MTO.GDT 15/04/10

RECORD OF BOREHOLE No GBH-172

3 OF 4

METRIC

PROJECT 09-1132-0039

W.P. _____

LOCATION N 4682120.1 :E 328054.0

ORIGINATED BY MR

DIST WEST HWY _____

BOREHOLE TYPE POWER AUGER, HOLLOW STEM, MUD ROTARY WITH NQRC

COMPILED BY DMB

DATUM GEODETIC

DATE May 19, 2009 - May 21, 2009

CHECKED BY **SJB**

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE					
							20	40	60	80	100					
					</											

SHEET 4 OF 4

DATUM: GEODETIC

DRILLING CONTRACTOR: LANTECH DRILLING SERVICES INC.

[illegible]

DN_ROCK_03 0911320039-ROCK.GPJ GLDR_LDN.GDT 11/19/09 DATA INPUT: WDF

DEPTH SCALE

1:75



LOGGED: *SP/ML*

CHECKED:

RECORD OF BOREHOLE No GBH-193

1 OF 3

METRIC

PROJECT 09-1132-0039

W.P.

LOCATION

N 4682284.0 :E 328306.8

ORIGINATED BY SM

DIST

WEST HWY

BOREHOLE TYPE

POWER AUGER, HOLLOW STEM, MUD ROTARY WITH NQRC

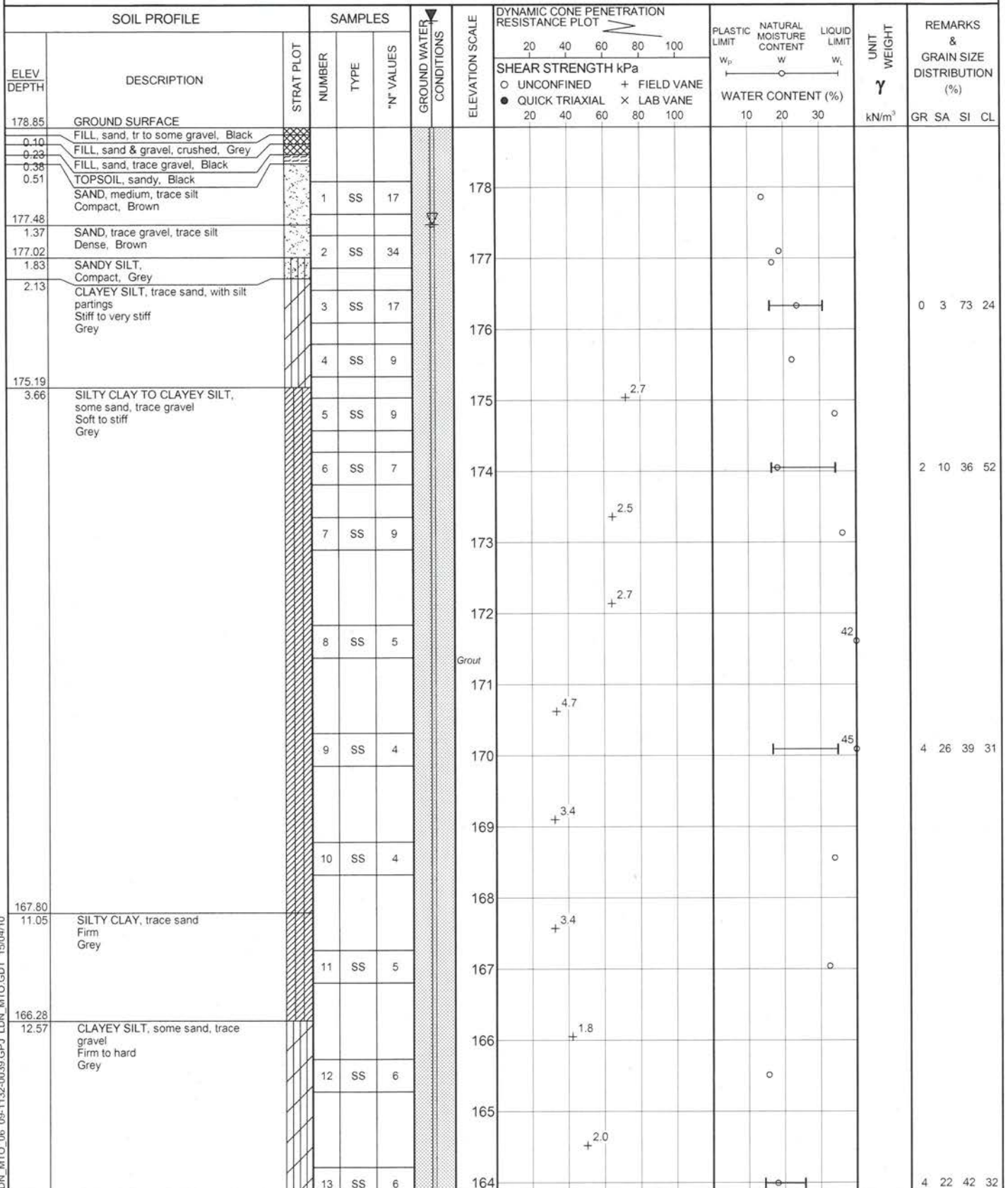
COMPILED BY DMB

DATUM GEODETIC

DATE

June 8, 2009 - June 9, 2009

CHECKED BY



Continued Next Page

+ 3, x 3; Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

LDN_MTO_06 09-1132-0039.GPJ LDN_MTO.GDT 15/04/10

PROJECT: 09-1132-0039

RECORD OF DRILLHOLE: **GBH-193**

SHEET 3 OF 3

LOCATION: N 4682284.0 ;E 328306.8

DRILLING DATE: June 8, 2009 - June 9, 2009

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: MUD ROTARY WITH HQ TRICONE, NQRC

DRILLING CONTRACTOR: LANTECH DRILLING SERVICES INC.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR	% RETURN	ELEVATION	JN - Joint		BD - Bedding		PL - Planar		PO - Polished		Br - Broken Rock		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION					
											FLT - Fault	SHR - Shear	VN - Vein	CJ - Conjugate	FO - Foliation	CO - Contact	OR - Orthogonal	CL - Cleavage	CU - Curved	UN - Undulating			ST - Stepped	IR - Irregular	K - Slickensided	SM - Smooth	Ro - Rough
											TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRAC INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec										
		ROCK SURFACE		154.13																							
25	MUD ROTARY NO ROCK CORE	LIMESTONE, slightly weathered, medium strong, thinly laminated, moderately fractured, fine to medium grained, faintly porous, greyish brown, fossiliferous - Broken core from 25.09m to 25.12m, 25.25m to 25.30m, 25.40m to 25.45m and 25.53m to 25.60m depth		24.72						154																	
		153.25		1																							
26		25.60																									
27																											
		LIMESTONE, fresh, medium strong, thinly laminated, moderately fractured, fine to medium grained, faintly porous to vuggy porosity in sections, light tannish grey, fossiliferous		151.47						152																	
28		LIMESTONE, slightly weathered, medium strong, thinly laminated, moderately fractured, fine to medium grained, vuggy porosity, brown-grey, fossiliferous		27.38																							
		END OF DRILLHOLE		150.61						151																	
				28.24																							
29																											
30																											
31																											
32																											
33																											
34																											
35																											
36																											
37																											
38																											
39																											

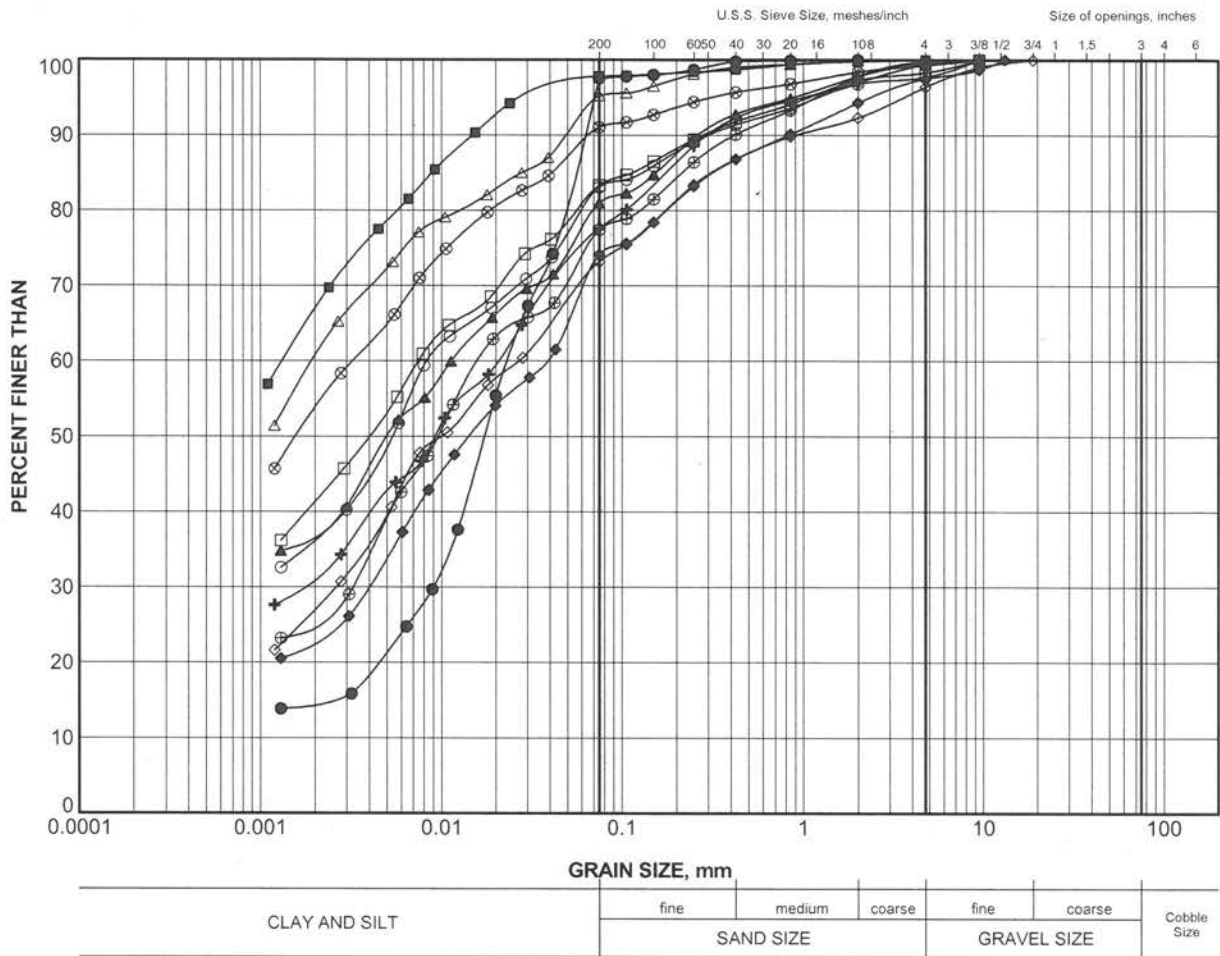
DEPTH SCALE

1 : 75



LOGGED: SM

CHECKED: SJB



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GBH-167	4	176.7
■	GBH-167	5	174.2
▲	GBH-167	7	171.5
+	GBH-167	9	168.4
◆	GBH-167	11	165.4
◇	GBH-167	12	163.8
○	GBH-167	14	157.7
△	GBH-170	6	172.8
⊗	GBH-170	9	168.3
⊕	GBH-170	12	163.7
□	GBH-170	14	157.6

PROJECT

GEOTECHNICAL DATA REPORT
CANADIAN INSPECTION PLAZA
AND RELATED INFRASTRUCTURE
WINDSOR, ONTARIO

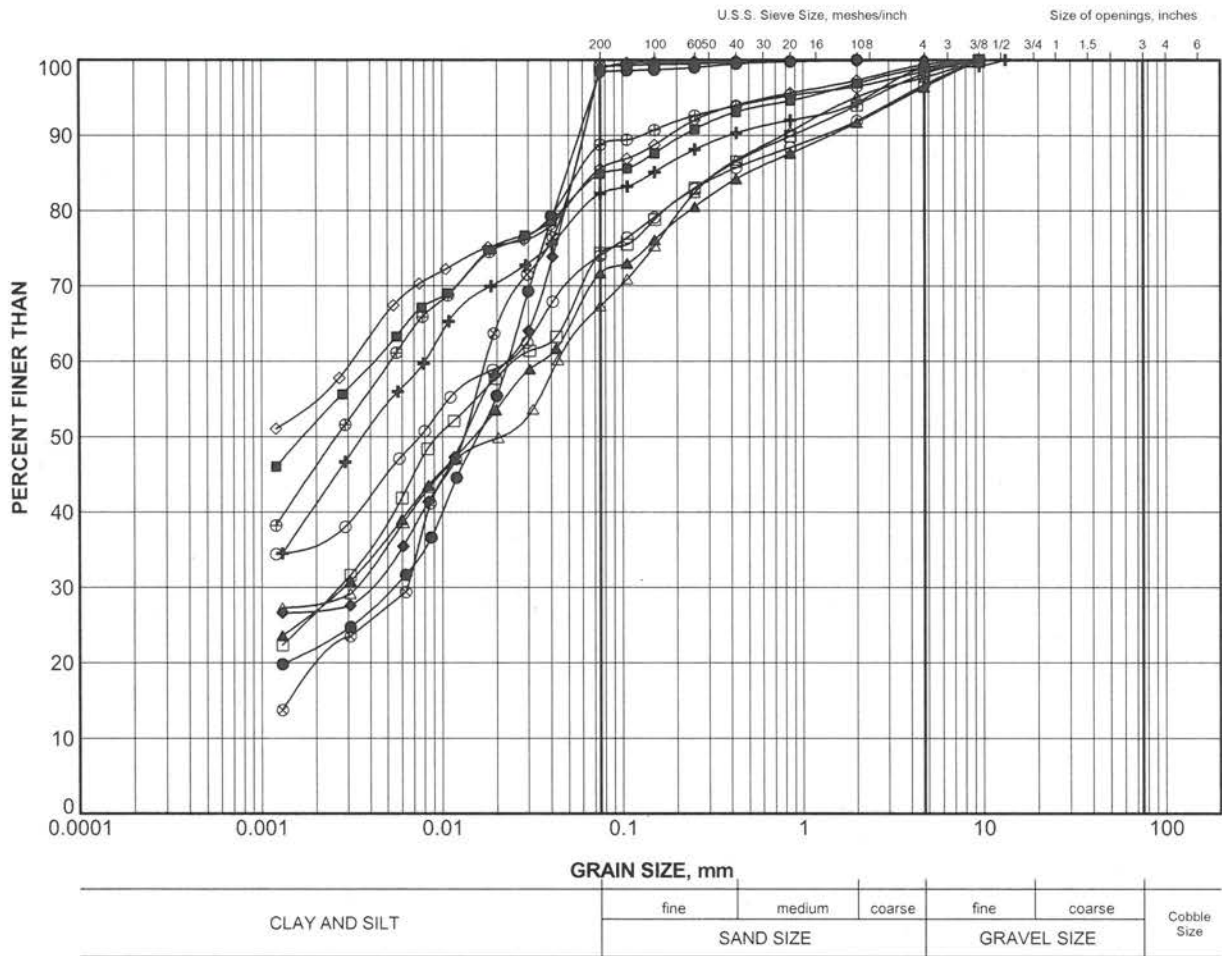
TITLE

GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY



**Golder
Associates**
LONDON, ONTARIO

PROJECT No.	09-1132-0039	FILE No.	0911320039-1000-R020E2
DRAWN	WDF	Apr 9/10	SCALE N/A REV.
CHECK	WDF	Apr 10	FIGURE E-2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GBH-172	3	175.7
■	GBH-172	6	172.4
▲	GBH-172	11	164.9
+	GBH-172	14	157.3
◆	GBH-176	2	175.6
◇	GBH-176	5	171.4
○	GBH-176	10	163.8
△	GBH-176	13	156.1
⊗	GBH-178	4	175.0
⊕	GBH-178	8	169.4
□	GBH-178	12	163.2

PROJECT

GEOTECHNICAL DATA REPORT
CANADIAN INSPECTION PLAZA
AND RELATED INFRASTRUCTURE
WINDSOR, ONTARIO

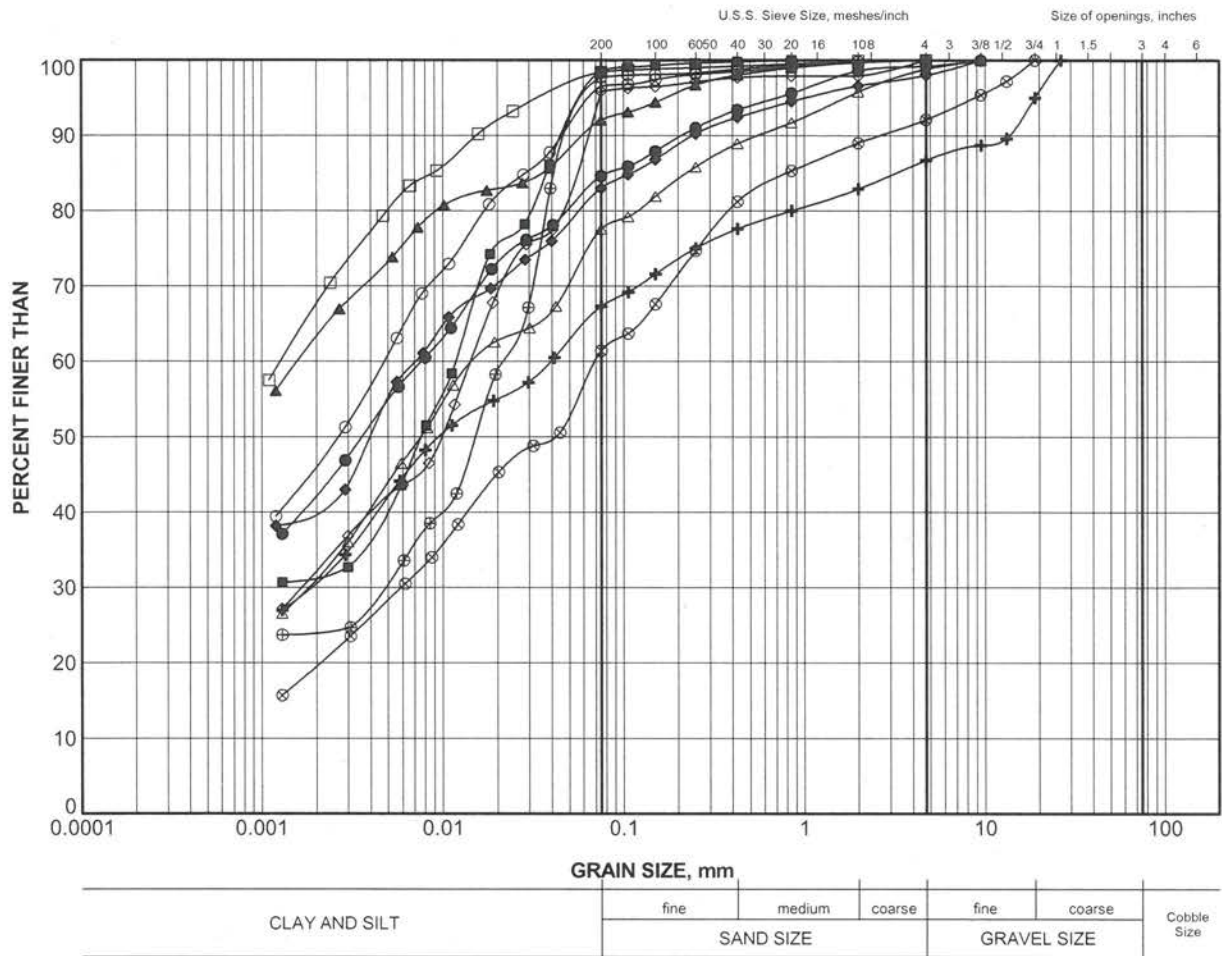
TITLE

GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY



PROJECT No.	09-1132-0039	FILE No.	0911320039-1000-R020E3
DRAWN	WDF	Nov 19/09	SCALE N/A REV.
CHECK	4/13 Apr/10		

FIGURE E-3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GBH-178	14	157.2
■	GBH-184	4	174.3
▲	GBH-184	7	170.1
+	GBH-184	11	164.0
◆	GBH-184	14	156.4
◇	GBH-185	3	175.0
○	GBH-185	8	168.6
△	GBH-185	13	159.5
⊗	GBH-185	15	153.4
⊕	GBH-191	3	174.2
□	GBH-191	5	171.9

PROJECT

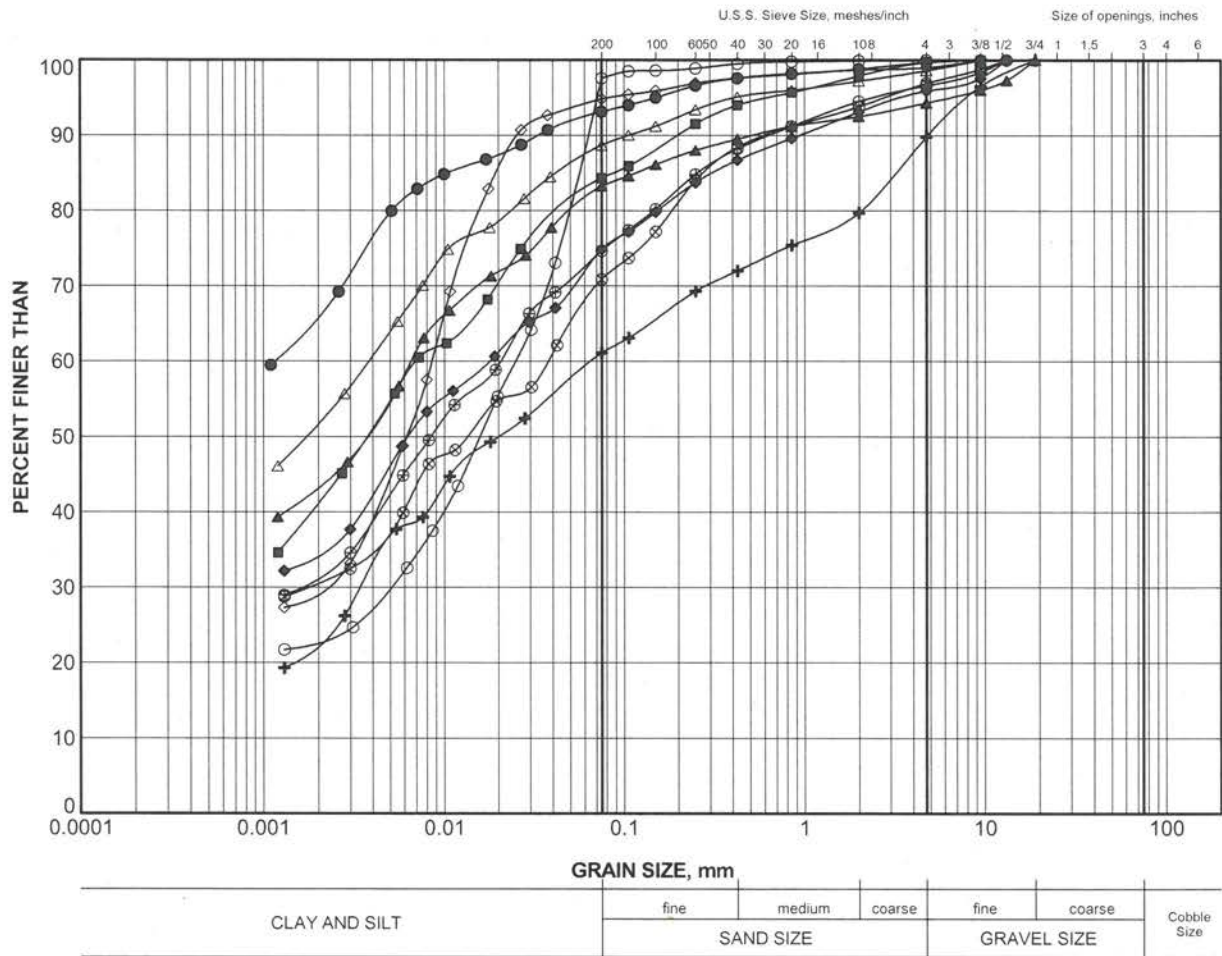
GEOTECHNICAL DATA REPORT
CANADIAN INSPECTION PLAZA
AND RELATED INFRASTRUCTURE
WINDSOR, ONTARIO

TITLE

GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY



PROJECT No.	09-1132-0039	FILE No.	0911320039-1000-R020E4
DRAWN	WDF	Nov 19/09	SCALE N/A REV.
CHECK	SXB	Apr 1/10	FIGURE E-4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	GBH-191	6	170.9
■	GBH-191	9	166.3
▲	GBH-191	10	164.8
+	GBH-191	12	161.7
◆	GBH-191	13	158.7
◇	GBH-191	15	152.6
○	GBH-193	3	176.3
△	GBH-193	6	174.1
⊗	GBH-193	9	170.1
⊕	GBH-193	13	164.0

PROJECT

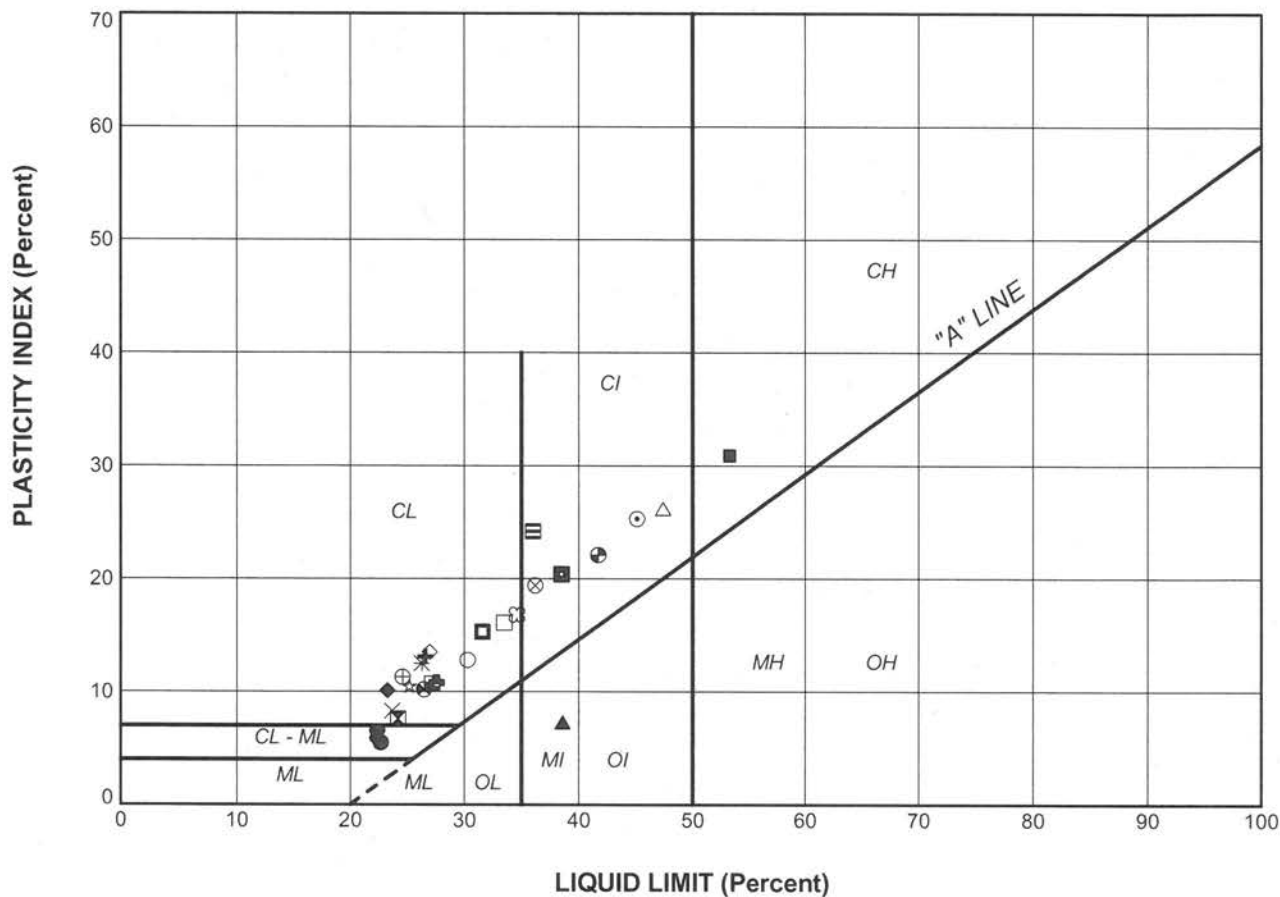
GEOTECHNICAL DATA REPORT
CANADIAN INSPECTION PLAZA
AND RELATED INFRASTRUCTURE
WINDSOR, ONTARIO

TITLE

GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY




PROJECT No.	09-1132-0039	FILE No.	0911320039-1000-R020E5
DRAWN	WDF	Nov 19/09	SCALE N/A REV.
CHECK	SJB	Apr 1/10	FIGURE E-5

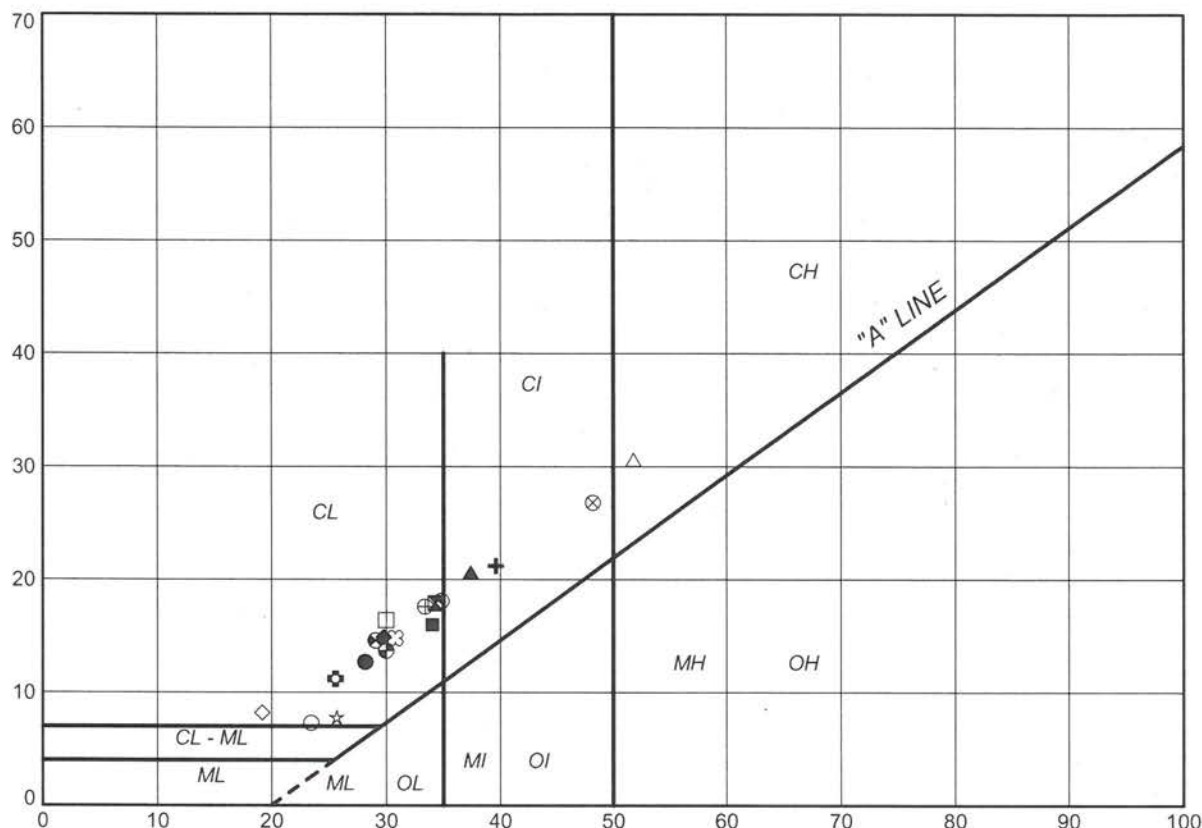


LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	GBH-167	4	22.7	17.2	5.5
■	GBH-167	5	53.3	22.4	30.9
▲	GBH-167	7	38.6	31.3	7.3
+	GBH-167	9	26.6	13.5	13.1
◆	GBH-167	11	23.3	13.2	10.1
◇	GBH-167	12	27.0	13.5	13.5
○	GBH-167	14	30.3	17.5	12.8
△	GBH-170	6	47.4	21.2	26.2
⊗	GBH-170	9	36.2	16.8	19.4
⊕	GBH-170	12	24.6	13.3	11.3
□	GBH-170	14	33.5	17.4	16.1
⊙	GBH-172	3	26.5	16.3	10.2
⊛	GBH-172	6	41.7	19.6	22.1
☆	GBH-172	11	25.2	14.7	10.5
⊠	GBH-172	14	34.6	17.8	16.8
⊡	GBH-176	2	24.2	16.6	7.6
⊙	GBH-176	5	45.1	19.8	25.3
⊛	GBH-176	10	27.6	16.8	10.8
×	GBH-176	13	23.7	15.4	8.3
■	GBH-178	4	22.4	16.2	6.2
■	GBH-178	8	38.5	18.1	20.4
⊛	GBH-178	11	26.3	13.8	12.5
□	GBH-178	14	31.6	16.3	15.3
⊠	GBH-184	4	27.2	16.5	10.7
⊡	GBH-184	7	36.0	11.8	24.2

PROJECT				GEOTECHNICAL DATA REPORT CANADIAN INSPECTION PLAZA AND RELATED INFRASTRUCTURE WINDSOR, ONTARIO			
TITLE				PLASTICITY CHART			
PROJECT No.		09-1132-0039		FILE No.		0911320039-1000-R020E6	
DRAWN		WDF		SCALE		N/A	
CHECK		583		REV.		110	
 Golder Associates LONDON, ONTARIO				FIGURE E-6			

PLASTICITY INDEX (Percent)




LIQUID LIMIT (Percent)

SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	GBH-184	11	28.2	15.5	12.7
■	GBH-184	14	34.0	18.0	16.0
▲	GBH-185	3	37.4	16.8	20.6
+	GBH-185	8	39.6	18.4	21.2
◆	GBH-185	13	29.8	14.9	14.9
◇	GBH-185	15	19.2	11.0	8.2
○	GBH-191	3	23.5	16.2	7.3
△	GBH-191	5	51.8	21.2	30.6
⊗	GBH-191	6	48.2	21.4	26.8
⊕	GBH-191	9	33.4	15.8	17.6
□	GBH-191	10	30.0	13.6	16.4
⊙	GBH-191	12	29.1	14.5	14.6
⊛	GBH-191	13	30.0	16.3	13.7
☆	GBH-191	15	25.7	17.9	7.8
⊞	GBH-193	3	30.8	16.0	14.8
⊠	GBH-193	6	34.3	16.4	17.9
⊡	GBH-193	9	34.8	16.7	18.1
⊣	GBH-193	13	25.6	14.4	11.2

PROJECT		GEOTECHNICAL DATA REPORT CANADIAN INSPECTION PLAZA AND RELATED INFRASTRUCTURE WINDSOR, ONTARIO	
TITLE		PLASTICITY CHART	
PROJECT No.	09-1132-0039	FILE No.	0911320039-1000-R020E7
DRAWN	WDF	Nov 12/09	SCALE N/A
CHECK	SSB	Apr 1/10	REV.
 Golder Associates LONDON, ONTARIO		FIGURE E-7	



APPENDIX G

Results of Analytical Laboratory Testing

Attention:Dirka U. Prout

Golder Associates Ltd
309 Exeter Rd
Unit 1
London, ON
N6L 1C1

Report Date: 2015/12/11

Report #: R3802879

Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B5P0837

Received: 2015/12/07, 09:10

Sample Matrix: Soil
Samples Received: 1

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	1	N/A	2015/12/10	CAM SOP-00463	EPA 325.2 m
Conductivity	1	N/A	2015/12/10	CAM SOP-00414	OMOE E3138 v2 m
pH CaCl2 EXTRACT	1	2015/12/10	2015/12/10	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	1	2015/12/07	2015/12/10	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	1	N/A	2015/12/10	CAM SOP-00464	EPA 375.4 m
Redox Potential (1)	1	2015/12/08	2015/12/09	SLA SOP-00101	In house

Remarks:

Maxxam Analytics has performed all analytical testing herein in accordance with ISO 17025 and the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act. All methodologies comply with this document and are validated for use in the laboratory. The methods and techniques employed in this analysis conform to the performance criteria (detection limits, accuracy and precision) as outlined in the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act.

Maxxam Analytics is accredited for all specific parameters as required by Ontario Regulation 153/04. Maxxam Analytics is limited in liability to the actual cost of analysis unless otherwise agreed in writing. There is no other warranty expressed or implied. Samples will be retained at Maxxam Analytics for three weeks from receipt of data or as per contract.

Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.

* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

(1) This test was performed by Maxxam Sladeview Petrochemical

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Sara Singh, B.Sc, Senior Project Manager

Email: sarasingh@maxxam.ca

Phone# (905)817-5730

=====

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

SOIL CORROSIVITY PACKAGE (SOIL)

Maxxam ID		BMB537	BMB537		
Sampling Date		2015/11/16 14:00	2015/11/16 14:00		
COC Number		541628-01-01	541628-01-01		
	UNITS	BH101-SA6 15-16½'	BH101-SA6 15-16½' Lab-Dup	RDL	QC Batch
Calculated Parameters					
Resistivity	ohm-cm	840			4301801
Inorganics					
Soluble (20:1) Chloride (Cl)	ug/g	81	79	20	4306895
Conductivity	umho/cm	1180		2	4307035
Available (CaCl2) pH	pH	7.75		N/A	4305737
Soluble (20:1) Sulphate (SO4)	ug/g	1100		40	4306896
Subcontracted Analysis					
Redox Potential	mV	+227	+226		4304139
RDL = Reportable Detection Limit QC Batch = Quality Control Batch Lab-Dup = Laboratory Initiated Duplicate					

TEST SUMMARY

Maxxam ID: BMB537
Sample ID: BH101-SA6 15-16½'
Matrix: Soil

Collected: 2015/11/16
Shipped:
Received: 2015/12/07

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4306895	N/A	2015/12/10	Alina Dobreanu
Conductivity	AT	4307035	N/A	2015/12/10	Lemeneh Addis
pH CaCl2 EXTRACT	AT	4305737	2015/12/10	2015/12/10	Neil Dassanayake
Resistivity of Soil		4301801	2015/12/10	2015/12/10	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	4306896	N/A	2015/12/10	Alina Dobreanu
Redox Potential	PH	4304139	2015/12/08	2015/12/09	Grace Sison

Maxxam ID: BMB537 Dup
Sample ID: BH101-SA6 15-16½'
Matrix: Soil

Collected: 2015/11/16
Shipped:
Received: 2015/12/07

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	4306895	N/A	2015/12/10	Alina Dobreanu
Redox Potential	PH	4304139	2015/12/08	2015/12/09	Grace Sison

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	14.0°C
-----------	--------

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD		QC Standard	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits	% Recovery	QC Limits
4304139	Redox Potential	2015/12/09					+104	mV	0.44	20	+244	238 - 248
4305737	Available (CaCl ₂) pH	2015/12/10			99	97 - 103			0.73	N/A		
4306895	Soluble (20:1) Chloride (Cl)	2015/12/10	NC	70 - 130	111	70 - 130	<20	ug/g	NC	35		
4306896	Soluble (20:1) Sulphate (SO ₄)	2015/12/10	NC	70 - 130	102	70 - 130	<20	ug/g	NC	35		
4307035	Conductivity	2015/12/10			100	90 - 110	<2	umho/cm	0.67	10		

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

QC Standard: A sample of known concentration prepared by an external agency under stringent conditions. Used as an independent check of method accuracy.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spiked amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than 2x that of the native sample concentration).

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (one or both samples < 5x RDL).

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Cristina Carriere

Cristina Carriere, Scientific Services



Grace Sison, B.Sc., C.Chem, Senior Project Manager - Petroleum Division

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.



APPENDIX H

Summary of Geotechnical Parameters - Settlement Analyses



CANADIAN INSPECTION PLAZA AND PERIMETER ACCESS ROAD

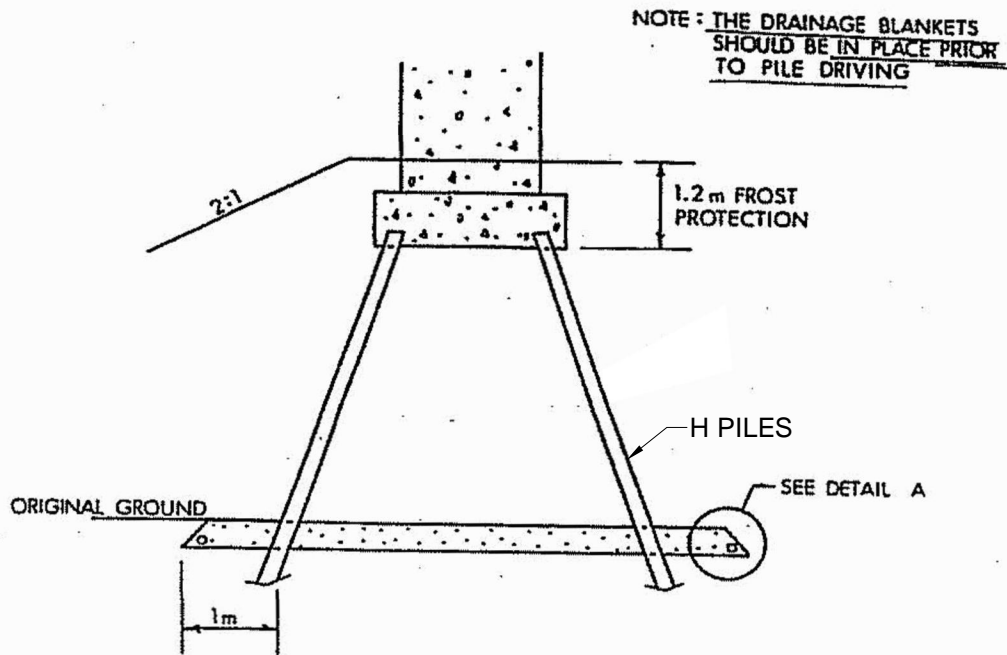
Table 5: Summary of Geotechnical Parameter Values for Settlement Analyses, Geologic Area 1 (Bridge B1 Approach)

Layer	Elevation (m)		Thickness (m)	W_n (%)	γ_t (kN/m ³)	C_c	C_r	C_α	e_o	σ'_p (kPa)	c_v (m ² /day)	c_h/c_v
	From	To										
1	176.7	176.0	0.7	23	20.3	0.168	0.018	0.0047	0.633	800	0.00864	2
2	176.0	175.0	1.0	33	18.8	0.285	0.031	0.0080	0.908	360	0.00864	2
3	175.0	174.5	0.5	33	18.8	0.285	0.031	0.0080	0.908	330	0.00864	2
4	174.5	174.0	0.5	33	18.8	0.285	0.031	0.0080	0.908	230	0.00864	2
5	174.0	173.0	1.0	33	18.8	0.285	0.031	0.0080	0.908	160	0.00864	2
6	173.0	172.3	0.7	36	18.4	0.320	0.035	0.0090	0.990	160	0.00864	2
7	172.3	170.5	1.8	36	18.4	0.320	0.035	0.0090	0.990	140	0.00864	2
8	170.5	170.0	0.5	41	17.9	0.378	0.042	0.0106	1.128	140	0.00864	2
9	170.0	169.5	0.5	41	17.9	0.378	0.042	0.0106	1.128	137	0.00864	2
10	169.5	167.8	1.7	34	18.7	0.296	0.033	0.0083	0.935	140	0.00864	2
11	167.8	166.2	1.6	29	19.4	0.238	0.026	0.0067	0.798	148	0.00864	2
12	166.2	165.0	1.2	22	20.5	0.156	0.017	0.0044	0.605	159	0.00864	2
13	165.0	164.0	1.0	21	20.7	0.144	0.016	0.0040	0.578	155	0.00432	1
14	164.0	163.0	1.0	21	20.7	0.144	0.016	0.0040	0.578	190	0.00432	1
15	163.0	162.0	1.0	21	20.7	0.144	0.016	0.0040	0.578	240	0.00432	1
16	162.0	161.0	1.0	21	20.7	0.144	0.016	0.0040	0.578	330	0.00432	1
17	161.0	160.0	1.0	21	20.7	0.144	0.016	0.0040	0.578	410	0.00432	1
18	160.0	158.5	1.5	26	19.8	0.203	0.022	0.0057	0.715	350	0.00432	1
19	158.5	156.5	2.0	26	19.8	0.203	0.022	0.0057	0.715	240	0.00432	1

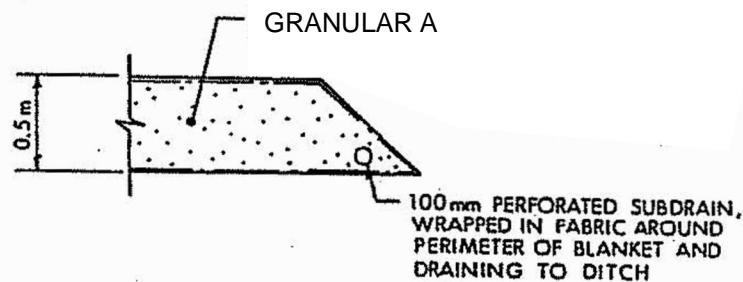


APPENDIX I

Drainage Blanket Details



ABUTMENT SECTION (TYP)



DETAIL A.

DRAINAGE BLANKET DETAILS

NOTES

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

ALL LOCATIONS ARE APPROXIMATE ONLY.

PLAN NOT TO SCALE.

REFERENCE

SCHEMATIC BASED ON DRAWING "DRAINAGE BLANKET DETAILS FOR ABUTMENTS AND PIERS" PROVIDED BY MTO FOUNDATIONS OFFICE.

PROJECT

OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2
HIGHWAY 401 (RHHGP)
GWP 3028-14-00

TITLE

DRAINAGE BLANKET DETAILS



**Golder
Associates**

PROJECT No.13-1132-0053

FILE No.1311320053-1000-F01001

CADD DCH May 20/16

SCALE AS SHOWN REV. 0

CHECK

FIGURE I-1



APPENDIX J

Special Provisions

CELLULAR CONCRETE – Item No.

Special Provision

1.0 SCOPE

This specification covers the requirements for the supply and placement of lightweight cellular concrete used as embankment fill in accordance with the contract drawings. The cellular concrete shall be placed in dry conditions and above the groundwater table.

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS.PROV 539	Temporary Protection System

Ontario Provincial Standard Specifications, Material

OPSS 1301	Cementing Materials
OPSS 1302	Water
OPSS.PROV 1303	Admixtures for Concrete
OPSS.PROV 1350	Concrete – Materials and Production

American Society for Testing and Materials (ASTM)

ASTM C 150	Portland Cement
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	Standard Test Method for Compressive Strength of Lightweight Insulating Concrete

Ministry of Transportation Publication:

LS-407	Method of Test for Compressive Strength of Moulded Cylinders
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3.0 DEFINITIONS

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

Cellular Concrete: Cellular concrete is a material with flowable consistency during placement, produced by the substitution of a uniform cellular structure of air cells (voids) for some or all of the aggregate particles found in standard concretes.

CELLULAR CONCRETE – Item No.

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

Quality Verification Engineer: 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 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Prequalification of Cellular Concrete Product

Prior to the commencement of work, the Contractor shall submit to the Contract Administrator a statement from the Supplier verifying that the Supplier has successfully put the product through the MTO Prequalification Process for Lightweight Fill and confirming that the product has been prequalified for use as lightweight fill by the MTO.

4.2 Qualifications

The Contractor shall submit a resume of the contractor's experience in the production and placement of cellular concrete. The resume shall include the qualifications of contractor's superintendent and/or foreman. The resume shall be submitted to the Contract Administrator for information purposes a minimum of three weeks prior to the start of cellular concrete construction.

The Contractor shall have satisfactorily completed at least five (5) projects of similar nature and complexity during the last three (3) years.

Workers, including the contractor's superintendent and/or foreman, shall be fully qualified and thoroughly trained and experienced in the production and placement of cellular concrete.

At the commencement of the work, the Contractor shall have on site 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.

4.3 Submission of Shop Drawings

At least six 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.

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The contractor shall submit full details of the following:

- a) The method of foundation excavation and preparation.
- b) The 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 on a layer by layer basis.
- d) The method for obtaining the required surface slope at the top of the cellular concrete mass.
- e) The method of protecting the top cellular concrete surface from damage during pavement structure placement and compaction.
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

Production and placement of cellular concrete in the RSS application for this contract shall not commence until the MTO RSS Committee has received, reviewed and approved the shop drawings.

4.4 Submission of Certificate of Conformance

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of the cellular concrete work and prior to any backfilling. The Certificate shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

4.5 Submission of Environmental Protection Strategy

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of an environmental protection strategy as specified under Section 7.5.

5.0 MATERIALS

5.1 Cementing Materials

Cementing materials shall be according to OPSS 1301. Supplementary cementing materials shall not be used.

5.2 Water

Water shall be free of contamination and any deleterious substance. Water shall conform to OPSS 1302.

5.3 Admixtures

Admixtures shall conform to OPSS.PROV 1303.

5.4 Foaming Agents

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.

5.5 Cellular Concrete Properties

Cellular concrete shall be prequalified by the MTO Lightweight Fill Committee Prequalification Process and have the following properties:

- a) Minimum unconfined compressive strength at 28 days of 1 MPa.
- b) Wet cast density of 510 kg/m³ (5.0 kN/m³) (+/-5%)
- c) Must not contain any fly ash or any other waste or process by-product.

6.0 EQUIPMENT

The specialized batching, mixing, and placing equipment shall be automated and certified for the purpose by the manufacturer of the cellular concrete material. Dry-mix 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 1,000 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.

7.0 CONSTRUCTION

7.1 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.

The prepared subgrade shall be good competent level ground. Snow and ice must be removed from the area prior to placement.

7.2 Dewatering

The prepared subgrade shall be free of standing water during placement of cellular concrete and until backfill is placed on top of the cellular concrete. If necessary, dewatering shall be continuous during placement of materials.

Dewatering shall be according to OPSS 517.

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7.3 Protection System

The construction of all protection schemes shall be according to OPSS.PROV 539 and paid for under the appropriate tender item. Where the stability, safety or function of an existing roadway, railway, other works, or proposed works may be impaired due to the method of operation, such protection as may be required shall be provided.

7.4 Cellular Concrete Placement

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.

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.

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. Cold weather protection shall be provided in accordance with OPSS.PROV 1350.

If temperatures above 38°C are expected during casting, special precautions shall be considered including casting before dawn and use of additives to maintain moisture in the mixture and minimize plastic-shrinkage cracking.

Cellular concrete must not be mixed or placed during precipitation events (rain or snow).

The Contractor shall consult the foaming agent manufacturer for specific hot or cold weather placement recommendations.

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.

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.

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%, if required, shall be created in accordance with the contractor's approved method statement and shop drawings.

CELLULAR CONCRETE – Item No.

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. Backfilling can commence when cellular concrete supports foot traffic without leaving an indentation.

7.5 Environmental Protection

The Contractor shall handle materials and conduct the work in a manner that will ensure protection of the natural environment and prohibit cellular concrete from entering surface or ground water. The Contractor shall take measures as necessary to prevent the material from entering the natural environment and/or leaking outside of the intended placement location, and shall have established methods for stopping flow of the product as required, and for prompt remediation of any leaks or spills. These measures and any other contingency planning requirements shall be documented in an Environmental Protection Strategy.

8.0 QUALITY CONTROL

8.1 Sampling Frequency and Methods

The fresh cellular concrete shall be collected for density testing once per production run, or once for every 25 cubic metres, or once per 15 minutes, whichever is more frequent.

Cellular concrete samples shall be captured, cured, and tested at the point of placement to verify the specified compressive strength and the dry unit weight. The unit weight shall be maintained within +/- 5% of the design unit weight and shall be adjusted as required to obtain the specified density at the point of placement. One sample should be taken for each placement, or every 100 m³, whichever is more frequent. One sample is comprised of one set of four cellular concrete cylinders, with cylinders cast in 75 mm by 150 mm cylindrical plastic molds. Cellular concrete cylinders shall be cured and tested for compressive strength as per ASTM C495 and LS 407.

8.2 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.

9.0 MEASUREMENT FOR PAYMENT

Measurement will be Plan Quantity as may be revised by adjusted Plan Quantity of the cellular concrete in cubic metres.

10.0 PAYMENT

10.1 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.

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.

CONCRETE PAD – Item No.

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.

EARTH EXCAVATION AND GRADING - Item No.

Non Standard Special Provision

General

OPSS.PROV 206 shall apply and govern except as amended or extended herein.

Scope of Work

The Contractor shall excavate for the fill placement area, including grading and shaping the proposed subgrade. The Contractor shall make his own arrangements for the disposal of excess excavated materials.

The Contractor shall note that existing utilities could be located within the excavation area. The Contractor shall protect these utilities from damage. No additional payment will be considered for working around these utilities, unless otherwise specified herein.

Contractor is required to proof-roll the subgrade to determine the presence of any areas of unsuitable material.

Protection of the Subgrade

The Contractor shall take all necessary precautions required to protect the subgrade, including the following:

- Only tracked equipment will be permitted to drive on the subgrade; and
- No trucks will be allowed to drive on the subgrade.

206.07.05.01.01 General and subsections: Boulders, cobbles and fragments of rock, RAP, and RCM over 150 mm in their maximum dimension shall not be incorporated into any grading fill or earth embankments regardless of lift thickness.

Measurement

No measurement shall be made for excavation and grading.

Subexcavation below the theoretical subgrade as directed by the Engineer shall be measured in cubic metres and is a Provisional Item.

Payment

Payment shall be made at the Unit Price and Lump Sum Price bids in the Form of Tender and shall be compensation in full for all labour, equipment and materials required to complete this work, including disposal of excess excavated material.

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.

PILE DRIVING – VIBRATIONS - Item No.

Non Standard Special Provision

SCOPE

This non-standard special provision is intended to establish controls for pile driving in the interest of protection of nearby structures, property, and soils that remain in place. Multiple utilities are located adjacent to the abutments and piles of Bridge B-1 which will be pile supported. All of the Contractor's responsibilities apply equally to any Subcontractor involved in pile driving activities. Pile driving shall be conducted according to the operational restrictions as outlined in the Contract Specifications and Drawings.

REFERENCES

OPSS.PROV 903 Construction Specification For Deep Foundations

CSA S6-14 Canadian Highway Bridge Design Code

NCHRP Synthesis 253 Dynamic Effects of Pile Installations on Adjacent Structures

DEFINITIONS

Frequency means the number of oscillations that occur in one second during a vibration event. The frequency units given are in hertz (cycles per second).

Limiting Particle Velocity means the maximum vibration level not to be exceeded in order to prevent damage.

Peak particle velocity means the maximum rate of change of position with respect to time, measured on the ground. The velocity amplitudes are given in units of millimetres per second (mm/s), zero to peak amplitude.

Quality Verification Engineer (QVE) means an Engineer with a minimum of five (5) years of experience in the field of installation of piling and vibration monitoring or alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to this Contract. The QVE shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue Certificates of Conformance.

Scaled distance is equal to the distance from the pile driving to a buried utility or other target, measured along the path travelled by the vibrations, divided by the square root of the energy expended in each blow of the pile driving or each cycle of the vibratory pile driving. Common units are metres (m), and Newton-metres (N-m).

PILE DRIVING – VIBRATIONS - Item No.

DESIGN AND SUBMISSION REQUIREMENTS

No less than three weeks prior to commencing pile driving operations, or at the preconstruction meeting (whichever is earliest), or at any time the Contractor proposed to change the driving method, the Contractor shall submit a driving plan to Contract Administrator for review. The Driving Plan shall contain:

- a) All information required under the general piling specifications, OPSS.PROV 903, and
- b) All information relative to vibrations and vibration controls, as described in the following sections.

MATERIALS – Not Used

EQUIPMENT

Pile Driving Equipment

Two types of pile drivers can be used: impact or vibratory hammers. The Contractor shall be aware of the fact that ground vibrations induced by these machines are of different nature, and therefore utmost care shall be taken in the selection of the equipment and driving method.

The weight and length of the hammer shall be minimized to the extent possible to prevent development of excess vibrations yet permit proper installation of the driven piles.

Use of proper pile cushioning can be effective in reducing excess vibrations. Where helmets are used to protect the head of steel piles, the cushioning material shall be kept fresh and resilient throughout the pile driving operations.

Seismographs

Vibrations in the form of particle velocities shall be monitored using a seismograph. The instrument is to be programmed to record peak ground levels in strip chart mode on a continuous basis providing peak levels at 15 second intervals, while also recording waveform data upon receipt of a minimum threshold ground vibration level of 1.5 mm/s.

CONSTRUCTION

Pile Driving Requirements and Restrictions

Loose, saturated native granular deposits are located across the project site. These materials may be prone to settlement due to ground vibration. Several utilities are fully or partially located in these materials. Pre-drilling of piles as indicated in the Contract Documents is required to reduce vibration related settlement.

The stroke shall be minimized to the extent possible to prevent development of excess vibrations yet permit proper installation of the driven piles. This will be critical when driving through zones with cobbles and boulders and when the pile tip is approaching the bedrock surface.

Installation of piles in each group shall start at points farthest from existing utilities or structures then continue in a direction towards such utilities.

PILE DRIVING – VIBRATIONS - Item No.

Earth moving activities, particularly those that involve use of vibratory rollers or packers, shall not occur in the same time period as pile driving operations.

Particle Velocity Control

Pile driving shall be controlled by limiting ground particle velocity. Peak particle velocity shall be the measure of the level of vibration, and it should be measured with the instrumentation and methods described in this special provision and the Contract Documents. The peak particle velocity shall be controlled using a maximum peak velocity independent of frequency.

The peak particle velocity shall be less than a specific control limit at the nearest structure. The type of structure and distance between this structure and the nearest pile will dictate the allowable value as described in Table 1. Particle velocity shall be recorded in three mutually perpendicular axes. The maximum allowable peak velocity shall be that measured on any of the three axes.

Table 1 Limiting Particle Velocity

Structure	Limiting Particle Velocity (mm/sec)
Concrete and grout <72 hours after placement (OPSS.PROV 120)	10
All buried utilities	50
Essex Terminal Railroad tracks and all other existing structures	100

The Contract Administrator reserves the right to adjust the values designated in the paragraphs above, if, in his or her opinion, the pile driving procedures being used are damaging the adjacent structures or soils.

Review Level

If the Contractor exceeds 80% of the ground vibration control limit for any single axis during a pile driving operation. A notice should be provided to the Contractor.

Alert Level

If the Contractor exceeds 100% of the ground vibration control limit for any single axis during a pile driving operation, pile driving activities shall cease and the Contractor shall submit a written report to the Contract Administrator. This report shall provide driving and vibration data and include the proposed corrective action for the next pile to be driven to avoid exceeding the specified limit. The next pile shall not be driven until the Contract Administrator acknowledges, in writing, a driving process change has been approved.

Monitoring of Vibrations

Real-time vibration monitoring shall be carried out when pile driving is occurring at separation distances of 5 m or less from a utility or structure. This requirement may be revised by the Contract Administrator, subject to the findings of the Validation/Test Program or the Contractor's performance with respect to vibration control.

The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment or pier) starting with the pile furthest away from the existing structure. As a minimum, readings should be taken and recorded during the first 3 m of driving and during seating of the pile onto the bedrock.

PILE DRIVING – VIBRATIONS - Item No.

The results shall be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specification. The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location set criteria and driving log must be submitted with vibration monitoring results.

If the results do not exceed the Alert or Review Levels, they shall be deemed acceptable and the Contractor may continue with the remaining piles with readings taken during driving of each pile. Subsequent vibration readings should be taken for each pile during bedrock seating. The results of the subsequent piles should be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specifications. The results shall be submitted to the Contract Administrator at the end of each day.

If the readings are not within the limits stated above, the Contractor must cease pile driving operations and follow the process described for Alert Level.

Recorded Data

All three components (longitudinal, transverse, vertical) of particle velocity will be measured on the ground at the location of the nearest and other strategic structures and/or at any locations the Contract Administrator deems necessary for any particular pile driving operations. These measurements shall be made on the ground adjacent to these structures as the pile driving is going on.

The Contractor shall maintain a Pile Driving Log and shall submit daily reports to the engineer on piles driven and vibrations measured. These logs shall be in the form specified in the Driving Plan.

Instrumentation

The Contractor shall provide the instrumentation agreed to in the pile driving plan to monitor the pile driving vibrations and permanent deformation of the strategic structures.

Transducer Attachment (Coupling)

In most cases the transducers will be placed on the ground surface. In this case the transducer is to be spiked, sandbagged or buried to ensure proper coupling. It may be necessary, or where directed by the Contract Administrator, to place the seismograph on a measuring location other than earthen materials. When the measurement surface consists of rock, steel (or other metal), or concrete, the transducers shall be bolted to the measurement surface or bonded with high strength adhesive. On other surface the mass of the seismograph and/or transducer package may be sufficient for good coupling. For significant accelerations (greater than 1.0g), adhesive bolts shall be used on all solid surfaces. All transducers on vertical surfaces shall be bolted in place.

Number and Location

The number of instruments required is dependent on the specific site. However there shall be, as a minimum, two monitors. One monitor will be used on site, while the second is held in reserve or used at a specific complaint or potential complaint site.

PILE DRIVING – VIBRATIONS - Item No.

Record Keeping

The Contractor will provide the Contract Administrator with all data necessary for record-keeping purposes. These data shall include, as a minimum, the following information:

- All surveys conducted for vibration control purposes.
- The original driving plan, as well as any adjustments made to it during the course of the construction activities.
- All monitored data, relative to each and every pile installed. These driving records shall contain all information as required and approved in the pile driving plan, including all information concerning the type and characteristics of the monitoring instruments used, their locations and orientations.
- All driving records correlated with monitored data.
- All weather conditions occurring during the driving activities.

Validation/Test Pile Program

Definition/Responsible Party

Analyses for ground vibration attenuation for this project were conducted with empirical formulae. Site-specific data obtained through a program of vibration monitoring during pile installation is required to validate estimates of the Peak Particle Velocity for the ground attenuation.

The Contractor shall provide any necessary cooperation with the Contract Administrator for conducting a test pile program. While the Contract Administrator, and a Vibration Specialist retained by the Contract Administrator, will take the lead role in this program, the Contractor shall concur in the intent, design, and process of the testing. This program shall be performed on the first pile driven for each pile group (i.e. at each corner of the abutment or pier), starting with the pile furthest away from the existing structure. It shall be conducted to show how the vibrations decrease with increasing wave travel path distances from the pile and vary with the type of pile used. This program is intended to provide subsequent guidance for the choice of pile placement technique for this particular project, and not to define any envelope or relationship to be used as a control.

Monitoring

The number, type and location of the seismographs used to monitor the test pile program shall be determined by the Contract Administrator and the Vibration Specialist.

Analyses

Statistical analysis of the test data will be performed by the Vibration Specialist. The results of these analyses will be transmitted to the Contractor within three weeks after completion of the test pile program.

QUALITY ASSURANCE – Not Used

MEASUREMENT OF PAYMENT – Not used

BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material required to do the work.

SETTLEMENT MONITORING – Item No.

Non Standard Special Provision

SCOPE

This non standard special provision contains the requirements for the labour, materials, equipment and professional services for a geotechnical instrumentation program that is to include:

- Settlement Plates (SP) and
- Vibrating Wire Piezometers (VWP).

The purpose of this Monitoring Program is to monitor settlements and pore water pressures in the foundation soils at select locations during construction of the preload and final approach embankments at Highway 401 Bridge Sites 6-600/1 and 6-600/2 collectively known as Bridge B-1. This information is required for the owner's documentation and subsequent analysis purposes to assist in placement of the permanent lightweight backfill during construction of the abutments.

An independent specialist Foundation Engineering consultant will be retained by the Owner to supervise geotechnical instrument installation and collect and transmit resulting data. A land surveying firm licensed in Ontario shall be retained to complete all surveying services under this specification.

Monitoring of existing VWPs installed in exploratory boreholes shall also be undertaken as part of this specification for all functional instrumentation in existing locations. A record of testing of individual existing VWP and readings shall be submitted as part of the monitoring plan required by this specification.

Review of monitoring records collected by the Windsor-Detroit Bridge Authority (WDBA) for the existing portion of the west embankment and those collected by the Windsor Essex Mobility Group (WEMG) for the existing portions of the east embankment shall be undertaken as part of this specification. In particular the review shall focus on settlement monitoring instrumentation located within 100 metres of each abutment. Review of the performance of the existing embankment sections is critical for planning preload durations, the timing of preload and permanent fill placement and determining the magnitude and extent of any lightweight fill placement.

The monitoring program specified herein shall not relieve the Contractor of its responsibility for undertaking whatever actions are required including installation of additional instrumentation and independent reading of instrumentation to ensure that the Work proceeds in a safe, stable and secure manner and in conformance with the requirements of the Contract Documents.

REFERENCES

[not used]

SETTLEMENT MONITORING – Item No.

DEFINITIONS

Alert Level: Value of instrumentation readings at which the Contract Administrator can order the Contractor to cease construction operations, make Site and affected properties secure, and take necessary and agreed upon measures to mitigate unacceptable movements and assure the safety of the Work and the public. The Alert Level for each instrument represents the maximum permissible ground movement, structure movement, maximum vibration levels, maximum load or pore water pressure reading associated with construction of the Work.

Baseline Reading: Initial readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements and changes in stress or pressure. The Contractor shall not undertake any construction in the area of the instrument until the baseline reading is established and agreed with the Contract Administrator. The baseline reading for each instrument shall not be obtained without the presence of the Contract Administrator's qualified representative.

Calibration: Means to program or set an instrument to measure accurate and precise data.

Review Level: Value of instrumentation readings at which the Contract Administrator and the Contractor jointly assess the necessity of altering the method, rate or sequence of construction. Unless stated otherwise within this specification, the Review Level shall be taken as 80 per cent of the Alert Level.

Response Levels: Review Level and Alert Level values as defined above and specified herein. Response Levels encompass the cumulative movement, deformation, stress, strain, and water pressure changes from the Baseline Readings.

DESIGN AND SUBMISSION REQUIREMENTS

Monitoring Plan

As deemed necessary, the Contractor shall prepare and submit a Foundation Monitoring Plan, in compliance with the requirements of this specification, to control the safety and stability of their work. The monitoring plan shall be submitted a minimum of three weeks prior to installation of any instrument. The monitoring plan shall indicate:

- a) Experience and qualifications of specialist staff, the role of each in executing the plan and reporting relationships.
- b) Sequence of filling and grading operations.
- c) Locations and depths of all instruments including surveyed northing, easting and elevation coordinates.
- d) Include a plan layout of all instruments provided in AutoCAD or compatible electronic file format indicating unique monitoring station and instrument identification numbers.
- e) A record of testing of individual existing VWP installed in existing boreholes and readings shall be submitted as part of the monitoring plan and data reports required by this specification.
- f) Define Alert and Review Levels for each VWP. The Alert Level for each VWP shall be established as the baseline pore water pressure plus 0.9 times the total vertical stress of new grading and surcharge fill placed above the finished wick drain drainage blanket surface elevation at the monitoring station location. The Alert Level shall be defined for each 1.0 m of fill to be placed at the monitoring station location.
- g) Sequence of instrument installation.
- h) Methods of and equipment for instrument installation and data collection.
- i) Draft data reporting documents and example electronic file formats.

SETTLEMENT MONITORING – Item No.

- j) Catalogue data and manufacturer's calibration records for each instrument demonstrating compliance of instruments with specifications.
- k) Grout mixes in accordance with vibrating wire piezometer manufacturer recommendations.

Data Reports

Monitoring data shall be reported to the Contract Administrator within five (5) working days after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Data Reports and these reports shall include:

- a) A plot of each settlement plate (SP) vertical displacement relative to the baseline reading (i.e., settlement or heave) versus time. This plot shall also include a graph illustrating the fill height within 25 m of the instrument. These data shall be plotted together on two separate graphs with the horizontal axes as follows:
 - a. linear by date
 - b. logarithmic time in days based on elapsed time since baseline reading.
- b) A plot of each vibrating wire piezometer (VWP) pore water pressure versus time. This plot shall also include a graph illustrating the fill height within 25 m of the instrument. These data shall be plotted together on two separate graphs with the horizontal axes as follows:
 - a. linear by date
 - b. logarithmic time in days based on elapsed time since baseline reading.
- c) The use of automatic plot scales shall be limited to the degree practical. The horizontal axis of logarithmic plots shall include at least two orders of magnitude (i.e., 1 to 1,000 days). The vertical axes of settlement and pore-water pressure plots shall be selected to be equal among all graphs to allow rapid visual comparison among data sets.
- d) Alert levels shall be included on all VWP data plots for comparison.
- e) Survey data shall be reported in millimetre format (not decimal metres) to the nearest millimetre.
- f) Fill elevation data shall be reported to the nearest 0.1 m.
- g) Pore water pressure data shall be reported to the nearest 0.1 kPa.
- h) Data plots may include data from more than one instrument provided that the plots are readily readable in both colour and black and white formats and that the instruments are from nearby geographic locations within the site.
- i) Pertinent information from the monitoring reports prepared for WDBA and WEMG for the existing west and east embankments, respectively shall be included in the monitoring reports.

All raw and reduced data shall be provided in electronic form compatible with commonly available computer spreadsheet or database software.

SETTLEMENT MONITORING – Item No.

Each VWP reading provided in electronic format shall include:

- a) unique instrument identification number
- b) ground surface elevation at surface of granular drainage layer at time of installation
- c) pore water pressure transducer depth
- d) pore water pressure transducer elevation
- e) excitation/sweep frequency range used during reading
- f) calibration constants (if applicable)
- g) resonant frequency representing baseline value
- h) resonant frequency representing individual reading
- i) calculated pore water pressure value at time of baseline
- j) thermistor/thermocouple temperature at time of reading
- k) calculated pore water pressure value at time of reading
- l) fill surface elevation at time of reading
- m) applicable Review and Alert level values
- n) technician/operator name
- o) date and time of reading

Each SMP reading provided in electronic format shall include:

- a) unique instrument identification number
- b) elevation of SMP base at time of installation
- c) pipe extension number
- d) top of pipe elevation at time of reading
- e) elevation of fill surface within 25 m of SMP at time of reading
- f) ambient temperature at time of reading
- g) technician/operator name
- h) date and time of reading

In addition, any time an extension is added to the SMP, the elevations of the previous pipe top shall be surveyed. The elevation of the top of the added pipe shall be surveyed as soon as it is installed and the extension shall be numbered in sequence for the particular SMP. Both elevation readings and extension length shall be reported.

Progress Reports

Progress and Data Reports shall be submitted to the Contract Administrator. Weekly reports shall be issued from the beginning of construction monitoring to the end of the one month period immediately after the top of the grading and surcharge fill is reached in all areas. Thereafter, one report shall be submitted after each set of readings is taken. The progress reports shall discuss the Contractor's operations with respect to the installation of instrumentation and a summary of the monitoring completed.

SETTLEMENT MONITORING – Item No.

Monitoring Diary

The Foundation Engineering Consultant shall maintain a daily Monitoring Diary. The diary shall document original conditions, work in progress, including any unusual or problem situations that arise, record of actions taken by the Contractor to rectify the situation, and restored conditions. The diary shall be supported by photographs of these conditions. The diary shall be submitted to the Contract Administrator at the conclusion of the Contract with the Final Report.

Final Report

At the completion of the monitoring program, a final monitoring report shall be issued to the Contract Administrator. The monitoring results shall be presented in tabular and graphical form as described above for each instrument type. The final monitoring report shall be accompanied by electronic data files of all data suitable for use in commonly available computer spreadsheet or database software.

Response Plan For Review and Alert Levels

The Contractor shall develop and submit to the Contract Administrator a preliminary Response Plan for Review and Alert Levels for review thirty calendar days prior to construction. The Contractor shall develop preliminary means and methods to respond to Review and Alert Level scenarios, based on the types of geotechnical instruments showing Review and Alert Level indications. At a minimum, the Response Plan shall include the following:

- a) Names, telephone numbers, and locations of persons responsible for implementation of contingency plans.
- b) Materials and equipment required to implement contingency plans.
- c) The location on Site of all required material and equipment to implement the contingency plans.
- d) The step-by-step procedure for performing the Work involved in the implementation of the contingency plans.
- e) The specific actions related to the Response Level values for all instruments, including means of reducing or eliminating movements and rates of movements.
- f) The inspection of affected facilities, structures, and utilities, and the performance of acceptable corrective and restorative measures.
- g) A clear indication of objectives of the contingency plans and methods to measure the plan's success.

EQUIPMENT AND MATERIALS

Monitoring shall be conducted year-round. All monitoring equipment and materials shall be maintained and rendered operational throughout the monitoring period. Any equipment damage or malfunction shall be investigated and attempts shall be made to remedy the damage or malfunction. The Contract Administrator shall be notified within one week of identifying the damage or malfunction of any equipment that cannot be repaired. Documentation of the possible causes and planned remedial measures shall be forwarded to the Contract Administrator for approval. Malfunctioning or otherwise irreparably damaged instruments shall be replaced within two weeks of the identified malfunction at no additional cost to the Owner.

SETTLEMENT MONITORING – Item No.

Vibrating Wire Piezometers (VWP)

Vibrating wire piezometers shall be either standard borehole piezometer or push-in piezometer. The standard borehole piezometer shall be directly installed in a borehole using the “fully-grouted” method and bentonite-cement grout. Push-in piezometer shall be fitted with a special housing that allows it to be pushed a short distance into soft, cohesive soils. All VWP shall have a rated full-scale range of 3.5 to 5 bar, provide a resolution of 0.025% FS, accuracy of $\pm 0.1\%$ FS and a maximum pressure of 1.5 x rated range, and shall be equipped with a temperature sensor.

Each VWP shall be marked with a unique numbering system that identifies its plan position and transducer tip elevation. Lead wires and connection points for each vibrating wire piezometer shall be protected by a PVC pipe for vertical casing extending to and above the surface of the future fill. These casings shall be co-located with settlement plates as identified below and the installation shall protect wires leading from the gauges to the vertical casing. Connection devices and wires shall be protected from weather, vandalism and construction activities. All grout mixes used for installation of VWP shall be in accordance with manufacturer’s recommendations for fully-grouted piezometers installed within soft cohesive soils.

Push-in VPW shall not be used unless it can be demonstrated that the VWP can be installed with sufficient time after installation and prior to filling for pore water pressures to stabilize to background values.

All VWP installed in existing boreholes shall be fitted with additional cables and protection as required to permit continued monitoring of these instruments in accordance with this specification.

Vibrating Wire Data Recorder

All vibrating wire instruments shall be compatible with the VW Data Recorder as manufactured by the Durham Geo Slope Indicator Company.

Settlement Monitoring Plates (SMP)

Provide and install settlement monitoring gauges (plates) meeting the minimum requirements as identified in this specification and as provided on the attached drawing. Settlement plates shall be designed to permit monitoring by surveying the top of metal riser pipes periodically during placement of fill on and around the plate to an accuracy of ± 2 mm. Metal pipes shall be fabricated to allow extension in 1 m increments by flush-joint threaded couplings. Installation of each extension shall be carried out using methods to secure the previously installed extension against rotation. Threaded couplings shall be securely installed at the top of each extension and survey readings shall be taken at the top edge of the coupling. The lowest coupling at the floor flange shall be fabricated with a left-hand thread and shall be lubricated to allow subsequent removal of the casing at completion of monitoring. All other couplings shall be fabricated with right-hand thread. Each settlement plate shall be marked with a unique numbering system that identifies its plan position and transducer tip elevation.

SETTLEMENT MONITORING – Item No.

CONSTRUCTION

Foundation Engineering Consultant

The Foundation Engineering Consultant shall:

- a) Coordinate all instrumentation and monitoring activities including surveying services with the planned construction, Contractor and Contract Administrator;
- b) Observe and supervise all instrument installation;
- c) Calibrate and maintain monitoring equipment;
- d) Review pertinent settlement monitoring reports prepared for the WDBA and WEMG for sections of the existing embankments, take instrument readings, reduce data, prepare reports;
- e) Transmit instrumentation readings and reports to the Contract Administrator;
- f) Interpret instrumentation readings as needed by the Contractor for the purpose of on-going construction;
- g) Notify Contractor and Contract Administrator of required modifications to the construction procedures, if necessary; and
- h) Notify Contract Administrator if any instrument readings exceed Response Levels for any instrumentation and submit a response action plan to prevent failure of constructed works within 24 hours.

Surveyor

The Surveyor shall:

- a) Coordinate all surveying services with the Foundation Engineering Consultant, planned construction and Contractor;
- b) Carry out all survey monitoring to a repeatable accuracy of ± 2 mm or better;
- c) Carry out Quality Assurance reviews of all survey data acquisition completed using Global Positioning Systems implemented by the Contractor or Foundations Engineering Consultant.

Instrument Installation Locations

All instrumentation shall be installed a minimum of three weeks prior to construction of any embankment or grading fill above the elevation of the wick drain drainage layer surface. Installation of instrumentation shall be coordinated with the installation of wick drains as specified in this Contract. Replacement of instrumentation damaged as a result of the Contractor's activities will be the responsibility of the Contractor.

All installed instruments shall be surveyed and location coordinates shall be established within ± 0.3 m and elevations established to within ± 0.1 m.

Any instruments located in areas subject to installation of wick drains shall be installed at a location in plan equidistant from surrounding wick drains.

Monitoring stations shall consist of one settlement plate and two vibrating wire piezometers. Monitoring stations shall be located as indicated on the Contract Drawings.

SETTLEMENT MONITORING – Item No.

VWP Installation

Prior to installation, all VWP shall be checked for proper operation and calibration by immersing the piezometer below a known head of water not less than 2 m deep. Two readings separated by at least 15 minutes time duration shall be recorded for this calibration test. One reading shall be taken immediately upon installation of piezometer at specified depth or elevation.

Any holes created during installation of push-in VWP shall be filled to the surface with cement-bentonite grout immediately upon completion of installation.

VWP wiring components shall be protected from damage by earthwork equipment and marked by appropriate flagging, barrels, fencing or other warning devices. Any instruments damaged by the Contractor's forces shall be replaced immediately at no additional cost to the Owner.

Underground service clearances shall be obtained prior to installation.

VWP shall be installed at each monitoring station identified on the Contract Drawings and the following elevations:

- a) elevation 172 m and
- b) elevation 167 m.

SMP Installation

Settlement plates shall be firmly installed on level ground at the base of the wick drain drainage layer. Settlement plates shall be adequately weighted with earth materials compacted with manually operated equipment to prevent displacement and dislodgement during subsequent earthwork activities. Settlement plates shall be protected from damage by earthwork equipment and marked by appropriate flagging, barrels, fencing or other warning devices. Any instruments damaged by the Contractor's forces shall be replaced immediately at no additional cost to the Owner.

Benchmarks

A minimum of 3 permanent reference deep surveying benchmarks shall be established for the project site. All deep benchmarks shall be established outside areas that could be affected by settlement due to earthwork or other loads. Coordinates and elevations shall be established at least 3 months prior to monitoring with baseline survey readings for these benchmarks taken on at least 4 occasions during this time. The number of reference benchmarks established shall be sufficient to provide adequate site distances to permit monitoring as specified. Benchmark coordinates and elevations shall thereafter be resurveyed annually.

Instrument Monitoring Schedule and Frequency

Monitoring shall commence immediately after the installation of an instrument. The Contractor shall notify the Contract Administrator of each reading event with at least 24 hours written notice. Monitoring shall continue to six (6) months following completion of all grading and fill placement activities to the final grades specified in the Contract. The monitoring frequency shall be no less than:

SETTLEMENT MONITORING – Item No.

- a) Baseline Readings: Three readings on 3 consecutive days, no sooner than 7 days following installation for VWP. In the case of push-in VWP reading shall be obtained daily until it can be demonstrated that pore water pressures have stabilized consistent with a ground water pressure head elevation no more than 3 m above the elevation of the ground surface prior to any site grading.
- b) First Monitoring Data: One day prior to start of grading construction within 25 m of instrument location.
- c) Regular Monitoring: During grading and fill placement within 25 m of instrument construction (including surcharge placement), once daily or for every 1.0 m thickness of fill whichever results in the more frequent readings;
- d) Each time an extension pipe and coupling are added to the SMP the length of each installed new section shall be measured between the tops of the couplings to an accuracy of ± 2 mm and both couplings shall be surveyed for elevation.
- e) Post-Grading Monitoring: After completion of grading and filling (including surcharge placement) to grades specified in the Contract, weekly for first month, every two weeks for second month; monthly thereafter.

Response Levels and Actions

At no time shall the rate of fill placement be such that the Review Level is exceeded. If Alert Level is reached grading or other construction within 100 m of the affected VWP shall be halted. The Contractor shall meet with the Contract Administrator to discuss the response action(s) necessary to resume construction.

QUALITY ASSURANCE

Foundation Engineering Consultant Qualifications

The Foundation Engineering Consultant services required for this assignment have been categorized as “Geotechnical Specialty – High Complexity”. The Foundation Engineering Consultants registered in MTO’s consultant registry acquisition system (RAQS) at complexity ratings in the required specialty that meet or exceed the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Engineer supervising the work shall have a minimum of five (5) years’ experience in the supply, installation and monitoring of vibrating wire piezometers, standpipe piezometers, settlement plates and survey bench marks.

Surveyor Qualifications

The independent surveyor shall be RAQS listed with a demonstrated capability to meet the accuracy and precision repeatability as required by this specification.

MEASUREMENT FOR PAYMENT

No measurement shall be made for this work

PAYMENT

Payment shall be made at the Lump Sum Price bid in the Form of Tender and shall be compensation in full for all labour, equipment and materials required to complete this work.

SETTLEMENT MONITORING – Item No.

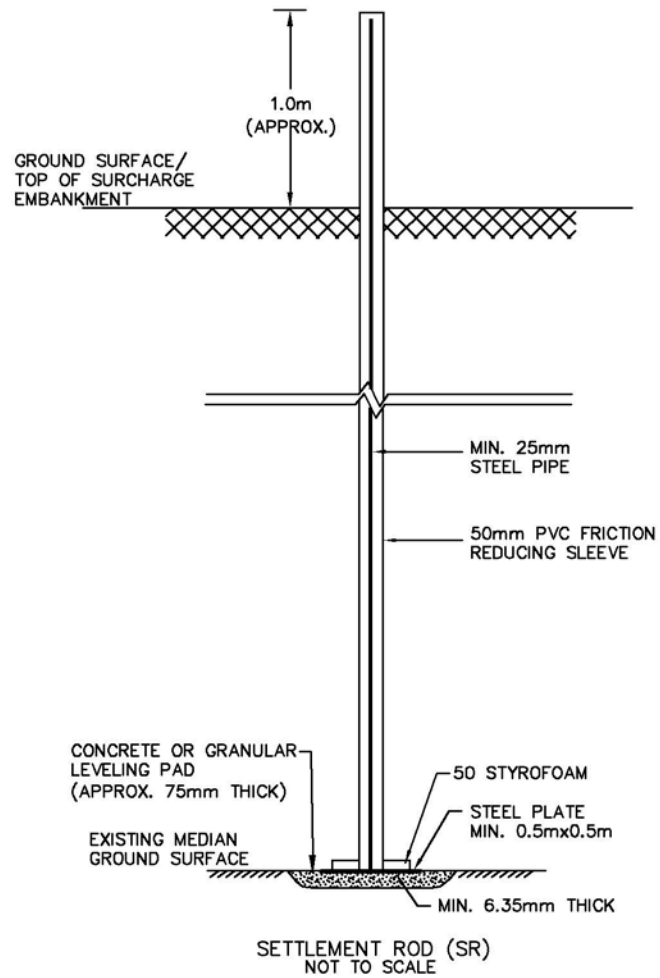


Figure 1. Typical Settlement Plate

WICK DRAINS – WEST APPROACH EMBANKMENT - Item No.

Non Standard Special Provision

General

All work under this item shall be carried out as per OPSS.PROV 220, with the following amendment.

Construction

During wick drain installation the contractor shall regularly monitor for atmospheric hydrogen sulfide within the work zone using personal monitoring devices worn as recommended by the manufacturer and any other systems deemed necessary by the Contractor. All personal monitoring devices shall have a range of 0 to 100 ppm or better and include automatic alarm levels consistent with Occupational Exposure Limits for Ontario Workplaces Required under Regulation 833 (O.Reg. 883). If adverse levels are detected with respect to O.Reg. 883, the contractor shall undertake the necessary measures to adequately vent the work zone and protect all workers, notify the Contract Administrator, cover any wick drains suspected of discharging hydrogen sulfide with a minimum of 1 m of fill, revise the wick drain termination elevation to 160 m, continue wick drain installation work in other areas at least 30 m distant from the affected area as specified, and proceed with remaining wick drain installation within the affected area at revised termination elevation and only after plans to protect workers and complete the work have been submitted to and accepted by the Contract Administrator.

The Contractor shall ensure that all workers involved in wick drain installation shall be trained in the hazards and monitoring of H₂S and provide all such documentation to the Engineer (H₂S Alive course completion).

Measurement

Measurement shall be by length in metres for all accepted wick drains, including the protruding portion up to 150 mm per installation.

Properly installed obstructed wick drains and replacement wick drains shall be measured for payment.

Payment

Payment shall be made at the Unit Price bid in the Form of Tender and shall be compensation in full for all labour, equipment and materials required to complete this work

DEWATERING – Item No.

Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of temporary dewatering systems to facilitate removal of existing uncontrolled fill beneath the footprint of the approach embankments or retaining walls or to permit construction of pile caps for the piers. The uncontrolled fill is variable in nature and contains granular materials which may be saturated particularly after precipitation events or during/after the spring freshet. In addition to removal of the uncontrolled fill material, excavation for these elements may extend into the underlying granular deposits (sand, gravel and silt) and below the groundwater level. Non-cohesive soils below the groundwater table will be subjected to conditions of unbalanced hydrostatic head and can slough, boil, ravel and cave in during temporary excavation work.

REFERENCES

- OPSS 517 Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
- OPSS 518 Construction Specification for Control of Water from Dewatering Operations

SUBMISSION AND DESIGN REQUIREMENTS

Written details for the proposed dewatering system shall be submitted to the Contract Administrator for information purposes a minimum of ten business days prior to commencing dewatering operations. The Contractor shall reference borehole records included in the Contract Documents as a guide in determining requirements.

CONSTRUCTION

Dewatering System

The Contractor is responsible for the design, installation, operation and maintenance of an adequate dewatering system to lower the groundwater level to at least 0.3 m below the embankment subgrade level or founding level for pile caps and other foundations to allow excavation in dry conditions and for construction of the watermain depending on the installation method chosen and adopted by the Contractor. The Contractor is alerted that the fill and native granular deposits overly cohesive deposits. The lower 0.3 metres above these cohesive or any low permeability deposits will be difficult to dewater by any means. Supplementary groundwater control and measures to prevent loss of ground will be required at or near such interfaces.

DEWATERING – Item No.

Operation

A continuous dewatering operation shall be provided to facilitate sub-excavation of the uncontrolled fill and native granular deposits at all times during the work. All components of the dewatering system shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the Contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their preconstruction conditions, unless specified otherwise.

BASIS OF PAYMENT

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

DRIVEN H-PILES – Control of Artesian Groundwater – Item No.

Special Provision

SCOPE

The work under this item includes the design, installation, operation, maintenance and removal of drainage blanket to facilitate control of artesian groundwater conditions during pile driving operations. Flowing artesian conditions are present at or near the bedrock surface. Voids can be created if groundwater is permitted to flow in an uncontrolled manner along the annulus created by the pile.

REFERENCES

OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1840	Non-Pressure Polyethylene Plastic Pipe Products
OPSS 1841	Polyvinyl Chloride (PVC) Pipe Products
OPSS 1860	Geotextiles
OPSD 3000.1000	Foundation Piles, Steel H-Pile Driving Shoe
OPSD 3000.1500	Foundation Piles, Steel H-Pile Splice

MATERIALS

Granular A

Granular A for the drainage blanket shall be in accordance with OPSS.PROV 1010.

Subdrain

Subdrain and associated outlet pipe shall be 100 mm in diameter and in accordance with OPSS 1840 or 1841 and as specified elsewhere in Contract Documents.

CONSTRUCTION

Driving Shoes and Splices

To minimize the size of the pile annulus, driving shoes, reinforcement to flanges, splice plates and the like should add as little as possible to the pile cross-sectional dimensions.

Pile Driving Requirements and Restrictions

Piles shall not be driven until embankment work or excavation work has been completed to the underside of the footing and a drainage blanket as indicated in the Contract Drawings is in place.

DRIVEN H-PILES – Control of Artesian Groundwater – Item No.

Drainage Blanket

Piles be driven from a 0.5 metres thick blanket of Granular A. The edge of the drainage blanket shall be no closer than 1 metre from the inside face of any pile.

The drainage blanket shall be provided with a drainage system consisting of 100 mm subdrain wrapped in fabric around the perimeter of the blanket and draining to a positive outlet or ditch.

Operation

All components of the drainage blanket shall be maintained in an effective, functioning and stable condition at all times during the work. Notwithstanding the above, the work shall be completed in accordance with the environmental and operational constraints specified elsewhere in the Contract.

Restoration

All equipment and materials placed shall be removed from the right-of-way upon the completion of the work and all areas disturbed as part of this work shall be restored to their preconstruction conditions, unless specified otherwise.

BASIS OF PAYMENT

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

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.

NOTICE TO CONTRACTOR - Pile Driving – Soil Conditions - Item No.

Non-Standard Special Provision

The soil conditions are described in the Foundation Investigation Report prepared for this project. Reference should be made to the reports for a description of the soil conditions. The factual data presented on the Record of Borehole Sheets governs any interpretation of the site conditions.

Fills –Debris

The Contractor is informed that the existing uncontrolled fills at this site may contain debris that may impede the progress of driving piles and/or augering operations.

Saturated Granular Soils Below Groundwater Level

Pre-drilling at pile locations will be required as a vibration control measure. The Contractor is informed that the excavation of granular materials below the groundwater level will be required during pre-drilling operations. Dewatering and a temporary liner will be required to support the side of the excavation.

Surface water shall be directed away from the excavation.

Cobbles and Boulders

The presence of cobbles and boulders was inferred from auger grinding and resistance to split-spoon driving encountered during the subsurface investigation. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for pile driving.

Hydrogen Sulphide and Methane

Elevated levels of hydrogen sulphide gas and methane gas were encountered in the artesian aquifer located at the soil/bedrock interface. Suitable precautions should be taken to protect the health and safety of the workers and environment.

INSTRUMENTATION – GENERAL

Special Provision

1.0 SCOPE

The Contractor shall retain a Geotechnical Consultant with MTO classification of Geotechnical (Structures and Embankments) - High Complexity, to undertake the supply and installation of geotechnical instruments and to provide foundation monitoring services during construction.

“The Contractor” shall be understood to refer to the Contractor and their Geotechnical Consultant.

1.1 General

This special provision and the other Item Specific special provisions contain the requirements for the supply and installation of the following geotechnical instruments:

- Settlement Rods (SR);
- Vibrating Wire Piezometers (VWP).

Or other instrumentation as considered necessary by the specialist subcontractor’s designer.

1.2 Purpose

The purpose of the settlement rods and vibrating wire piezometers is to monitor the progress of settlement / lateral movement and dissipation of excess pore water pressure in the foundation soils at in areas where embankment preloads have been placed. These predicted responses are due to the construction existing embankments and preloading. In this project, preloads may be placed in differing stages based on the construction sequence outlined in the Contract Documents.

In the case of preloads, the wait time before placement of additional fill (where applicable and as shown elsewhere in the contract documents) will be controlled by the instrumentation readings.

The requirement for staging the construction and preloading of the embankments is specified elsewhere in the Contract Documents. Partially completed preloaded or fully completed embankments shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of consolidation improvement of the foundation soil has been achieved. The minimum consolidation period shall be specified elsewhere in the contract. No placement of additional fill shall take place until sufficient consolidation has been achieved as determined by the Contract Administrator.

1.3 Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

INSTRUMENTATION – GENERAL

1.4 Utility Locates and Protection

The Contractor shall arrange with the appropriate utility authorities for the stake out of all underground utilities and service connections which may be affected by the Work, in accordance with Subsection GC7.01.16 of the MTO General Conditions of Contract, November 2006.

2.0 REFERENCES

2.1 Drawings

Reference shall be made to the following contract drawings that are contained elsewhere in the Contract:

- Settlement Monitoring Instrument Locations
- Settlement Monitoring Elevation Details

2.2 Subsurface Conditions

The subsurface conditions at the site are described in the following report:

Draft Foundation Investigation Report	Hwy	WP	Site	Report Date	Originating Organization
<i>Geocres No. 40J6-71, Ojibway Parkway/ETR Overpass, Sites 6-600/1 & 2 (Bridge B-1) Highway 401 (Rt. Hon. Herb Gray Parkway)</i>	<i>401</i>	<i>3028-14-00</i>	<i>6-600/1 & 6-600/2</i>	<i>November 2016</i>	<i>Golder Associates Ltd.</i>

3.0 DEFINITIONS

3.1 Or equal

The term, “or equal” shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

4.0 DESIGN SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor shall submit details of proposed installation methods including locations and types of survey benchmarks, and installation schedule to the Contract Administrator, a minimum of three weeks before the start of instrument installation.

5.0 MATERIALS

The Contractor shall supply all materials required for the installation of instrumentation unless otherwise noted.

INSTRUMENTATION – GENERAL

6.0 EQUIPMENT

The Contractor shall supply all equipment required for the installation of instrumentation unless otherwise noted.

6.1 Equipment Operation and Weather Conditions

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. Monitoring will be conducted year round (by others).

7.0 CONSTRUCTION

7.1 Installations

7.1.1 Quantities and Locations of Instruments

The quantity and location of instruments are given elsewhere in the Contract documents.

7.1.2 Survey Bench Marks

The Contractor shall provide a minimum three (3) non-yielding deep seated survey bench marks and shall establish the geodetic elevation and position of each such benchmark.

The number and locations of bench marks shall be such that direct sighting is possible from all geotechnical instruments to at least one bench mark.

7.1.3 Personnel

Instrument installation shall be undertaken under the full time supervision of the Contractor and the Foundation Engineering consultant retained by the Owner.

7.1.4 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location. The Contractor shall be responsible for locating and protecting all underground utilities prior to installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Contract Administrator.

7.1.5 Marking and Labelling

The location of any above ground monitoring fixtures shall be made clearly visible to nearby traffic before, during and after embankment widening/surcharge construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments and/or their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for the entire period of monitoring.

INSTRUMENTATION – GENERAL

7.1.6 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction or by vandalism. Any instrument damaged by the Contractor's work shall be immediately replaced at the Contractor's cost.

7.1.7 Accuracy of Surveying

Elevations of the deep benchmarks and all other elevations to be determined by the Contractor shall be surveyed to an accuracy of 2 mm +/- or better.

7.1.8 Survey Personnel

Surveying to establish the benchmarks and other elevations shall be carried out by a registered surveyor with appropriate equipment and experience. The surveyor shall be retained by the Contractor.

7.1.9 Boreholes

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled. In-situ or laboratory testing is not required.

Boreholes shall be advanced using conventional drilling methods and shall be as straight and vertical as practical.

7.1.10 Installation Program

The Contractor shall install settlement plates/rods and vibrating wire piezometers as per the Contract Documents.

The settlement rods shall be attached to a plate installed on a levelling pad on the existing ground surface within the shoulder area where new embankments are being constructed or within the shoulder or bench areas where existing embankments are being widened. As construction of the new or widened embankments proceeds, the rods, sleeves and CSP protection shall be extended above the new top of embankment.

Vibrating wire piezometers shall be installed as detailed elsewhere in the Contract. As construction of the preload proceeds, the vibrating wire piezometer cables and CSP protection shall be extended above the new top of embankment or preload.

7.2 Monitoring

7.2.1 Personnel/Access

Data collection, interpretation and reporting shall be conducted by others, under the direction of the Contract Administrator.

The Contractor shall provide access and assistance to others reading all geotechnical instruments. This may include, but not necessarily be limited to: safe access to each instrument location, snow clearing (if required), a stable platform to support the technician and equipment to access instrumentation during construction.

INSTRUMENTATION – GENERAL

7.2.2 Monitoring Program

The Contractor shall meet with the Contract Administrator and staff responsible for the on-going monitoring immediately after installation of the instruments and before completing the embankment to top of preload level. This meeting is referred to as the “hand over” meeting.

At the meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments and all equipment to be supplied by the Contractor.

Monitoring by others for the baseline readings shall commence within seven working days after the “hand over” meeting. The monitoring shall continue on the schedule indicated or as determined by the Contract Administrator throughout the completion of the new embankment, embankment widening or preload as dictated by the instrumentation readings, or until completion of the warranty period for the ground improvement, as applicable.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the number of monitors placed. The unit of measurement is each.

10.0 BASIS OF PAYMENT

Payment at the contract price for the above tender items shall include full compensation for all labour, equipment and material required to provide settlement monitoring during construction.

SETTLEMENT MONITORING PLATES (SMPs)

Special Provision

1.0 SCOPE

1.1 General

This special provision contains the requirements for the supply and installation of settlement monitoring plates (SMPs) and data collection during construction.

The purpose of the settlement monitoring plates is to directly monitor settlements of the base of the embankment or preload in areas where new embankments are to be constructed or preloaded. Settlement is measured by survey of the top of the rod with reference to stable, non-settling benchmarks.

1.2 General Procedure

The settlement monitoring plates shall be attached to a plate installed on a levelling pad on the existing ground surface within the area where the preload is to be constructed. As construction of the preload or final embankment proceeds, the rods shall be extended above the new top of embankment preload.

Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate. The sleeves shall be extended above the new top of embankment or preload as construction proceeds.

The Contractor Administrator shall be notified a minimum of 2 working days prior to the extension of rods/sleeves/protective surround.

Where the rods are located within the future roadway width, the rods, sleeves and protective surround shall be cut down to just below subgrade level after the monitoring program is complete.

1.3 Location

The locations of the settlement monitoring plates are shown on the Contract Drawings. A typical installation detail is also given elsewhere in the Contract.

2.0 REFERENCES - Not Used

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

SETTLEMENT MONITORING PLATES (SMPs)

5.0 MATERIALS

5.1 General

The Contractor shall supply all materials and equipment required for the installation of the settlement rods.

5.2 Plate

The Contractor shall supply a steel plate with thickness of at least 6.35 mm. It shall be at least 0.5 m by 0.5 m in plan dimensions.

5.3 Rod

The Contractor shall supply a steel pipe with an outside diameter of at least 25 mm.

The top of the rod shall be capped in such a way that a single survey point can be clearly identified and returned to.

5.4 Friction Reducing Sleeve

The Contractor shall supply a PVC pipe, friction reducing sleeve with an internal diameter slightly larger than the rod diameter.

6.0 EQUIPMENT

6.1 Monitoring Equipment

An experienced registered surveyor, retained by the Contractor, to provide the datum readings, shall survey the elevation of the top of the settlement monitoring plates. The surveyor shall provide suitable equipment capable of surveying settlement monitoring plate elevations to an accuracy of +/- 2 mm or better.

7.0 CONSTRUCTION

7.1 Installation

7.1.1 General

The Contractor shall install settlement rods as detailed elsewhere in the Contract, in addition to what is stated or emphasized below.

7.1.2 Settlement Plate

The settlement plate shall be installed horizontally on level and compacted fill, just below the top of the existing ground surface or at the base of the wick drain layer, as applicable. Following removal of any unsuitable/deleterious materials, a levelling pad consisting of granular fill should be placed on top of the prepared ground surface.

The elevation of the base of the plate shall be surveyed by the Contractor before backfilling.

SETTLEMENT MONITORING PLATES (SMPs)

7.1.3 Rod

The rod shall be fixed to the centre of the plate and perpendicular to the plate.

The rod shall be extended in 1.5 m increments as the embankment preload increases in height.

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.1.4 Friction Reducing Sleeve

The friction reducing sleeve shall extend over the entire length of the rod that is below ground and within the new embankment fill.

7.1.5 Installation Details

The elevation, easting and northing of the centre of the base of the plate shall be surveyed by the Contractor.

The elevation, easting and northing of the top of the rod shall be surveyed by the Contractor.

The total distance from the base of the plate to the top of the rod shall be measured and recorded to an accuracy of +/- 2 mm or better.

The contractor is responsible for preventing damage to the SMP during placement of the embankment or preload fills. If the rod or extension is damaged during the filling, the rods and protective casing shall be replaced and surveyed before resuming the filling.

7.2 Monitoring

The SMPs shall be monitored by a licensed surveyor, under the direction of the Contract Administrator as described below and elsewhere in the Contract Documents.

7.2.1 Baseline Readings

Monitoring of the SMPs shall commence within seven (7) working days after the “hand over” meeting as described elsewhere in the Contract Documents.

Prior to the start of preload construction, a minimum of three baseline readings must be obtained. Anomalous readings which cannot be repeated are to be discarded and the average of the remaining readings used as a datum.

7.2.2 Monitoring Frequency

The monitoring schedule and frequency shall be as indicated in the Special Provision for Settlement Monitoring.

SETTLEMENT MONITORING PLATES (SMPs)

Anomalous readings should be flagged, checked and discarded, if necessary. The reason for the anomalous reading should be identified and corrected, if possible. Damaged SMPs shall be reported to the Contract Administrator.

The monitoring data should be regularly reviewed and analysed in order to assess performance of the embankment, and to determine if adjustment of the monitoring schedule or construction methodology or schedule is necessary.

In all areas monitoring shall continue until the nearby VWPs indicate that all excess porewater pressures beneath the permanent embankment area have sufficiently dissipated and the resulting consolidation settlement curve is essentially horizontal.

7.3 Removal

After completion of the settlement monitoring period, the SMPs and sleeves should be removed to at least 0.3 m below the subgrade by excavating and cutting of the rods and casings. The voids resulting from the removal of the settlement rods should be backfilled with compacted granular.

7.4 Reporting

7.4.1 Installation Records

The Contractor shall record and report relevant installation details to the Contract Administrator as described elsewhere in this Contract. These include, but are not limited to:

- settlement rod and plate location, easting, northing;
- elevation of plate and rod;
- distance between base of plate and top of rod;
- dates of installation and datum readings;
- installation notes / sketches; and
- description of settlement rods, sleeve, plate.

7.4.2 Monitoring Records

The party responsible for monitoring the SMPs shall record and report the readings to the Contract Administrator as described elsewhere in the Contract Documents. For each SMP, these shall include, but are not limited to:

- baseline, datum and subsequent readings;
- date and time of each reading and prevailing weather conditions;
- stage of embankment or preload construction including fill height above original ground at time of reading;
- notes relating to anomalous readings; and
- notes about damaged or inoperable equipment.

SETTLEMENT MONITORING PLATES (SMPs)

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the number of settlement rods placed. The unit of measurement is each.

10.0 BASIS OF PAYMENT

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

VIBRATING WIRE PIEZOMETERS

Special Provision

1.0 SCOPE

1.1 General

This special provision contains the requirements for the supply and installation of vibrating wire piezometers (VWPs) and data collection during construction.

The purpose of the piezometers is to monitor pore water pressure at depths within the clayey foundation soil. The piezometer readings shall help to establish the timing for placement of additional fill or construction of the permanent embankment after a period of preloading.

1.2 General Procedure

The piezometers shall be installed in boreholes or pushed-in prior to construction of the new embankments, embankment widenings/preloads. The boreholes shall be of sufficient diameter to accommodate installation of the VWP sensor, filter sand and grout.

The vibrating wire piezometer cables should be of sufficient length to allow monitoring of the instruments at a location outside the limits of the embankment fills. CSP protection shall be provided above the ground surface at the monitoring location.

The cables should be laid within a minimum 50 mm diameter PVC conduit buried at least 0.5 m below the ground surface. Bentonite water stops should be placed at appropriate locations. Kinking or use of small radius bends in the cable should be avoided to prevent damage during construction.

Installation details are shown elsewhere in the Contract.

Boreholes containing VWP sensors shall not be located closer than 3 m from any other adjacent existing boreholes or piezometers.

1.3 Locations

The Contractor shall install VWP sensors at the locations shown on the Contract Drawings.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS – Not Used

4.0 DESIGN SUBMISSION REQUIREMENTS – Not Used

VIBRATING WIRE PIEZOMETERS

5.0 MATERIALS

5.1 VWP Piezometers

The Contractor shall supply VWP borehole piezometers by Slope Indicator, Model 52611020 (-5 to 50 psi) - or equal, compatible with the Slope Indicator VS DataMate Model 52620900 - or equal. All piezometers shall be of the same make.

All piezometers shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

5.2 Signal Cable

The Contractor shall supply Slope Indicator Model 50613524 cable – or equal. The length of cable for each piezometer shall be estimated from the Contract Drawings and specifications with sufficient additional length to ensure that there is enough signal cable for each piezometer to allow for extending the cables as embankment or preload construction proceeds. In addition, there shall be sufficient additional length to provide enough slack in the borehole and in the protective casing sufficient to allow monitoring. The cables shall be protected from earthmoving equipment and water intrusion at all times. Cables should be labelled for later identification.

5.3 Bentonite

The Contractor shall supply bentonite in pellet form in sufficient quantity to form borehole plugs, and water stops in the signal cable trenches, as required.

The Contractor shall supply bentonite in powder form in sufficient quantity for the bentonite-cement grout mix for general borehole backfilling.

5.4 Filter Sand

The Contractor shall supply clean washed sand for filter around VWP sensors. The sand shall be Sakcrete washed general purpose sand - or equal.

5.5 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design shall be provided by the Contractor.

5.6 Protection Riser Pipe

The Contractor shall supply a sufficient quantity of steel pipe with a nominal inside diameter of approximately 50 mm. The protection riser pipe is required within the embankment or preload fill and must extend through and above the preload/surcharge portion of the embankment as it is constructed following installation of the VWPs.

VIBRATING WIRE PIEZOMETERS

5.7 Protective Surround

The CSP protective surround shall be extended in 1.5 m increments with the riser pipe to the final top of surcharge surface.

The piezometer signal cable shall be within the riser pipe in the centre of the CSP.

The annulus between the CSP and the riser pipe shall be filled with compacted sand to a level no higher than the final top of surcharge surface.

5.8 Protective Housing

The Contractor shall supply a protective housing consisting of galvanized steel pipe or box section with a minimum internal dimension of 100 mm and equipped with a locking cap to enclose the portion of the piezometer installation that is above ground surface after completion of the surcharge embankment. The protective housing shall be installed after surcharge filling is completed. Each installation shall be protected during completion of construction as per the approval of the Contract Administrator.

5.9 Conduit for Signal Cables

The Contractor shall supply a sufficient quantity of PVC conduit with a nominal inside diameter of approximately 50 mm for protection of the signal cable leads. Fittings (caps, elbows, bends, etc.) required to enable daylighting or redirection of the cables shall also be provided by the Contractor.

6.0 EQUIPMENT

6.1 Readout Unit

The Contractor shall supply a Slope Indicator Company VS DataMate Model 52620900, or equal, to read and store the piezometer readings. The Contractor shall also supply Slope Indicator Company IDAgraph software Model 58710030, or equal, to generate plots of pore pressure with time. The readout box and software shall become the property of the Ministry and shall be handed over to the Contract Administrator at the “hand over” meeting following completion of instrument installation.

7.0 CONSTRUCTION

7.1 Installation

7.1.1 General

Installation of the VWP piezometers shall be as per the manufacturer’s recommendations in addition to what is stated or emphasised below.

7.1.2 Borehole Installation

The borehole shall be advanced to no more than 200 mm below the design tip elevation using suitable drilling techniques. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

VIBRATING WIRE PIEZOMETERS

The piezometer sensor shall be saturated, as per the manufacturer's recommendations. In addition, the borehole should be filled with water upon installation of the sensor and filter sand into the base of the hole to maintain saturation of the sensor throughout the installation process.

The maximum length of the filter sand zone shall be 600 mm.

The piezometer shall be installed as shown elsewhere in the Contract.

7.2 Monitoring

The VWP's shall be monitored by experienced personnel, under the direction of the Contract Administrator as noted elsewhere in the Contract Documents.

7.2.1 Baseline Readings

Monitoring of the VWP sensors shall commence within seven working days after the "hand over" meeting as described elsewhere in the Contract Documents. Readings shall be taken at least once daily and repeated with enough frequency to establish that the porewater pressures have stabilized immediately after installation of the VWP's.

Prior to the start of preload construction, but after the post installation porewater pressures have stabilized, a minimum of three baseline readings must be obtained. Anomalous readings which cannot be repeated are to be discarded and the average of the remaining readings used as a datum.

7.2.2 Monitoring Frequency

The monitoring frequency for VWP's shall follow the minimum frequencies indicated in the Special Provision for Settlement Monitoring.

The monitoring data should be regularly reviewed and analysed in order to assess performance of the embankment, and to determine if adjustment of the monitoring schedule or construction methodology or schedule is necessary.

In all areas monitoring shall continue until it is clear that all excess porewater pressures beneath the embankment or preload area have sufficiently dissipated as indicated by stable porewater pressure readings.

Anomalous readings should be flagged, checked and discarded, if necessary. The reason for the anomalous reading should be identified and corrected, if possible. Damaged or non-functional VWP's shall be reported to the Contract Administrator.

7.3 Reporting

7.3.1 Installation Records

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWP location, easting, northing;
- elevations of VWP sensors;

VIBRATING WIRE PIEZOMETERS

- stratigraphic log of subsurface conditions, including drilling method notes;
- dates of installation;
- installation notes / sketches;
- model, make and serial numbers of VWP sensors, readout unit and signal cable; and
- calibration details of VWP sensors.

7.3.2 Monitoring Records

Others responsible for monitoring the VWP shall record and report the readings for the Contract Administrator. For each VWP, these shall include, but are not limited to:

- baseline, datum and subsequent porewater pressure and temperature readings;
- date and time of each reading and prevailing weather conditions;
- stage of embankment or preload construction including fill height above original ground at time of reading;
- notes relating to anomalous readings; and
- notes about damaged or inoperable equipment.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the number of vibrating wire piezometers placed. The unit of measurement is each.

10.0 BASIS OF PAYMENT

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



APPENDIX K

Vibration Assessment

DATE June 14, 2016**PROJECT No.** 13-1132-0053-1002-M01**TO** Chris Schueler, P.Eng., Dept. Head - Highway Design
AECOM Canada Ltd.**CC** Mahfuz Alam, M.Eng, P.Eng., Senior Project Engineer
AECOM Canada Ltd.**FROM** Daniel Corkery, B.Sc.
Dirka U. Prout, P.Eng.
Fintan J. Heffernan, P.Eng.**EMAIL** daniel_corkery@golder.com
dirka_prout@golder.com
fheffernan@golder.com**VIBRATION ASSESSMENT FOR THE PILING INSTALLATION AT THE
OJIBWAY PARKWAY / ETR OVERPASS
SITES 6-600/1 & 2, HIGHWAY 401 (RHHGP)
GWP 3028-14-00**

1.0 BACKGROUND

We understand that three-span twin structures (Sites 6-600/1 & 2) have been designed to convey T. Honourable Herb Gray Parkway (RHHGP) over Ojibway Parkway, the Essex Terminal Railway (ETR) line and the Perimeter Access Road (PAR) at the western terminus of the RHHGP and are now scheduled to be constructed. The installation of piles (sheet and H piles) will be required for the abutments and two piers. Because existing buried infrastructure is located near the proposed piling, there have been questions and concerns raised whether the pile driving operations may impact that infrastructure. AECOM Canada Ltd. (AECOM) has retained Golder Associates Ltd. (Golder) to conduct a ground vibration assessment to determine the peak ground vibrations likely to be generated at the nearby infrastructure from the piling operations and estimate the potential impact on the infrastructure. A Key Plan for the project is shown on Figure 1, attached.

AECOM provided Golder with a copy of a Subsurface Utility Engineering (SUE) mapping investigation report, prepared by T2 Utility Engineers titled "MTO Bridge B-1 Structure over Ojibway Parkway – Windsor, Ontario, Report Subsurface Utility Engineering Services", dated September 8, 2015. This investigation identified several utilities in an area south of the intersection of Ojibway Parkway and the E.C. Row Expressway, specifically south of the Broadway Boulevard curblin, about 15 metres east of the Ojibway Parkway East curblin and approximately 30 metres west of the ETR tracks. Four additional utilities were identified on the General Arrangement (GA) Drawing for Bridge B-1. These utilities are situated between the west abutment and Pier #1 and beyond the SUE west limit. These utilities were included in the vibration assessment for completeness, subject to confirmation of their location and provision of additional details by AECOM.

2.0 VIBRATION LIMITS

The intensity of ground vibrations, which is an elastic effect measured in units of peak particle velocity (PPV), is defined as the speed of excitation of particles within the ground resulting from vibratory motion. For the purposes of this report, PPV is measured in millimetres per second (mm/s). Ground vibration guidelines are typically



established for pile driving activities at construction sites to prevent damage to adjacent facilities or infrastructure. Exceeding these levels does not in itself imply that damage has occurred but only increases the potential that damage might occur. Table 1 provides a summary of PPV limits typically adopted for a variety of infrastructure types.

Table 1: Vibration Limits for Various Infrastructure Types

Infrastructure Type	PPV Limit (mm/s)	Comments
Power Transmission Towers *	100	Concrete footings
Wooden Hydro Poles *	240	
Electrical Sub-stations *	10 – 30	Depending on switch type. Manufacturer should be consulted.
Railway Tracks *	100	
Buried Natural Gas Pipelines **	50	
Underground Fibre Optics Line *	100	
Concrete and grout <72 hours after placement ***	10	
Mine Plant and Industrial Buildings *	100	Unoccupied structures of reinforced concrete or steel construction

* Suggested by Richards and Moore (2007)

** Vibration limits adopted by the National Energy Board (NEB) for operating lines and imposed by several natural gas producers to protect their pipelines.

*** OPSS .PROV 120

Based on the information provided by AECOM, the main infrastructure near the proposed overpass structure foundations are gas lines, underground electrical cables, watermain, sanitary sewer lines and a railroad track. A vibration limit of 100 mm/s should be adopted for the ETR track and a conservative vibration limit of 50 mm/s is suggested for the remaining infrastructure.

If ground vibration limits for any of the infrastructures identified differ from those proposed above and are made known to AECOM or Golder, those limits shall be used in order to mitigate impact on that structure.

3.0 GROUND VIBRATION PREDICTION MODEL

3.1 Impact Hammer

A basic empirical relationship for predicting vibration levels from piling operations was developed by Attewell and Farmer (1973) and modified by many authors since then. The model which is defined as:

$$V = K \left(\frac{\sqrt{W}}{R} \right)^x$$

Where,

V = the PPV (mm/s)

W = the input energy (hammer energy) (J)

R = radial distance between the pile and the monitoring point (m)

x = empirically determined dimensionless constant of proportionality

Based on studies by Hope and Miller (2000) that compared eight vibration attenuation models for piling operations, the recommended K values for the prediction of ground vibration levels, where on site field trials have not been carried out, are those established by Whyley and Sarsby (1992). The model is used for both H-pile and sheet pile installation (Whyley and Sarsby, 1992). The assumed K and x values assumed are based on the applicable soil types shown below:

$K = 1.50$ for stiff or dense soil;

$K = 0.75$ for firm to stiff or medium dense soil;

$K = 0.25$ for loose soil; and

$x = 1.0$ for all soil types.

Studies of the site indicates predominant soil x as a clayey silt to silty clay deposit and is, on average, firm to stiff. Based on the soil conditions indicated, a K value of 0.75 has been assumed for the subject site.

3.2 Vibratory Hammer

A study presented by Athanasopoulos & Pelekis (2000) concluded that the vibratory sheet pile driving using 1,000 to 3,000 Nm hammer in competent soil induces a maximum PPV given by the following:

$$V = 80R^{-1.5}$$

Where,

V = the PPV (mm/s)

R = radial distance between the pile and the monitoring point (m)

3.3 Summary

Because of the level of uncertainty associated with estimates using models from published literature, the assumed constants may be more conservative than site-specific constants derived from a regression analysis of the data from monitored vibrations at the site. The use of such literature-derived site constants should be used only until such time as actual data for the site is obtained. Golder understands that this data will be collected at a later date.

Information provided to Golder indicates that both H-piles and sheet piles will be used. Since a specific hammer is not currently known, a 60,000 J impact hammer has been assumed for the impact piling and 3,000 Nm hammer for vibratory piling.

The estimated ground vibrations at a range of standoff distances are shown in Figure 1.

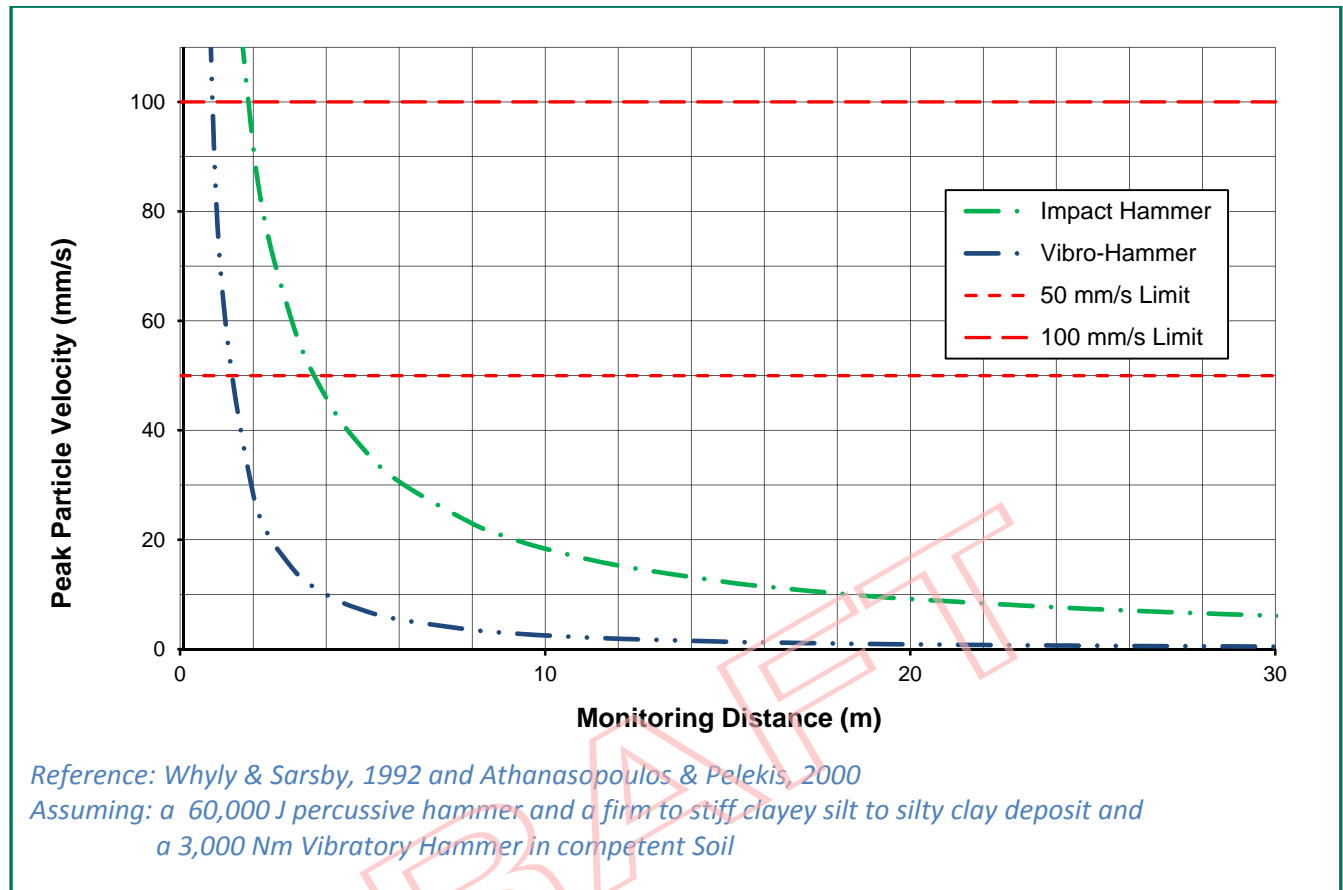


Figure 1: Ground Vibration Attenuation Curve for an Energy Rating of anticipated Hammer Piling Operations

Because the impact hammer estimates are significantly higher than that for the vibratory hammer, the estimated PPVs for a range of separation distances will be based on the use of the impact hammer. These are summarized in Table 2.

Table 2: Estimated Maximum Ground Vibrations from Proposed Piling Operations

Separation Distance (m)	PPV (mm/s)
2	92
3	61
4	46
5	37
6	31
7	26
8	23
9	20
10	18

Separation Distance (m)	PPV (mm/s)
15	12
20	9
25	7
30	6

Assuming: a 60,000 J percussive hammer and a firm to stiff clayey silt to silty clay deposit.

The estimated PPV level from the identified infrastructure (listed east to west from the east abutment) are summarized in Table 3:

Table 3: Estimated Ground Vibration PPV at for Identified Utility ¹

Utility ID	Utility Type	Distance (m)	PPV (mm/s)	Comments
Gas-Union 100	Gas Line	5.6	32.8	Distance from east abutment
Gas-Union 75	Gas Line	7.7	23.9	Distance from east abutment
BE-SL	Buried electric street light cable	9.9	18.6	Distance from east abutment
STM	Storm sewer	29.3	6.3	Distance from east abutment
BE-SL	Buried electric street light cable	17.3	10.6	Distance east of Pier #2
SAN 675	Sanitary Sewer	14.7	12.5	Distance east of Pier #2
BE	Buried electric	7.0	26.2	Distance east of Pier #2
WM 600	Water main	16.3	11.3	Distance east of Pier #1
ETR	Rail tracks	32.0	5.7	Distance east of Pier #1
SAN FORCEMAIN 800	Sanitary Sewer	5.7	32.2	Distance east of Pier #1
Gas ⁴	Gas Line (150 mm)	3.2	57.4	Distance west of Pier #1
Conc Case ⁴	Concrete Encased Duet Bank	6.5	28.3	Distance west of Pier #1
WM 300 ⁴	Watermain (diameter unknown)	10.6	17.3	Distance west of Pier #1
INVERT STM ⁴	Storm Sewer (300 mm)	11.4	16.1	Distance from west abutment

1) At smallest separation between the utility and proposed piling operations

2) Utilities denoted as "ABND" in the drawings were assumed abandoned and not listed in the table.

3) Assuming a 60,000 J percussive hammer and a firm to stiff clayey silt to silty clay deposit.

4) Utility located outside of the limits of the August 2015 SUE mapping investigation.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Peak ground vibration levels emanating from the proposed piling operations trend off rapidly from the source. No adverse vibrations are anticipated as a result of vibratory piling operations.

At all but one of the identified infrastructures, the PPV levels for the currently proposed H-pile and sheet pile installation activities (using impact hammers) are estimated to be below the proposed vibration limits. The PPV level at the gas line located approximately 3.2 m west of Pier #1 is estimated to be above the proposed 50 mm/s limit. The actual ground vibration levels can only be confirmed through on site monitoring during the proposed site

works. If the ground vibrations recorded during monitoring exceed the limits, the pile driving operator shall take steps to reduce the PPV level at the structure. If these steps are unsuccessful, piling shall cease and the contractor shall submit a proposal detailing the proposed plan to reduce the vibrations.

It should be noted that the analyses for ground vibration attenuation presented in this report are based on literature-derived site constants for empirical formulae. The models are intended as a first approximation until site-specific data can be obtained. In order to provide a refined estimate of the PPV for the ground vibration attenuation, a ground vibration program of monitoring pile installation is proposed. The program will entail monitoring the induced vibrations at a range of separation distances from the test piles during installation, starting from areas away from the 150 mm gas line. Curve fitting of the data to the upper 95% confidence level will enable a refined, site-specific estimate of the attenuation and the maximum level likely to be induced at the identified utilities. This, in turn will provide greater confidence in the potential of the pile driving to impact the utilities.

Because of the proximity of the pile driving operations to several structures, vibration monitoring, in accordance with the Canadian Highway Bridge Design Code (2014), is proposed for separation distances of less than 5.0 m between the structure and the piling. This should be re-evaluated following the collection of site-specific data and subsequent refinement of the attenuation model.

5.0 REFERENCES

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- Ontario Provincial Standards Specification (OPSS), 2014. "General Specification for the Use Of Explosives", OPSS.PROV 120, November 2014, 10 pp.
- Whyley, P.J. and Sarsby, R.W. (1992) "Ground borne Vibration from Piling", *Ground Engineering*, Vol. 26, No. May 1992, pp 32-37.

6.0 CLOSURE


We hope that we have satisfactorily addressed the effects of the predicted peak ground vibration levels at the utilities adjacent to the proposed pile installations for the during the sheet pile installations.

GOLDER ASSOCIATES LTD.



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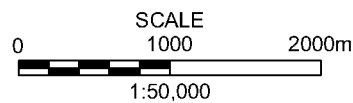
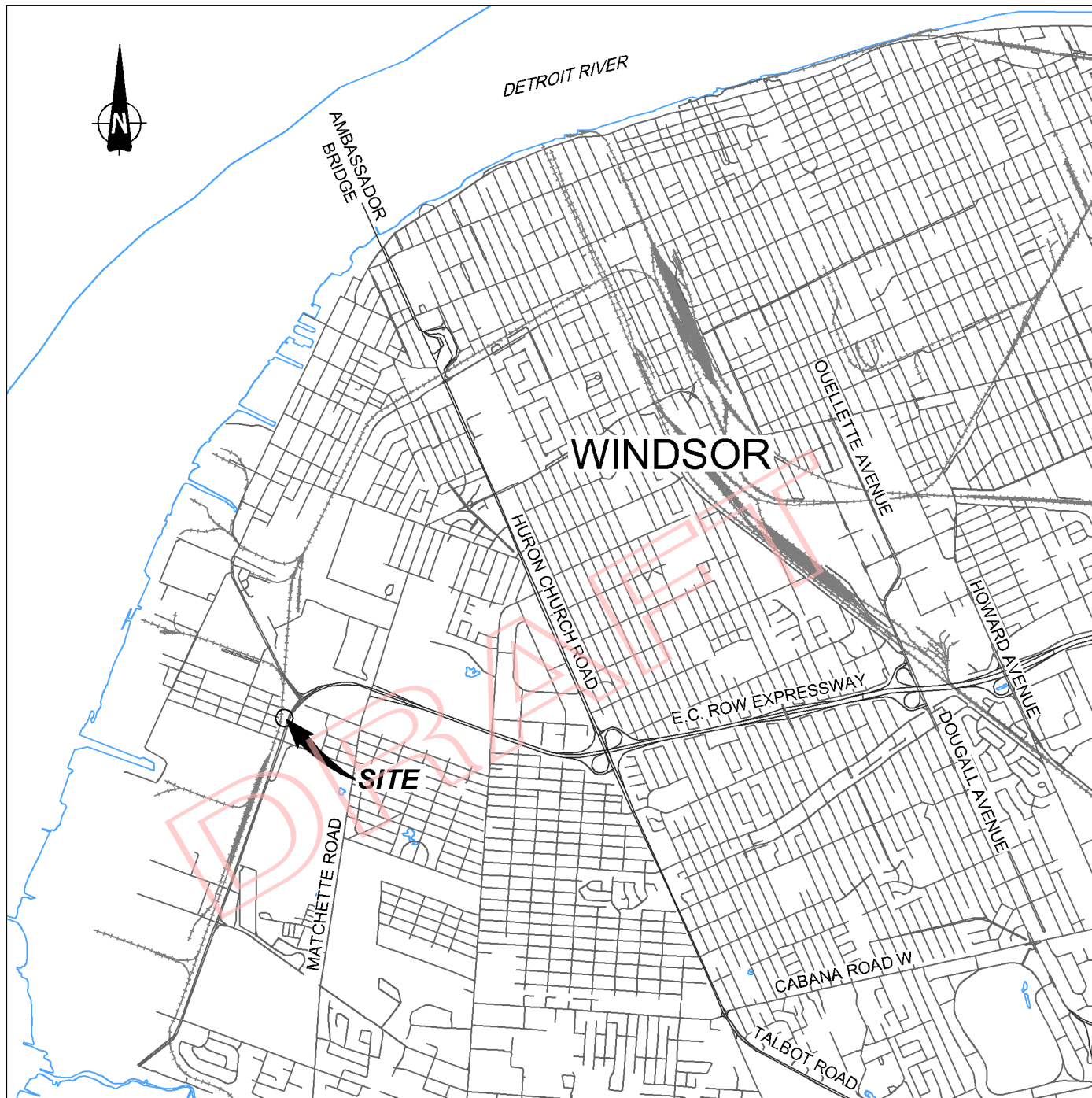


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Senior Geotechnical Engineer

DJC/DUP/FJH/cr

Attachment: Figure 1 – Key Plan

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REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.

NOTE

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

PROJECT

OJIBWAY PARKWAY/ETR OVERPASS, SITES 6-600/1 & 2
HIGHWAY 401 (RHHGP)
GWP 3028-14-00

TITLE

KEY PLAN



**Golder
Associates**

PROJECT No.		13-1132-0053	FILE No.		1311320053-1000-F01001
CADD	DCH	May 20'16	SCALE	AS SHOWN	REV. 0
CHECK			FIGURE 1		

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