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FOUNDATION INVESTIGATION AND DESIGN REPORT

Replacement of QEW Twin Bridges over Welland River
(Site Nos. 34-65/1 & 2)
Queen Elizabeth Way (QEW), City of Niagara Falls, Ontario
MTO WP 2430-15-00, Contract No. DB 2018-2013

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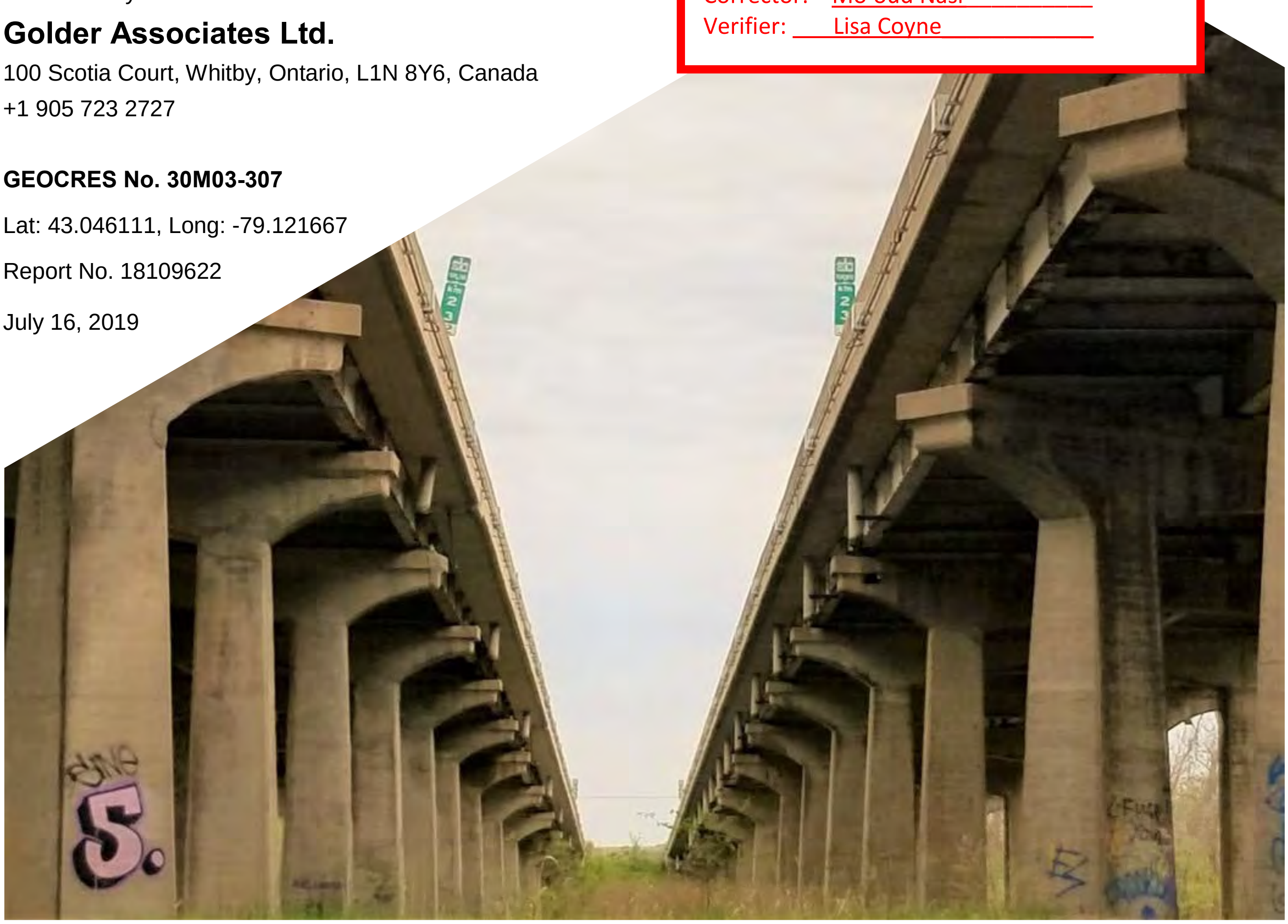

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FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER
(SITE NOS. 34-65/1 & 2)
QUEEN ELIZABETH WAY (QEW), CITY OF NIAGARA FALLS, ONTARIO
MTO WP 2430-15-00, CONTRACT NO. DB 2018-2013

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Rankin Construction Inc. (Rankin), as a sub-consultant to Parsons Corporation (Parsons) to prepare a Foundation Investigation and Design Report for the replacement of the QEW Twin Bridges over the Welland River in Niagara Falls, Ontario. The terms of reference for the scope of work are outlined in MTO's Request for Proposal (RFP) titled, "Replacement of the Welland River Bridges and Rehabilitation of Three Structural Culverts, Design-Build Contract Number 2018-2013", issued November 14, 2018 and subsequent addenda, and in Golder's proposal P18109622, dated May 23, 2019.

2.0 SITE DESCRIPTION

The existing bridge structure is located on Queen Elizabeth Way (QEW) about 1.7 km south of the QEW/McLeod Road interchange in the City of Niagara Falls, Regional Municipality of Niagara, Ontario. The QEW in this area is a four-lane divided highway carrying two lanes of traffic in each direction with a concrete median barrier at the bridge location. The QEW in this area runs in a north-south orientation with the "Eastbound Lanes (EBL)" travelling north towards Toronto and the "Westbound Lanes (WBL)" travelling south towards Fort Erie.

The existing Welland River Bridges are each 290 m long, eighteen-span, steel-girder composite bridge structures, constructed in 1941, that carry the QEW over the Baden Powell Trail, the Canadian Pacific Rail (CPR), the Welland River and Oakwood Drive, from south to north. Based on the Pile and Footing Plan as-built drawing (No. 1) circa 1941 for the existing structure, the north abutment and Bent #1, #2 and #3 are supported on driven timber piles that are 13.7 m (45 feet) long, generally vertical although some were indicated to be battered. The foundation type and founding depth of the remaining abutments and bents are unknown, however it is likely that they are similarly supported on driven timber piles implied (for assumed 13.7 m long piles) to be founded within firm to very stiff silty clay deposit or possible the dense to very dense till deposits. The approach embankments at the north and south abutments are approximately 7.2 m and 8.5 m high above the original ground in the valley, respectively. The grade of the QEW is at about Elevations 180.0 m and 184.0 m at the north and south abutments, respectively.

Welland River is a major river which flows from west to east through the Niagara Region and discharges into the Niagara River. At the structure site, the river is approximately 70 m wide and its base is located at approximately Elev. 165.5 m. It is located within a broad and relatively flat floodplain which ranges from about Elevation 172 m immediately north of the river up to as high as Elevation 177 m south of the river. The total width of the floodplain is about 250 m and the floodplain is well vegetated with many bushes and shrubs and deciduous trees which rise several metres above the existing bridge deck. Based on the General Arrangement drawing, the water level in the Welland River was measured at Elev. 170.87 m on January 27, 2018, which corresponds to a water depth of about 5 m. It is understood based on information contained in the RFP that the flow direction in the river is subject to reverse due to nearby hydro power operations.

Grassy Brook Culvert is located approximately 140 m south of the south abutment and consists of a 6.1 m wide by 78 m long concrete box culvert.

Based on available historical information obtained from MTO's GEOCRES database, the QEW SBL south approach embankment (between approximately Stations 10+200 and 10+310) has a history of settlement and lateral movement (GEOCRES No. 30M3-212). This area has been rehabilitated unsuccessfully over the years, with resurfacing carried out in 1965 and again in 1983. Finally, in 1994, a stabilizing berm was constructed to arrest further distress and lateral movement of the approach embankment.

Based on asphalt thickness at the site, being thicker closer to the abutments, it is likely that settlement has been ongoing over the lifespan of the approaches to the bridge, particularly where substantial filling was carried out.

3.0 INVESTIGATION PROCEDURES

3.1 Previous (2018) Investigation

Geotechnical investigation and testing were completed in 2018 by Thurber Engineering Ltd. (Thurber) at the site of the existing Welland River Bridges and the Foundation Investigation and Design Report (FIDR) titled "*Foundation Investigation and Design Report, Replacement of Welland River Twin Bridge Structure, Queen Elizabeth Way (QEW), City of Niagara Falls, Ontario*", dated October 2, 2018, GEOCRES No. 30M03-307, was provided in the RFP.

During the 2018 investigation, 27 boreholes (Boreholes 18-1 to 18-27) and two Cone Penetration tests (CPTs, designated as SCPT18-01 and SCPT18-02) were advanced at the site as shown on Drawing 1. Field and laboratory methods used in the development of the data are described in the above-noted report and are in general accordance with typical MTO procedures. The previous borehole records, CPT records and laboratory test data are included in Appendices A and B.

The borehole locations and ground surface elevations of the previous investigation are show on the original Record of Borehole sheets in Appendix A, and in plan on Drawing 1. The coordinates, ground surface elevations and drilled/investigated depths of the boreholes and CPTs carried out in the previous investigations are summarized in the table below.

Borehole/CPT No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole/CPT Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
18-01	4,767,217.4 (43.044147)	335,654.1 (-79.121259)	183.7	32.1
18-02	4,767,191.5 (43.043907)	335,631.7 (-79.121663)	183.1	22.0
18-03	4,767,239.4 (43.044355)	335,651.3 (-79.121260)	184.3	38.7
18-04	4,767,238.2 (43.044343)	335,634.7 (-79.121488)	184.2	38.5
18-05	4,767,237.6 (43.044330)	335,627.0 (-79.121719)	184.2	39.8
18-06	4,767,304.6 (43.044934)	335,650.6 (-79.121340)	175.6	28.3

Borehole/CPT No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole/CPT Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
18-07	4,767,278.7 (43.044702)	335,638.2 (-79.121495)	175.4	28.6
18-08	4,767,276.5 (43.044683)	335,616.2 (-79.121766)	176.0	28.4
18-09	4,767,350.3 (43.045346)	335,648.7 (-79.121363)	175.0	29.6
18-10	4,767,343.7 (43.045286)	335,631.3 (-79.121577)	175.3	29.8
18-11	4,767,337.2 (43.045230)	335,614.0 (-79.121788)	175.3	29.3
18-12	4,767,397.8 (43.045773)	335,643.8 (-79.121420)	172.2	24.7
18-13	4,767,383.6 (43.045788)	335,623.2 (-79.121632)	173.8	25.4
18-14	4,767,394.0 (43.045741)	335,608.7 (-79.121851)	173.1	25.4
18-15	4,767,487.9 (43.046584)	335,641.6 (-79.121441)	171.4	23.7
18-16	4,767,490.0 (43.046604)	335,622.8 (-79.121673)	171.5	24.0
18-17	4,767,483.5 (43.046547)	335,603.1 (-79.121914)	171.3	23.4
18-18	4,767,538.9 (43.047048)	335,628.4 (-79.121526)	180.1	32.4
18-19	4,767,537.5 (43.047036)	335,611.4 (-79.121755)	180.1	32.6
18-20	4,767,536.9 (43.047024)	335,603.0 (-79.121986)	180.1	32.3
18-21	4,767,592.1 (43.047527)	335,623.9 (-79.121586)	178.9	23.1
18-22	4,767,565.9 (43.047284)	335,601.8 (-79.121990)	179.4	19.9

Borehole/CPT No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole/CPT Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
18-23	4,767,641.2 (43.047968)	335,603.2 (-79.121829)	177.7	26.1
18-24	4,767,139.9 (43.043458)	335,646.2 (-79.121369)	181.5	32.5
18-25	4,767,090.2 (43.043003)	335,660.7 (-79.121229)	180.8	26.5
18-26	4,767,040.3 (43.042554)	335,665.2 (-79.121177)	178.7	26.0
18-27	4,767,690.4 (43.048409)	335,601.8 (-79.121919)	176.9	24.4
SCPT18-01	4,767,240.1 (43.044363)	335,633.6 (-79.121557)	183.8	37.1
SCPT18-02	4,767,540.6 (43.047069)	335,618.5 (-79.121725)	180.0	23.7

3.2 Current (2019) Investigation

Golder carried out an additional investigation at the site between May 27 and 29, 2019. During this time two boreholes, designated as Boreholes 19-1 and 19-2, were advanced at the south and north abutments, respectively. The locations of the boreholes are shown on Drawing 1 and the borehole records are provided in Appendix C. Lists of abbreviations and symbols are also provided in Appendix C to assist in the interpretation of the borehole records.

The field work was carried out using a CME-75 truck-mounted drill rig. The equipment was supplied and operated by Davis Drilling Ltd. of Milton, Ontario. The boreholes drilled by the truck-mounted drill rigs were advanced through the overburden using 190 mm outer diameter (O.D.) hollow-stem augers as well as tricone equipment. Soil samples were generally obtained at intervals of depth of 0.75 m and 1.5 m using a 50 mm O.D. split-spoon sampler operated by an automatic hammer on the drill rigs, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586[1]). Field vane shear tests were carried out in the cohesive soils for assessment of undrained shear strength (ASTM D2573[2]) using an MTO standard N-size vane. The results of the in-situ field tests (i.e. SPT “N” values and undrained shear strengths from the field vane tests) as presented on the Record of Borehole sheets and in Section 4 are uncorrected.

[1] ASTM D1586-11 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils, ASTM International, PA, 2011

[2] ASTM D2573-15 Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils, ASTM International, PA, 2015

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations. The boreholes were backfilled upon completion in accordance with Ontario Regulation 903, Wells (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in-situ testing operations, logged the boreholes, and examined the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratories where the samples underwent further visual examination and geotechnical laboratory testing. All of the geotechnical laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples.

The borehole locations and ground surface elevations for the current investigation were measured using a GPS unit (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and horizontal directions. The locations provided on the borehole records and shown on Drawing 1 are positioned relative to the NAD83 CSRS CBNv6-2010.0 coordinates system, and the ground surface elevations are referenced to Geodetic datum. The borehole/CPT locations, ground surface elevations, and drilled/investigation depths from the current investigation are summarized in the table below.

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole/CPT Depth (m)
	Northing (m) (Latitude, °)	Easting (m) (Longitude, °)		
19-1	4,767,231.3 (43.044284)	335,626.2 (-79.121648)	184.0	36.6
19-2	4,767,546.6 (43.047122)	335,628.3 (-79.121605)	179.9	28.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This project area is located within the Haldimand Clay Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1984)[3]. The Haldimand Clay Plain is characterized by a broad undulating plain which occupies nearly all of the Niagara peninsula and covers an area of about 2,000 square kilometers that is flat to undulating and is characterized by glaciolacustrine clay deposits overlying glacial till underlain by shale and dolostone bedrock of the Salina formation.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes and CPTs advanced during the previous and current investigations, together with the results of the laboratory tests and in situ testing

[3] Chapman, L.J. and Putman, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

carried out, are presented on the Record of Borehole sheets (i.e. “borehole records”), CPT report and laboratory test sheets in Appendices A to D.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile and cross-sections on Drawings 1 to 5 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and CPT results. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change and moreover, the interpreted stratigraphy shown on Drawings 1 to 5 represents a simplification of the subsurface conditions. Furthermore, subsurface conditions will vary between and beyond the borehole and CPT locations.

In general, the subsurface conditions in the area of the proposed bridge replacement structure consist of asphalt or topsoil and fill associated with the construction of the existing highway embankment and bridge structures underlain by a native deposit of clayey silt to clay which is subsequently underlain by deposits of clayey silt till and sandy silt to silty sand till. Deposits of peat and organic silt were encountered beneath the fill at some locations near the river. Silt to sand deposits were encountered both as layers within and also beneath the glacial till. The overburden was underlain by dolostone bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes from the previous and current investigations is provided in the following sections.

4.2.1 Asphalt/Topsoil

Boreholes 18-1 to 18-5, 18-18 to 18-27, 19-1, and 19-2 were advanced on the highway and encountered asphalt with a thickness between approximately 75 mm and 250 mm. Borehole 18-20 encountered an approximately 250 mm thick layer of concrete beneath the asphalt. Boreholes 18-6 to 18-17 encountered topsoil with a thickness between approximately 50 mm and 175 mm.

4.2.2 Fill

Non-cohesive fill was encountered underlying the asphalt/topsoil in Boreholes 18-1 to 18-8, 18-18 to 18-27, 19-1 and 19-2. The non-cohesive fill is variable in composition and consists of sandy silt to sand and gravel. A cohesive silty clay fill was encountered underlying the asphalt/topsoil in Boreholes 18-11, 18-12, 18-15, 18-16 and 19-1. The cohesive fill was also encountered underlying the non-cohesive fill in Boreholes 18-1 to 18-7 and 18-18, 18-19, 18-20, 18-21, 18-23, 18-24, 18-25, 18-26, and 19-2. The surface of the fill was encountered between Elevations 171.3 m and 184.1 m and the total fill thickness was between 0.7 m and 10 m.

The SPT “N”-values measured within the non-cohesive fill range from 8 blows and 63 blows per 0.3 m of penetration, indicating that the non-cohesive fill has a loose to very dense state of compactness. Two SPT “N”-values were greater than 50 blows likely indicative of an obstruction. The SPT “N”-values measured within the cohesive fill range from 3 blows to 29 blows per 0.3 m of penetration with one value of 47 blows per 0.3 m of penetration. In-situ field vane tests carried out within the cohesive fill measured undrained shear strengths ranging from about 61 kPa to 111 kPa with calculated sensitivities of about 3 and one sensitivity of 20. The field vane test results along with the measured SPT “N”-values indicate that the cohesive fill layer has a soft to hard consistency and is generally firm to very stiff.

The natural water content measured on samples of the non-cohesive fill ranges from about 2 per cent to 26 per cent. The natural water content measured on samples of the cohesive fill ranges from about 16 per cent to 49 per cent.

Grain size distribution tests were carried out on 19 samples of fill and the results are shown on Figures B1 to B5 in Appendix B and Figure D1 in Appendix D.

Atterberg limits testing was carried out on eight samples of cohesive fill and measured liquid limits ranging from about 36 per cent to 51 per cent, plastic limits ranging from about 18 per cent to 23 per cent, and plasticity indices ranging from about 18 per cent to 28 per cent. The Atterberg limits test results are shown on the plasticity chart on Figures B1 to B5 in Appendix B and Figure D2 in Appendix D and indicate that the material is typically classified as a silty clay of intermediate plasticity.

4.2.3 Peat

A 0.7 m thick deposit of dark brown peat containing roots and rootlets was encountered beneath the fill in Borehole 18-15 and 0.2 m of organics was encountered in Borehole 18-18. The top of the deposit was encountered between Elevation 169.9 m and 170.0.

An SPT “N”-value of 2 blows per 0.3 m of penetration was measured within the peat, suggesting a very soft to soft consistency.

The natural water content measured on a sample of the peat deposit is about 256 per cent.

4.2.4 Organic Silt

A deposit of organic silt was encountered underlying the topsoil in Borehole 18-9, beneath the peat in Borehole 18-15, and beneath the fill Boreholes 18-16 and 18-17. The top of the organic silt deposit ranges from Elevation 169.2 m to 174.9 m and the thickness is between 0.7 m and 3.2 m.

The SPT “N”-values measured within the organic silt deposit range from 0 blows (i.e. weight of hammer) to 6 blows per 0.3 m of penetration. In-situ field vane tests carried out within the organic silt measured undrained shear strengths of about 9 kPa and 27 kPa with one calculated sensitivity of about 6. The field vane test results along with the measured SPT “N”-values indicate that the organic silt deposit has a very soft to firm consistency.

The natural water content measured on samples of the organic silt deposit ranges from about 61 per cent to 147 per cent. Atterberg limits testing was carried out on one sample of the organic silt deposit and the measured liquid limit is about 76 per cent, the plastic limit is about 39 per cent and the corresponding plasticity index is about 37 per cent. The Atterberg limits test result is shown on the plasticity chart on Figure B25 in Appendix B and indicate that the material is classified as an organic silt of high plasticity.

4.2.5 Clayey Silt to Clay

In all of the boreholes advanced at the site, a cohesive deposit ranging in composition from clayey silt to clay, trace to some sand was encountered below the topsoil, fill or organic silt. The top of the deposit was encountered between Elevation 166.7 m and 177.5 m. The thickness of the deposit ranges between 8.5 m and 23.0 m. Borehole 18-2 was terminated within this deposit. During drilling, cobbles and/or boulders were encountered in some boreholes in the lower portion of the deposit, and their inferred depths are shown on the borehole records in

Appendix A. A 275 mm thick boulder was encountered in Borehole 18-11 at Elevation 156.8 m and rock coring equipment had to be utilized to advance through this zone.

The SPT "N"-values measured within the cohesive deposit range between 0 blows (weight of rods or hammer) to 54 blows per 0.3 m of penetration with 4 "N"-values in the lower portion of the deposit measuring greater than 100 blows per 0.3 m of penetration. In-situ field vane tests carried out within the cohesive stratum measured undrained shear strengths ranging from about 18 kPa to greater than 115 kPa with a calculated sensitivity between about 3 and 20. One in-situ field vane measured about 4 kPa just below the organic silt deposit in Borehole 18-15. The field vane test results along with the measured SPT "N"-values indicate that the clayey silt to clay deposit has a very soft to hard consistency and is generally soft to very stiff. In general, the upper and lower portions of the deposit were stiff to hard and the central portion of the deposit was soft to stiff.

The results of grain size distribution tests carried out on 90 samples of the clayey silt to clay deposit are shown on Figures B6 to B18 in Appendix B and Figure D3 in Appendix D. Atterberg limits tests were carried out on 60 samples of this deposit and measured liquid limits range between about 25 per cent and 55 per cent, plastic limits range between about 13 per cent and 23 per cent, and plasticity indices range between about 8 per cent and 33 per cent. These results, which are plotted on plasticity charts on Figures B26 to B34 in Appendix B and Figure D4 in Appendix D, indicate that the material ranges in classification from a clayey silt of low plasticity to clay of high plasticity. The natural water content measured on samples of the cohesive deposit ranges from about 10 per cent to 70 per cent and generally between 15 per cent and 50 per cent.

A 0.7 m to 1.5 m thick silt interlayer was encountered within the clayey silt to clay deposit at Elevation 164.3 m and 163.0 m in Boreholes 19-2 and 18-18, respectively. The partial SPT "N"-values measured in the silt interlayer are 3 blows and 18 blows per 0.3 m of penetration indicating a very loose to compact state of compactness. The results of a grain size distribution test carried out on a sample of the silt is shown on Figure D5 in Appendix D. One Atterberg limits test was carried out on a sample of this silt interlayer and the measured liquid limit is about 23 per cent, the plastic limit is about 21 per cent, and the corresponding plasticity index is about 2 per cent. These results, which are plotted on a plasticity chart on Figure D6 in Appendix D, indicate that this interlayer is classified as a silt of slight plasticity. The natural water content measured on two samples of the silt deposit are about 22 per cent and 29 per cent.

Laboratory consolidation tests were carried out on eight samples of the clayey silt to clay deposit obtained from Shelby tube samples. Preconsolidation stresses ranging between 180 kPa and 1170 kPa were estimated from the void ratio versus logarithmic stress plots, indicating an overconsolidation ratio (OCR) between 1.1 and 6.8.

Bulk unit weights ranging between about 19.3 kN/m^3 and 20.2 kN/m^3 and a specific gravity between 2.72 and 2.79 were measured on the consolidation test specimens. Details of the consolidation test results are included in Appendix B and the test results are summarized below. The compressibility characteristics will vary with depth in accordance with the moisture content and shear strength profiles.

Borehole Sample No.	Sample Depth/ Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	C_c	C_r	e_o	c_v^* (cm^2/s)
Borehole 18-01 Sample TW1	12.5 m/ 171.2 m	170	1170	1000	6.8	0.08	0.018	0.82	2.6×10^{-3}
Borehole 18-04 Sample TW2	15.4 m/ 168.8 m	200	275	75	1.4	0.29	0.02	0.90	8.5×10^{-3}
Borehole 18-11 Sample TW1	7.2 m/ 168.1 m	105	185	80	1.8	0.29	0.009	0.81	2.0×10^{-3}
Borehole 18-14 Sample TW1	4.9 m/ 168.2 m	70	230	160	3.1	0.29	0.033	0.97	3.4×10^{-3}
Borehole 18-19 Sample TW1	9.5 m/ 170.6 m	160	565	405	3.5	0.17	0.006	0.73	1.3×10^{-3}
Borehole 18-21 Sample TW2	14.0 m/ 164.9 m	200	260	60	1.3	0.17	0.006	0.66	1.5×10^{-2}
Borehole 18-24 Sample TW1	12.5 m/ 169.0 m	200	285	85	1.4	0.13	0.022	0.78	1.4×10^{-2}
Borehole 18-25 Sample TW1	12.5 m/ 168.3 m	195	270	75	1.4	0.24	0.004	0.79	7.1×10^{-3}

*For stress range between approximately the in-situ effective overburden stress and final stress due to proposed grade raise.

where: σ_{vo}' is the effective overburden stress in kPa
σ_p' is the preconsolidation stress in kPa
OCR is overconsolidation ratio
C_c is the compression index
C_r is the recompression index
e_o is initial void ratio
c_v is the coefficient of consolidation in cm^2/s

The ratio of preconsolidation pressures to vertical effective stress (OCR = P′c / P′0) derived from the oedometer test results indicate that the silty clay is overconsolidated. The OCR ratio inferred from CPTUs shows a heavily overconsolidated crust with OCR generally ranging from about 10 to 30. Beneath the crust, the silty clay is slightly overconsolidated with inferred OCR ranging between 1 and 2.

Two CPTUs (SCPT18-01 and SCPT18-02) were advanced at the north and south abutments for measurement of the tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). Two in-situ pore water pressure dissipation tests were carried out in the cohesive deposit during the CPT testing for assessing the horizontal coefficient of consolidation, c_h, at a specific horizon within the clayey silt to clay deposit. The results of the CPTU testing is presented in Appendix B. The coefficient of consolidation in the horizontal direction (c_h) obtained from the results of the dissipation tests is summarized below.

CPT	Depth (m)	c_h (cm^2/s)
SCPT18-01	12.8	5.07×10^{-3}
	15.8	2.35×10^{-2}
SCPT18-02	10.8	2.35×10^{-2}
	13.8	8.24×10^{-3}

The results of two Consolidated Isotopically Undrained (CIU) triaxial tests carried out on selected samples of the silty clay are presented in Appendix B. The test results indicate that silty clay has an effective cohesion of 3 kPa and a drained friction angle ranging from 23 degrees to 27 degrees.

4.2.6 Clayey Silt (Till)

A 1.0 m to 6.4 m thick glacial till deposit consisting of clayey silt, some to with sand, trace to some gravel, was encountered underlying the clayey silt to clay deposit in Boreholes 18-3 to 18-6 and 19-1. In Borehole 18-8, a silty clay till deposit was inferred below the silty clay deposit although a sample was not recovered. The surface of the clayey silt till was encountered between Elevation 152.2 m to 156.1 m. During drilling, cobbles and/or boulders were encountered, and their inferred depths are shown on the borehole records in Appendix A. Previous experience in the region indicates that the glacial deposits contain cobbles and boulders that are not identified by conventional drilling, sampling, and laboratory testing methods.

The SPT "N"-values measured within the clayey silt till deposit range from 12 blows to 47 blows per 0.3 m of penetration with two "N"-values greater than 50 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

The result of five grain size distribution tests carried out on samples from the clayey silt till deposit are shown on Figure B19 in Appendix B. The results of an Atterberg limits test measured a liquid limit of about 28 per cent, a plastic limit of about 13 per cent and a plasticity index of about 15 per cent. This result, which is plotted on a plasticity chart on Figure B35 in Appendix B, indicates that the sample tested can be classified as a clayey silt of low plasticity. The natural water contents measured on samples of the clayey silt till deposit range from 9 per cent to 35 per cent.

4.2.7 Sandy Silt to Silty Sand (Till)

A 0.7 m to 8.3 m thick deposit of glacial till consisting of sandy silt to silty sand till, trace gravel to gravelly, was encountered in Boreholes 18-1, 18-4, 18-5, 18-7, 18-9 to 18-27, 19-1, and 19-2 underlying the clayey silt to clay or clayey silt till deposits. The surface of the sandy silt to silty sand till was encountered between Elevation 150.4 m to 161.6 m. Boreholes 18-21 to 18-27 were terminated in the sandy silt to silty sand till. A 350 mm thick boulder was encountered in Borehole 18-19 at Elevation 160.5 m and rock coring equipment had to be utilized to advance through this zone. During drilling, cobbles and/or boulders were encountered, and their inferred depths are shown on the borehole records in Appendix A. Further, several indications of gravel and cobbles were also noted within the deposit. Previous experience in the region indicates that the glacial deposits contain cobbles and boulders that are not identified by conventional drilling, sampling, and laboratory testing methods.

The SPT "N"-values measured within the sandy silt to silty sand till deposit range between 2 blows and 106 blows per 0.3 m of penetration and several that were greater than 100 blows for 0.3 m of penetration, indicating a very loose to very dense state of compactness. In general, the SPT "N"-values were above 30 blows per 0.3 m of penetration, indicating the deposit typically has a dense to very dense state of compactness.

The results of grain size distribution tests carried out on 15 samples of the sandy silt to silty sand till deposit are shown on Figures B20 to B22 in Appendix B and Figure D7 in Appendix D. Atterberg limits tests were carried out on four samples of this deposit. The Atterberg limits tests measured liquid limits between about 17 per cent and 20 per cent, plastic limits between about 10 per cent and 11 per cent, and plasticity indices between about 7 and 9 per cent. These results, which are plotted on a plasticity chart on Figure B36 in Appendix Band 19-2, indicate that the fines portion of the sample tested can be classified as a clayey silt to silt of low plasticity. The natural water content measured on samples of the sandy silt to silty sand till deposit ranges from about 7 per cent to 27 per cent.

4.2.8 Silt to Sand

A non-cohesive deposit ranging in composition from silt to sand was encountered underlying the silty clay in Borehole 18-1, beneath the glacial till in Boreholes 18-10 and 18-15 to 18-19 and 19-2 and interlayered within the glacial till in Boreholes 18-18 and 18-24 to 18-26. Borehole 19-2 terminated in the sand deposit. The surface of the deposit was encountered between Elevation 153.8 m and 167.0 m and the thickness of the deposit is between 0.5 m and 5.9 m. During drilling, gravel, cobbles and/or boulders were encountered, and their inferred depths are shown on the borehole records in Appendix A.

The SPT "N"-values measured within the silt to sand deposit range from 18 blows to 75 blows per 0.3 m of penetration and several that were greater than 100 blows per 0.3 m of penetration, indicating the deposit has a compact to very dense state of compactness.

The results of grain size distribution tests carried out on ten samples from the silt to sand deposit are shown on Figure B23 in Appendix B and Figure D8 in Appendix D. The natural water content measured on samples of the silt to sand deposit ranges from about 8 per cent to 28 per cent.

4.2.9 Dolostone Bedrock

Bedrock was encountered in Boreholes 18-3 to 18-20 and 19-1. The surface of the bedrock was encountered between Elevation 148.6 m and 151.5 m. The depth to and elevation of the top of the bedrock, cored length and bottom of borehole elevation are presented in the following table.

Borehole	Depth to Bedrock (m)	Bedrock Elevation (m)	Cored Length (m)	Bottom of Borehole Elevation (m)
19-1	33.5	150.5	3.1	147.4
18-03	35.4	148.9	3.3	145.6
18-04	35.3	148.9	3.2	145.7
18-05	35.2	149.1	4.7	144.4
18-06	25.2	150.4	3.1	147.3
18-07	25.3	150.1	3.4	146.7
18-08	24.8	151.1	3.5	147.6
18-09	26.4	148.6	3.2	145.4
18-10	26.3	149.0	3.5	145.5
18-11	26.3	149.0	3.0	146.0
18-12	21.5	150.7	3.1	147.6
18-13	22.3	151.5	3.0	148.5
18-14	22.3	150.8	3.1	147.7
18-15	20.4	151.0	3.3	147.7
18-16	20.7	150.9	3.4	147.5
18-17	20.0	151.3	3.4	147.9
18-18	29.1	151.0	3.4	147.6
18-19	29.1	151.0	3.5	147.5
18-20	29.2	150.9	3.1	147.8

The bedrock generally consists of slightly weathered, thinly bedded, grey, fine to medium grained, faintly porous, strong to very strong dolostone of the Salina Formation. Details of the bedrock coring and core descriptions are shown on the Borehole records in Appendix A and on the Record of Drillhole sheets in Appendix C. Photographs of the recovered rock core samples are presented in Appendix B and on Figure D9 in Appendix D.

The degree of weathering of the bedrock samples, and the strength classification of the intact rock mass based on field identification are described in accordance with the International Society for Rock Mechanics (ISRM)[4] standard classification system.

The Total Core Recovery (TCR) ranges from 67 per cent to 100 per cent but is generally 100 per cent. The Solid Core Recovery (SCR) ranges from 38 per cent to 100 per cent. The Rock Quality Designation (RQD) ranges from 13 per cent to 100 per cent, indicating a rock mass of very poor to excellent quality as per Table 3.10 of the CFEM (2006)[5], but generally of fair to excellent quality.

Unconfined Compression (UC) tests (ASTM D7012)[6] were carried out on selected core samples of the dolostone bedrock from Borehole 19-1. The uniaxial compressive strength (UCS) of the intact samples are summarized below and the details are presented on the Rock Laboratory Test Results in Appendix D. Based on the UC test results and in accordance with Table 3.5 in CFEM (2006), the dolsotone bedrock is classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa).

Borehole No.	Run No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)
19-1	1	34.5 - 34.6	149.5 – 149.4	60.6
19-1	1	34.9 - 35.0	149.1 – 149.0	115.1
19-1	2	35.1 – 35.2	148.9 – 148.8	81.0
19-1	2	35.8 – 35.9	148.2 – 148.1	130.4

Axial and diametric Point Load Tests (PLT) were completed in the previous investigation on bedrock samples obtained from Boreholes 18-3 to 18-20. The UCS of the bedrock is estimated from the results of the point load tests. The results indicate that the correlated UCS ranges between about 71 MPa and 244 MPa within the strong to very strong classification. The detailed test results are contained in Appendix B and a summary of the correlated unconfined compressive strength results are presented in the table below.

Borehole	Sample Depth	Average Correlated UCS (MPa)	Borehole	Sample Depth	Average Correlated UCS (MPa)
	From (m) To (m)			From (m) To (m)	
18-03	35.1 – 36.2	221.4	18-11	28.1 – 29. 3	208.1
18-03	36.2 – 37.7	98.5	18-12	21.4 – 22.5	121.9
18-03	37.7 – 38.7	148.6	18-12	22.5 – 24.0	233.9
18-04	35.3 - 36.8	147.8	18-12	24.0 – 24.7	157.1

[4] International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech.Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

[5] Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

[6] ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Borehole	Sample Depth From (m) To (m)	Average Correlated UCS (MPa)	Borehole	Sample Depth From (m) To (m)	Average Correlated UCS (MPa)
18-04	36.8 – 37.8	123.6	18-13	22.1 – 23.6	169.2
18-04	37.8 – 38.5	180.0	18-13	23.6 – 24.9	197.7
18-05	35.2 – 36.2	71.2	18-13	24.9 – 25.3	193.0
18-05	36.2 – 36.9	226.5	18-14	22.1 – 22.6	130.0
18-05	36.9 – 38.4	135.9	18-14	22.6 – 24.2	142.8
18-05	38.4 – 39.8	167.8	18-14	24.2 – 25.4	106.3
18-06	25.1 – 25.4	118.5	18-15	20.3 – 21.3	78.4
18-06	25.4 – 26.8	188.7	18-15	21.3 – 22.3	222.3
18-06	26.8 – 28.3	178.2	18-15	22.3 – 23.7	160.1
18-07	25.2 – 25.7	171.5	18-16	20.7 – 21.3	149.2
18-07	25.7 – 27.3	168.8	18-16	21.3 – 22.3	176.1
18-07	27.3 – 28.7	158.8	18-16	22.3 – 24.0	191.9
18-08	24.8 – 26.4	126.7	18-17	19.9 – 21.2	145.2
18-08	26.4 – 27.9	159.1	18-17	21.2 – 22.7	229.4
18-08	27.9 - 28.3	193.9	18-17	22.7 – 23.4	187.6
18-09	26.3 – 27.8	193.3	18-18	29.1 – 30.2	135.1
18-09	27.8 – 29.3	136.0	18-18	30.2 – 31.7	172.9
18-09	29.3 – 29.6	244.3	18-18	31.7 – 32.4	141.9
18-10	26.1 – 27.7	153.8	18-19	29.1 – 30.2	153.1
18-10	27.7 – 29.0	148.1	18-19	30.2 – 31.7	211.5
18-10	29.0 – 29.8	113.0	18-19	31.7 – 32.6	172.4
18-11	26.1 – 27.2	207.0	18-20	29.2 – 30.5	179.0
18-11	27.2 – 28.1	157.6	18-20	30.5 – 32.3	139.6

4.2.10 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations. The details of these measurements are shown on the borehole records contained in Appendices A and C; however, it is noted that these measurements are not considered to represent the stabilized groundwater level at the site.

Standpipe piezometers were installed in Boreholes 18-2, 18-3, 18-5, 18-6, 18-8, 18-9, 18-11, 18-12, 18-14, 18-15, 18-17, 18-18, 18-20 and 18-21 to permit monitoring of groundwater levels. Details of the piezometer installation and measured groundwater levels are shown on the borehole records in Appendix B. The measured groundwater levels are summarized below:

Borehole No.	Location	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)	Screened Deposit
18-02	South Approach	183.1	12.6	170.5	12/07/2018	Clayey silt to silty clay
			9.1	174.0	28/05/2019	
18-03	South Abutment	184.3	5.1	179.2	12/07/2018	Clayey silt to silty clay till/ dolostone bedrock
			12.0	172.3	28/05/2019	
18-05		184.2	12.6	171.6	12/07/2018	Silt and sand till/dolostone bedrock
			11.3	172.9	28/05/2019	
18-06	Pier 4	175.6	3.9	171.7	24/07/2018	Silty clay till/dolostone bedrock
18-08		176.0	4.6	171.4	24/07/2018	Clayey silt to silty clay
18-09	Pier 3	175.0	-	-	-	Silt and sand till/dolostone bedrock
18-11		175.3	3.8	171.5	24/07/2018	
18-12	Pier 2	172.2	0.8	171.4	24/07/2018	Silt and sand till
18-14		173.1	2.9	170.2	24/07/2018	Clayey silt to silty clay/silt and sand till
18-15	Pier 1	171.4	0.1	171.3	24/07/2018	Clayey silt to silty clay
18-17		171.3	0.2	171.1	24/07/2018	Silt and sand till/silty sand

Borehole No.	Location	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	**Date** (dd/mm/yyyy)	Screened Deposit
18-18	North Abutment	180.1	7.6	172.5	28/05/2019	Silt and sand till/silt and sand
18-20		180.1	6.6	173.5	12/07/2018	Silt and sand till
			8.4	171.7	28/05/2019	
18-21	North Approach	178.9	7.0	171.9	12/07/2018	Silt and sand till
			12.1	166.8	28/05/2019	

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year. The normal water level in the Welland River as shown on the General Arrangement drawing is Elevation 170.87 m on January 27, 2018. The river water level is subject to fluctuations and the flow is known to reverse direction as a result of nearby hydro power operations.

4.2.11 Analytical Testing Results

Analytical testing was carried out on selected soil samples recovered from Boreholes 18-3, 18-10, 18-16, and 18-20 as part of the previous investigation. The soil samples were submitted to AGAT Laboratories of Mississauga, Ontario for corrosivity testing. Detailed analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix B and summarized below.

Borehole No.	**Sample ID**	**Depth (m)**	**Parameters**				
			Resistivity (ohm-cm)	**Electrical Conductivity (mS/cm)**	**Soluble Sulphate (So_4) Content (µg-g)**	**Chlorides (CL) Content (µg-g)**	**pH (pH)**
18-03	5	3.0 – 3.7	741	1.35	1190	264	7.66
18-10	9	9.1 – 9.8	3,020	0.331	197	39	7.86
18-16	12	15.2 – 15.8	2,840	0.352	260	24	7.95
18-20	11	13.7 – 14.3	2,460	0.407	310	15	7.88

5.0 CLOSURE

Ms. Katelyn Nero and Mr. Michael Bentley supervised Golder's 2019 field investigation. This Foundation Investigation Report was prepared by Mr. Yusuf Soliman, B.A.Sc., E.I.T., a geotechnical engineering intern with Golder and the technical aspects were reviewed by Ms. Sarah Poot, P.Eng., Associate of Golder and the Senior Foundation Engineer/Lead for this project. Ms. Lisa Coyne, P.Eng., a Principal and MTO Foundations Designated Contact for Golder, conducted an independent quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER
(SITE NOS. 34-65/1 & 2)
QUEEN ELIZABETH WAY (QEW), CITY OF NIAGARA FALLS, ONTARIO
MTO WP 2430-15-00, CONTRACT NO. DB 2018-2013

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering design recommendations for the proposed Welland River Twin Bridge replacement structure and associated raised approach embankments. The Foundation Investigation and Design Report and these interpretations and recommendations are intended for the use of the design-build team and shall not be used or relied upon for any other purpose or by any other parties.

The recommendations provided herein are based on an interpretation of the factual data obtained from the boreholes and CPTs advanced by others on behalf of MTO prior to tendering of this DB Contract, along with subsequent boreholes and testing completed at this site as part of this Contract. The interpretation and recommendations are intended to provide the design-build team with sufficient information to confirm the selected design and construction alternatives and to design the bridge foundations and raised approach embankments. This report was prepared in the context of a design-build contract. Therefore, general recommendations related to construction are also provided within this report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design and execution of the design-build project. The Contractor must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods and scheduling. These recommendations are provided subject to continued involvement of Golder throughout design and subsequent construction. If design changes and/or conditions exposed during construction are different then as understood at the time this report was prepared, based on the subsurface explorations and testing described herein, Golder must be consulted to update, supplement or otherwise revise these recommendations if and as necessary.

6.1 General

The existing Welland River Bridge structures are each 290 m long, eighteen-span, steel-girder composite bridge structures that carry the QEW over the Baden Powell Trail, the Canadian Pacific Rail (CPR), the Welland River and Oakwood Drive, from south to north, as shown on Drawing 1. It is understood from available information that the abutments and piers are supported on driven timber piles that are 13.7 m (45 feet) long, generally vertical although some were indicated to be battered. The foundation type and founding depth of the remaining abutments and bents are unknown, however it is likely that they are similarly supported on driven timber piles implied (for assumed 13.7 m long piles) to be founded within firm to very stiff silty clay deposit or possible the dense to very dense till deposits. The approach embankments at the north and south abutments are approximately 7.2 m and 8.5 m high above the original ground in the valley, respectively. The grade of the QEW is at about Elevations 180.0 m and 184.0 m at the north and south abutments, respectively and the ground surface in the relatively flat, 250 m wide Welland River valley floodplain ranges from about Elevation 172 m immediately north of the river up to as high as Elevation 177 m south of the river. The Grassy Brook Culvert is located approximately 140 m south of the south abutment and consists of a 6.1 m wide by 78 m long concrete box culvert.

Based on available historical information obtained from MTO's GEOCRES database, the QEW SBL south approach embankment (between approximately Stations 10+200 and 10+310) has a history of settlement and lateral movement (GEOCRES No. 30M3-212). This area has been rehabilitated unsuccessfully over the years, with resurfacing carried out in 1965 and again in 1983. Finally, in 1994, a stabilizing berm was constructed to arrest further distress and lateral movement of the approach embankment.

Also, based on asphalt thickness at the site, being thicker closer to the abutments, it is considered likely that settlement has been ongoing over the lifespan of the approaches to the bridge, particularly where substantial filling was carried out.

Based on the General Arrangement (GA) drawing in progress as provided by Parsons on May 28, 2019, the proposed replacement bridge will consist of a five-span welded steel plate girder and concrete deck structure with an overall length of about 300 m supported on driven steel H-piles. The proposed bridge will be about 28 m wide to accommodate two northbound (i.e. EBL) and two southbound (i.e. WBL) lanes of traffic with median and shoulders. The largest span will be approximately 80 m where it spans the Welland River.

Based on the GA, it is understood that the proposed grade at the north and south abutments for the replacement structure will be at approximately Elevations 182.0 m and 186.0 m, respectively. The proposed grade at the north and south abutments will result in grade raises of 2.5 m and 2.1 m above the existing grade, respectively. The south approach grade raise will extend from the south abutment (Station 10+200) to approximately 410 m south of the south abutment (Station 10+610). The north approach grade raise will extend from the north abutment (Station 21+550) to approximately 290 m north of the north abutment (Station 21+260). Based on the information contained in the RFP, the grade raise along the approach embankments was to be mitigated using lightweight fill. As part of the bid evaluation and development, the Contractor selected cellular concrete as the lightweight fill material to be used for this project. The rationale for this choice as it related to the ground conditions is provided in subsequent sections of this report.

The existing twin bridges are proposed to be replaced using half-and-half staged construction. The Contractor has selected to remove the northbound (EBL) bridge first to allow for construction of the east half of the new replacement bridge while the traffic is routed to the existing southbound (WBL) bridge. Once the new east half is constructed and the traffic re-routed, the southbound bridge will be removed, and the west half of the new replacement bridge constructed. Temporary protection systems will be required along the QEW median to allow for excavation and removal of the existing abutments and pile caps and for construction of new abutment walls supported on new foundations. Temporary cofferdams will also be required to construct the piers. In order to limit excavation in the environmentally sensitive wetlands, the existing foundations (i.e. piles and pile caps) are to be cut off at the depth specified in the RFP.

6.2 Consequence and Site Understanding Classification

The new bridge and immediate approach embankments (within 20 m of the bridge) are to be designed in accordance with the current *Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC*, 2014). In this regard, the proposed foundation system and approach embankments are classified as having a "typical consequence level" associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the *CHBDC* (2014), the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the CHBDC (2014) have been used for design.

6.3 Foundations

Several foundation options including spread footings on native soils or engineered fill, drilled shafts (i.e. augered caissons) socketed into bedrock and steel H-piles or pipe piles driven into the very dense "100-blow" cohesionless deposits or to bedrock have been considered and evaluated for support of the new bridge structure.

The design-build team's preferred foundation type is steel H-piles driven to found within the very dense "100-blow" cohesionless deposits or to the surface of the dolostone bedrock. The foundation recommendations provided in this report are limited to this preferred foundation type, which is feasible from a foundations perspective to support the abutments and piers for the proposed new structure.

Shallow foundations are not suitable for supporting the abutments and piers as the existing silty clay fill and native clayey silt to clay deposit would not have sufficient geotechnical resistance to support the relatively large bridge loads as a result of the large spans and also in consideration of the potential for adverse effects on the existing and new bridges during staged construction. Drilled shafts were considered during the bid phase for support of the bridge but were not considered as economical as driven piles as they would likely require permanent liners due to the soft to stiff clay and high groundwater levels.

Consideration must be given to the presence of cobbles and boulders within the glacially derived silty clay till and sandy silt to silty sand till and lower silt to sand deposits at the site. Cobbles ranging in size from 75 mm to 175 mm were encountered and cored in most of the boreholes at the site within these deposits. A 275 mm thick boulder was encountered and cored in the lower part of the silty clay deposit in Borehole 18-11 (Pier 3) and a 350 mm thick boulder was encountered and cored in the silt and sand till deposit in Borehole 18-19 (North Abutment). It is recommended that pile tip reinforcement be incorporated into the design to reduce the potential for damage to the piles during driving into the very dense deposits as discussed further in Section 6.7.4. The risk of piles “hanging-up” on cobbles/boulders at this site is considered moderate.

6.3.1 Pile Founding Elevations

Steel H-piles should be driven into the very dense “100-blow” soil strata underlying the silty clay strata or to the surface of the strong to very strong dolostone bedrock. The exact surface of the “100-blow” soil is variable across the bridge site and is also variable across the foundation elements. We understand from the Contractor that their piling equipment can likely advance piles between 1 m and 2 m into the very dense/hard soil stratum. In this regard, an interpretation of the elevation of the surface of the “100-blow” material and the thickness of this material above the bedrock surface has been made. For design, the following pile tip elevations may be used, assuming approximately 1.5 m of penetration into “100-blow” soils or onto the dolostone bedrock.

Foundation Element	Reference Borehole Nos.	Anticipated Founding Stratum	Assumed Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Length (m)
North Abutment	18-18 (east) 18-19 (centre) 18-20 (west)	Very dense silty sand/silt and sand or silt and sand till	176.2	158.5 (east) to 154.0 (centre) to 152.5 (west)	17.7 to 23.7
Pier 1	18-15 (east) 18-16 (centre) 18-17 (west)	Bedrock	169.0	151.0 (east) to 150.9 (centre) to 151.3 (west)	17.7 to 18.1
Pier 2	18-12 (east) 18-13 (centre) 18-14 (west)	Very dense silt and sand till	169.7	151.0 (east) to 152.5 (centre) to 153.5 (west)	16.2 to 18.7

Foundation Element	Reference Borehole Nos.	Anticipated Founding Stratum	Assumed Underside of Pile Cap Elevation (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Length (m)
Pier 3	18-09 (east) 18-10 (centre) 18-11 (west)	Very dense silt and sand till/hard clayey silt	173.2	150.0 (east) to 150.7 (centre) to 150.7 (west)	22.5 to 23.2
Pier 4	18-06 (east) 18-07 (centre) 18-08 (west)	Bedrock	173.7	150.4 (east) to 150.1 (centre) to 151.1 (west)	22.6 to 23.3
South Abutment	18-03 (east) 18-04 (centre) 18-05 (west)	Bedrock	179.6	148.9 (east) to 148.9 (centre) to 149.1 (west)	30.5 to 30.7

At the South Abutment, the thickness of the “100-blow” material varies with depth within the lower deposits where the SPT “N” values range from greater than 100 blows per 0.3 m of penetration, to less than 50 blows to greater than 100 blows again. It is possible that at the location of Pier 4, some or all of the piles could “hang up” within the very dense/hard deposits above the bedrock surface, and the structural designer should consider this in their design, and base their design on potentially lower resistances than may be obtained on the bedrock surface. Further, it is recommended that a provision be made to accommodate varying pile lengths (shorter or longer) at all foundation elements. The depths indicated above should be considered minimum depths, to minimize the potential to have to splice short lengths of additional pile to achieve the design geotechnical resistances.

6.3.2 Geotechnical Axial Resistances

For steel H-piles founded within the very dense (“100-blow”) silt and sand till, silty sand, silt and sand, hard clayey silt or dolostone bedrock at the tip elevations given in Section 6.3.1, the factored axial geotechnical resistance at Ultimate Limit States (ULS) and the geotechnical reaction at Serviceability Limit State (SLS) (for 25 mm of settlement) may be taken as outlined in the table below for both HP 310x110 and HP 360x132 pile sections.

Foundation Element	Pile Size	Founding Stratum	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
North Abutment	HP 310x110	Very dense silty sand/silt and sand or silt and sand till	1,500	1,600
	HP 360x132		1,900	1,800
Pier 1	HP 310x110	Bedrock	3,700	N/A
	HP 360x132		5,000	N/A

Foundation Element	Pile Size	Founding Stratum	**Factored Ultimate Geotechnical Resistance** (kN)	**Factored Serviceability Geotechnical Resistance** (kN)
Pier 2	HP 310x110	Very dense silt and sand till	1,300	1,900
	HP 360x132		1,600	2,100
Pier 3	HP 310x110	Very dense silt and sand till/hard clayey silt	1,500	1,600
	HP 360x132		1,900	1,800
Pier 4	HP 310x110	Bedrock	3,700	N/A
	HP 360x132		5,000	N/A
South Abutment	HP 310x110	Bedrock	3,700	N/A
	HP 360x132		5,000	N/A
	HP 310x110	Very dense silt and sand till/hard silty clay till	1,500	1,800
	HP 360x132		1,900	2,000

It should be noted that for some piles, the factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and as a result, the SLS condition does not apply; however, where applicable, the values have been provided to provide full information for assessment of limit state conditions. The geotechnical resistances for piles founded on bedrock have been calculated based on the compressive strength of the bedrock. The structural limitation of the piles is typically 2,000 kN and 2,400 kN for HP 310x110 and HP 360x132 piles, respectively, and the structural designer must take account of this in the structural design of the piles.

Pile installation should be in accordance with Design Build Special Provision (DBSP) No. 0903 *(Deep Foundations)*. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile. The set criteria must therefore be established at the time of construction once the piling equipment is confirmed. The pile capacity should be verified in the field during the final stages of driving by the use of both the Hiley formula (MTO Standard Drawing SS103-11) on all piles and pile dynamic analyzer (PDA) testing (i.e. high strain dynamic testing) on a minimum of two piles at each foundation element where the piles are founded within the native soils; such testing should be observed by a qualified/certified engineer.

The following note from MTO's Structural Manual should be shown on the Contract Drawing, based on the application of a resistance factor of 0.5 to the use of the Hiley formula (per MTO experience in Southern Ontario) and to the ultimate capacity as assessed by PDA testing:

- Piles to be driven in accordance with Standard SS103-11 plus PDA testing using an ultimate geotechnical resistance of x,xxx kN per pile, but should be driven to no higher than 1.5 m above the design pile tip elevations shown below at each foundation element prior to the start of Hiley and/or PDA testing.

The above note applies to the North Abutment, Pier 2, Pier 3 and potentially the South Abutment. At Pier 1, Pier 4 and potentially the South Abutment, the note should read as follows:

- Piles to be driven to bedrock.

Assessment of the ultimate geotechnical resistance by the Hiley formula and PDA testing should commence once the pile reaches a depth of not higher than 1.5 m above the design pile tip elevation shown above and at 0.5 m intervals of depth until the ultimate axial resistance is achieved.

Due to the proximity of the new piles to the existing bridge (which will remain in operation) and new bridge half as well as the fact that the existing piles likely do not extend into the dense till or bedrock, a specific pile driving procedure should be developed to limit vibration and prevent movement of the structures. In this regard a Non-Standard Special Provision (NSSP) will be developed in conjunction with the Contractor to include in the contract documents. The piles furthest from the existing/new structure should be advanced first, with vibration monitoring and monitoring of any movement on the existing piles. As the construction progresses towards the existing/new structure, the results of the monitoring can be reviewed to determine if the pile driving procedures need to be amended (such as a reduction in hammer energy, etc.). Details of the monitoring program are provided under separate cover.

6.3.3 Downdrag

At both abutments, the loading from the approach embankment grade raise will result in consolidation settlement of the underlying firm to stiff clayey silt to clay strata. Settlement of the cohesive deposit relative to the stiff piles will result in the development of downdrag loads (negative skin friction) on the piles if the piles are installed prior to completion of this settlement and if the piles are driven into the very dense stratum (i.e. “100-blow” soils) or bedrock. Although lightweight fill is being used to offset the majority of the grade raise, the resulting settlement will still result in downdrag loading at the abutments.

The structural design of the abutment piles should be based on the full downdrag load acting on the piles. The estimated unfactored downdrag load acting on a single HP310X110 pile is 850 kN at the north and south abutments. For an HP360X132 pile, the estimated unfactored downdrag load acting on a single pile is 1,000 kN at the north and south abutments. Downdrag loads do not apply to the piers, as no grade changes are proposed in the vicinity of the new piers.

The downdrag loads noted above are unfactored loads. The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section Table 6.2 in Section 6.9.2 of the *CHBDC* for ULS conditions.

During the first stage of the new bridge construction when the existing bridge is still in place, the short-term settlement of the existing timber abutment piles (taking into account the elastic shortening) is estimated to be less than 5 mm. Therefore, downdrag loads acting on the existing timber piles for the duration of the first stage of construction can be considered negligible. Settlement monitoring of the existing abutments is recommended during construction, and this is addressed in the monitoring plan.

6.3.4 Resistance to Lateral Loads

Resistance to lateral loading may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are most appropriate where the maximum pile deflections are less than 1 percent of the pile width or diameter, where the loading is static (no cycling) and where the pile material is linear as per the *Canadian Foundation Engineering Manual* (CFEM, 2006). Where these conditions are not met, and/or where required for the structural engineering model, the non-linear lateral behavior of the soil should be considered using P-y curves, which can be provided if required. It is understood that P-y curves are not required at this time.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in *CHBDC* (2014) *Commentary* (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h, (kPa/m) is based on the equations given below, as described by Terzaghi (1955) and the *Canadian Foundation Engineering Manual* (CFEM 1992).

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction (kPa/m), as given below;
Z is the depth (m) of pile below ground surface; and
B is the pile diameter/width (m)

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and
B is the pile diameter/width (m)

The following values of n_h (Terzaghi, 1955) and s_u may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties, the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection). In the table, the clayey silt to clay stratum is referred to as the silty clay deposit for simplicity.

Foundation Element	Soil Unit	Elevation Interval (m)	n_h (kPa/m)	s_u (kPa)
North Abutment	Stiff to firm silty clay fill	176.2 to 170.7	--	50
	Soft to firm silty clay	170.7 to 160.6	--	30
	Dense to very dense silt and sand till	160.6 to 156.7	6,000	--
	Very dense silt and sand/silty sand	Below 156.7	7,000	--
Pier 1	Very soft to firm silty clay	167.0 to 157.5	--	20
	Compact to very dense silt and sand till	157.5 to 154.5	6,000	--
	Compact to very dense silty sand	Below 154.5	6,000	--
Pier 2	Firm to stiff silty clay	173.8 to 156.0	--	40
	Dense to very dense silt and sand till	Below 156.0	7,000	--
Pier 3	Stiff to very stiff silty clay	173.8 to 171.5	--	100
	Firm silty clay	171.5 to 153.4	--	40
	Compact to very dense silt and sand till	Below 153.4	6,000	--
Pier 4	Stiff to very stiff silty clay	175.4 to 172.0	--	100
	Firm silty clay	172.0 to 151.5	--	40
	Very dense silt and sand till	Below 151.5	6,000	--
South Abutment	Firm silty clay fill	179.6 to 175.5	--	50
	Firm silty clay	175.5 to 162.0	--	40
	Stiff to very stiff silty clay/silty clay till	162.0 to 155.0	--	100
	Very dense silt and sand till	Below 155.0	7,000	--

Note: The lateral resistance of fill material or organic materials should be ignored in calculating the lateral resistance of piles.

Both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal reaction of the piles will be controlled by deflections and the horizontal resistance of the pile should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the abutment wall for units supporting the abutments (*CHBDC* (2014) *Commentary* Section 6.11.2.2).

For a single vertical HP310x110 pile advanced at the foundation elements noted below, to the design tip elevations provided in Section 6.3.1, the estimated factored lateral resistance at ULS and the factored lateral resistance at

SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPile Plus (Version 2016), produced by Ensoft Inc.

Foundation Element	Lateral Resistance (kN)	
	Factored ULS	Factored SLS (for 10 mm of deflection)
North Abutment	200 kN	250 kN
Pier 1	60 kN	200 kN
Pier 2	170 kN	250 kN
Pier 3	280 kN	300 kN
Pier 4	260 kN	300 kN
South Abutment	180 kN	250 kN

The lateral resistances given above are based on an assumed fixed-head condition and 2,000 kN unfactored axial load applied at the top of the pile. The lateral resistance should be reviewed if greater vertical loads are anticipated.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters between rows of driven steel H-piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM7.2, 1986) as follows:

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

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre-to-centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

6.3.5 Interference Between New and Existing Piles

The new H-piles will be located in close proximity to the existing timber piles at some foundation elements, and in some cases, even overlapping. As the existing foundations will be removed to 300 mm below grade after the existing bridge is removed, they will not be required to carry any of the new bridge loading. However, they may physically conflict with the proposed new pile locations and may also impact the lateral resistance of the new piles (in particular if the existing piles are removed if/where conflicts occur).

Provision should be made in the design and construction for field adjustment of new pile locations to avoid conflicts between the new and existing piles. All deep foundations will also require a minimum separation distance to be maintained between the existing and new foundation elements for the proposed bridge replacement, in order to minimize the influence of the pile installation on the performance of the existing pile foundations. In general, the minimum separation distance will increase with larger pile sizes; however, the type of pile (driven versus drilled) and the termination stratum will also affect the minimum separation distance required. A preliminary assessment of each of the factors that could influence the effect of the new construction on the existing is provided below for driven piles:

- **Effect of soil displacements due to pile driving**: Fellenius et al. (1982), Bozozuk et al. (1978), and Poulos (1994) suggest a minimum separation distance of 9 m to 12 m, and no less than 10 pile diameters is required to minimize soil movements (laterally away from and vertically upwards) due to displacement of the soil during installation of driven piles which could cause heave, tensile forces or bending in existing adjacent piles. These sources note that the effect increases with the number of piles in a group.
- **Effect of vibrations due to pile driving:** Based on the work of Lacy and Gould (1985), Massarsch (2000) and Drabkin et al. (1996), and consideration of the subsurface conditions at the site, in order to limit vibrations (or Peak Particle Velocity) to a level below which the risk of vibration-induced settlements might occur, it is estimated that a minimum separation distance of about 15 m may be required. It is recommended that measurements of vibrations/peak particle velocities be carried out at the site (as part of test pile installation program as well as during production piling) and that the existing bridge structure and foundations be monitored for vibrations and settlements during construction.
- **Conflicts between existing and new battered pile elements:** Based on the limited available as-built drawings for the existing bridge, the number of battered piles is unknown although most piles appear to be vertical. The structural engineer should confirm that the existing piles and proposed piles do not present conflict or allow for field adjustment and/or additional piles.

As the potential effects of the old timber piles being left in place and in close contact with new H-piles is not known, consideration should be given to exposing the piles to determine the batter angle prior to pile driving of any new foundation elements, where conflicts or potential conflicts exist.

Because the new and existing foundation elements are only separated by about 5 m, a detailed piling procedure (as discussed in Section 6.3.2) and a monitoring program (to be provided under separate cover) will be required.

6.3.6 Frost Protection

All pile caps should be provided with a minimum of 1.2 m of conventional soil cover for frost protection, in accordance with OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario)* or equivalent thickness of insulation below the cap. As a guide, the MTO has adopted a 25 mm thickness of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in conventional soil cover.

6.4 Seismic Design

6.4.1 Seismic Zone

The site falls within the Southern Great Lakes Seismic Zone (SGLSZ) according to the Geological Survey of Canada. The SGLSZ covers a large area that extends from Windsor to east of Kingston and into the United States to the south. The region has low to moderate levels of seismicity. According to the Geological Survey of Canada, historical seismicity within the SGLSZ includes three moderately sized (about magnitude 5) events: 1929 in Attica, New York, 1986 near Cleveland, Ohio, and 1998 at the Pennsylvania/Ohio border. Only about two to three magnitude 2.5 or larger earthquakes have been recorded in the region over the past 30 years.

The *CHBDC* states that the seismic hazard values associated with the design earthquakes should be those established for the *National Building Code of Canada (NBCC)* by the Geological Survey of Canada (GSC). The GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

6.4.2 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the shear wave velocity measurements collected during the seismic cone penetration tests (i.e., SCPT 18-01 and SCPT 18-02). Average shear wave velocity values, V_{s30}, between 0 m and 30 m of depth are used in determining the site class. As it was not penetrated by the SCPTs, the dolostone bedrock was assumed to have an average shear wave velocity of 760 m/s.

The reference elevations for calculation of the V_{s30} values at the site were taken as the underside of the pile caps (i.e. Elevation 173.0 m and 177.0 m at the south and north abutments, respectively). The average shear wave velocity in the 30 m below the pile caps was calculated to range from about 275 to 280 m/s. Therefore, the site may be classified as Site Class D ($180 < V_s < 360$) in accordance with Table 4.1 of the *CHBDC*.

6.4.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the *CHBDC* and based on the location of the bridge (43.046N, -79.122W), the following are the Site Class C (reference) peak seismic hazard values based on the 5th generation seismic hazard maps published by the GSC.

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA	0.205 g
PGV	0.12 m/s
Sa (0.2s)	0.317 g
Sa (0.5s)	0.155 g
Sa (1.0s)	0.071 g
Sa (2.0s)	0.032 g
Sa (5.0s)	0.008 g
Sa (≥10.0s)	0.003 g

The values given above are for the reference ground condition Site Class C and must be modified for other seismic site classifications, in accordance with Section 4.4.3.3 of the CHBDC. The Site Class D peak ground acceleration (PGA) and design spectral acceleration values for the site are presented below. As indicated in Section 4.4.3.3 of the CHBDC, the value of PGA_{ref} for use with Tables 4.2 to 4.9 was taken as 80 percent of the PGA for Site Class C where Sa(0.2)/PGA is less than 2.0. Based on this requirement, a PGA_{ref} value of 0.164 g for the 2,475 year return period was used.

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA	0.240g
PGV	0.163 m/s
Sa (0.2s)	0.363g
Sa (0.5s)	0.211g
Sa (1.0s)	0.103g
Sa (2.0s)	0.048g
Sa (5.0s)	0.012g
Sa (≥10.0s)	0.004g

6.4.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify certain types of soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading”, or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with the “simplified” approach outlined in the *CHBDC* and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using the data collected at the boreholes. The design groundwater level was estimated based on the groundwater levels recorded in the piezometers and reported by others. The CRR with depth was calculated at selected borehole locations with looser granular soil deposits (Boreholes 18-09 and

18-10) using the parameter, $(N_1)_{60cs}$, that is based on the SPT "N"-values obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment indicate that the granular site soils have a low potential for liquefaction, and may be considered to be non-liquefiable for design.

The susceptibility of the silty clay to clay deposit to cyclic mobility was assessed based on the methodology provided in Idriss and Boulanger (2008), in which the CRR for clay-like soil is calculated based on the undrained shear strength and approximate OCR of the soil. The CRR is equated with the CSR (for reference stress equal to 65% of peak shear stress) to calculate the factor of safety against cyclic softening that would be expected to result in greater than 3% shear strain. Based on the results of the analyses, the silty clay is not considered to be susceptible to cyclic softening.

6.5 Road Weather Information System (RWIS) Foundation

A Road Weather Information System (RWIS) exists at the Welland River bridge site. The existing tower is proposed to be replaced at approximately the same location on the west side of the north abutment. The new RWIS tower is proposed to be supported on a spread footing founded within the stiff to very stiff silty clay fill as per nearby Boreholes 18-20 and 18-22.

The spread footing (assumed to be 1.5 m x 1.5 m) may be founded within the existing embankment fill at or below Elevation 179.7 m. The footing should be designed based on a factored ultimate geotechnical resistance of 450 kPa and a factored serviceability geotechnical resistance of 300 kPa (for 25 mm of settlement). The footing should be provided with a minimum of 1.2 m of conventional soil cover for frost protection. Since these values will be dependent on the footing size, configuration and applied load, the geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. The designer should also advise if higher geotechnical resistances than given above are required.

In addition, the geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *CHBDC* (2014).

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with Design Build Special Provision (DBSP) No. 0902 (*Excavation for Structure)* to check that the existing fill is suitable subgrade material. Any fill containing organics and any loose/soft deleterious fill must be subexcavated below the founding level and replaced with concrete.

Resistance to lateral forces/sliding resistance between the concrete footing and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). For cast-in-place concrete footings constructed on the silty clay fill, the coefficient of friction, tan φ', can be taken as 0.5. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the factored horizontal resistance.

6.6 Lateral Earth Pressures for Design of Abutments and Wingwalls

The lateral earth pressures acting on the abutment walls and any associated wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Where required, seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment walls and wingwalls:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* (2014) Section 6.12.3 and Figure 6.6. Hand operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.2 m behind the back of the wall, per Figure C6.20(a) of the *Commentary to the CHBDC* (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap, per Figure C6.20(b) of the *Commentary to the CHBDC* (2014).

6.6.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For a restrained wall, the pressures are based on the existing and proposed embankment fill and the following parameters (unfactored) may be used assuming existing embankment fill (earth fill) behind the granular zone:

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Earth Fill	20 kN/m^3	0.50	0.33

- For an unrestrained wall, the pressures are based on the properties of the granular backfill and the following parameters (unfactored) may be used:

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular ‘A’	22 kN/m^3	0.43	0.27
Granular ‘B’ Type II	21 kN/m^3	0.43	0.27
Cellular Concrete	5 kN/m^3	0.43	0.27

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:
  - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
  - Horizontal translation of 0.001 times the height of the wall; or
  - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6.2 Seismic Lateral Earth Pressures

Seismic (earthquake) loading must also be taken into account in the design of abutment walls/wing walls in accordance with Section 4.6.5 of the *CHBDC* (2014). In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the *CHBDC* and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA. For both cases, the value of the vertical seismic coefficient (k_v) is taken as zero.
- The following seismic active pressure coefficients (K_{AE}) are provided for design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Seismic Active Pressure Coefficients, K_{AE}

	Design Earthquake	Site PGA	SSM	Granular A	Granular B Type II
Yielding wall	2,475 Yr	0.240g	0.38	0.34	0.34
Non-yielding wall	2,475 Yr	0.240g	0.47	0.43	0.43

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250 \cdot k_h$ (in mm), where k_h is the site-specific PGA as given in the above table. This corresponds to displacements of up to approximately 30 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined per Section C4.6.5 of the Commentary to CHBDC (2014) and as follows:

$$\sigma_h(d) = K_a\, \gamma\, d + (K_{AE} - K_a)\, \gamma\, (H\text{-}d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o\, \gamma\, d + (K_{AE} - K_a)\, \gamma\, (H\text{-}d), \text{ non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K_a is the static active earth pressure coefficient;
K_o is the static at-rest earth pressure coefficient;
K_{AE} is the seismic active earth pressure coefficient;
γ is the unit weight of the backfill soil (kN/m^3), as given previously;
d is the depth below the top of the wall (m); and,
H is the total height of the wall (m).

6.7 Embankment Design and Construction

Based on the GA drawing, the proposed grade at the north and south abutments for the replacement structure will be at approximately Elevations 182.0 m and 186.0 m, respectively. These proposed grades at the north and south abutments will be about 2.5 m and 2.1 m above the existing grade, respectively, tapering out some 400 m beyond the abutments. It is understood that the vertical profile has not yet been finalized and as such, revisions to the settlement estimates and recommendations presented in this section of the report will be required. In general, the south side of the site (i.e. south of Welland River) has the thickest cohesive deposit at about 20 m and on the north side of the site the cohesive deposit is about 10 m thick.

The following sections address the stability and settlement of the raised approach embankments on the existing QEW, including over the Grassy Brook Culvert. The stability and settlement analyses assume that the majority of the existing fill will be left in place where the grade raise is being proposed, except where required to be removed for placement of lightweight fill or for any minor widening at the site.

The existing QEW embankment side slopes at the bridge are at about 2H:1V. The southwest side slope has an existing 10 m wide berm at approximately Elevation 179 m that was constructed in 1994 making the overall slope orientation at about 4H:1V. The new fill as a result of the up to 2.5 m grade raise is expected to result in minor widening of the embankments and the new toe of slope is anticipated to tie into approximately the existing toe of slope or even further up along the slope. The height of new fill on the embankment side slope as part of the widening is typically less than the grade raise at any location.

The existing front slopes of the bridge are at approximately 2H:1V and as the new abutments will be behind the existing abutments, the final front slopes should be maintained at 2H:1V.

The global stability analyses were carried out assuming a 2H:1V side slope profile for the new widened embankment fill over the existing fill tying into the existing slope. Because the amount of fill widening is minimal, it is recommended that the new fill consist of OPSS.PROV 1010 Granular B Type II or Select Subgrade Material, properly placed and compacted and appropriately keyed in as discussed further in Section 6.8.1. Earth fill may also be considered for embankment widening; however, an evaluation of the potential source material, including its plasticity and water content, should be made to confirm the suitability in terms of settlement and surficial stability.

The critical sections used in the analyses are located just behind the abutments (at approximately Stations 21+540 and 10+ 220) where the grade raise is highest at each side of the bridge. One other critical section is located at the Grassy Brook Culvert (at approximately Station 10+350) where an approximately 1.6 m grade raise is proposed and zero additional load is specified per the RFP. The piezometric conditions used in the analyses are based on the groundwater level as encountered during the subsurface investigation. A groundwater level of Elevation 172.5 m and 172.9 m was used at the north and south abutments, respectively, and Elevation 179.0 m was used at the Grassy Brook Culvert.

6.7.1 Global Stability

6.7.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 7.031), produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the factors of safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety (FoS) is defined as the ratio of forces tending to resist failure to the driving forces tending to cause the failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, ψ, and the geotechnical resistance factor ϕ_{gu} (i.e. FoS = $1/(\psi * \phi_{gu})$). Accordingly, a target minimum FoS of 1.3 has been used for design of the temporary embankment side slopes, and a FoS of 1.5 for the design of the final embankment configuration as per Table 6.2 of CHBDC (2014) for the total stress (short-term undrained) and effective stress (long-term drained) condition, as applicable.

6.7.1.2 Parameter Selection

The simplified stratigraphy together with the associated strength and unit weights employed for the different soil types at the critical sections are summarized for all soil layers in the table below and are plotted for the cohesive deposit on Figure 1.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Effective Cohesion (c') (kPa)	Undrained Shear Strength (s_u) (kPa)
New granular fill	21	35	--	--
Cellular concrete	5	35	--	--
Silty clay fill	19	28	3	--
Very stiff clayey silt to clay (crust or lower portion)	19	27	3	See Figure 1
Soft to stiff clayey silt to clay	19	23	3	See Figure 1
Hard clayey silt till	21	32	0	--
Compact to very dense silt and sand till	21	35	--	--

The interpreted undrained shear strength (s_u) in the upper stiff silty clay to clay deposit (i.e. the crust) is shown on the design line on Figure 1 based on the results of the field shear vane tests and CPT tests. At this site, at the north abutment, there is little crust. On the south side and at the Grassy Brook culvert, up to approximately 10 m of the 20 m thick silty clay deposit is very stiff, with the very stiff portion varying between the upper crust of the deposit and the lower portion.

The subsoils encountered are composed of a combination of cohesive deposits (clayey silt to clay) and granular deposits (silt and sand). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle) for the granular soils were estimated from empirical correlations using the results of the SPT "N"-values as suggested by NAVFAC (1986) and Kulhawy and Mayne (1990), in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the short-term, undrained analyses while effective stress parameters were employed in the long-term, drained analyses.

The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of the in-situ field vane shear tests and CPT tests and were also inferred from estimates of preconsolidation stress (σ_p', kPa) from the results of the consolidation tests, by employing the following correlation proposed by Mesri (1975):

$$s_u = 0.22\sigma_p'$$

where:

s_u = average mobilized undrained shear strength (kPa)

σ_p' = preconsolidation stress (kPa)

Where appropriate, Bjerrum's correction factor was employed to estimate the average mobilized undrained shear strength from the results of the in-situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \text{(after Bjerrum, 1973)}$$

where:

- $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
- $s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
- μ = Bjerrum's correction factor based on Plasticity Index

The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils assessed from the results of the in-situ CPT tests, used the following equation (Lunne et. al., 1997):

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}}$$

where :

- s_u = average mobilized undrained shear strength (kPa)
- q_t = corrected cone tip resistance (kPa) – see below
- σ_{vo} = total in-situ vertical stress (kPa)
- N_{kt} = empirical cone factor equal to 19 for this site (selected to fit cone data to in-situ field vane shear tests)

The effective stress parameters (i.e. c' and ϕ') for the cohesive soils were estimated from empirical correlations using the results of the laboratory index testing as suggested by Ladd (1977) and Mitchell (1993) and were also inferred from the results of triaxial testing in conjunction with engineering judgement based on experience in similar soil conditions.

6.7.2 Settlement

Settlement of the approach embankments can be expected as a result of the loading from the new fills on the compressible foundation soils at this site. In addition, depending on the type of fill materials employed in the construction, settlements may also occur due to compression of the new embankment fill itself. The following sections outline the methods used to assess the design parameters and carry out settlement analyses.

6.7.2.1 Methodology

To estimate the magnitude of the expected settlements, analyses were carried out at the critical sections of the proposed embankment widening area using the commercially available program Settle^{3D} (Version 4.015) produced by Rocscience Inc., combined with hand/spreadsheet calculations, where appropriate.

The sources of settlement were considered to include the following:

- primary time-dependent consolidation of the cohesive deposits;
- secondary time-dependent (creep) compression of the cohesive deposits (long-term); and
- immediate settlement of the granular foundation soils and existing embankment fill.

6.7.2.2 Parameter Selection

The immediate compression of the native cohesionless soil layers was modelled by estimating an elastic modulus of deformation based on the SPT "N"-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in *CHBDC* (2006) and adjusted, as necessary.

The consolidation settlement of the cohesive deposit was assessed by evaluating compressibility parameters (i.e. σ_p', C_r, C_c, $C_{\alpha\varepsilon}$) from the results of the laboratory consolidation tests, in-situ field vane tests and the CPT tests along with the results of the laboratory index tests and using empirical correlations proposed in literature by Terzaghi and Peck (1967), Nishida (1956) and Azzouz et al. (1976).

The following correlation relating in-situ undrained shear strength to preconsolidation stress (Mesri, 1975) was employed:

$$\sigma_p' = \frac{s_{u(mob)}}{0.22}$$

where :

$s_{u(mob)}$ = $\mu s_{u(FV)}$
σ_p' = preconsolidation stress (kPa)
$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
μ = Bjerrum's correction factor based on Plasticity Index

The preconsolidation stress was also estimated from the results of the CPTs (Demer and Leroueil, 2002):

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4}$$

where

q_t = $q_c - u_2(1 - A_n)$ (kPa)
q_c = tip stress measured by the CPT (kPa)
u_2 = pore pressure measured at cone 'shoulder' (kPa)
A_n = cone constant
σ_{vo} = total vertical stress (kPa)

Based on previous experience with the clayey soils in the area, an approximate ratio between the compression index and the recompression index of 10 (i.e. $C_r = 0.10\ C_c$) was utilized to estimate C_r from the above noted empirical correlations.

In addition to primary consolidation within clays, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after substantial dissipation of excess pore pressure under a constant stress. The following relationships have been employed for estimating the magnitude of creep settlement for each log cycle of time following completion of primary settlement at each location.

$$S_c = HC_{\alpha\varepsilon} \log\left(\frac{t}{t_{EoP}}\right)$$

where :

S_c/log cycle =secondary consolidation (creep) settlement (mm)
$C_{\alpha\varepsilon}$ =modified secondary compression index
H =initial thickness of compressible clay deposit (mm)
w_n =average natural water content (%)
t =time period of interest (or design life of structure) (years)
t_{EoP} =time to reach 90% primary consolidation (years)

Values of $C_{\alpha\varepsilon}$ were estimated from the Dial Reading versus Log-Time plots from the consolidation tests for the appropriate range of stress level applicable for the proposed embankment widening. In addition, the following empirical correlation by Mesri (1973) was also utilized to estimate $C_{\alpha\varepsilon}$ from water content:

$$C_{\alpha\epsilon} = w_n/10{,}000$$

where: w_n = natural water content (%)

The coefficient of consolidation, c_v, required in the time-rate settlement analysis for the normally consolidated soils at this site is estimated to be 6.7 x 10^{-3} cm^2/s (based on the consolidation tests and CPT dissipation tests and correlations with natural water content). For secondary (creep) compression, the modified secondary compression index, $C_{\alpha\varepsilon}$ is estimated to be 0.00209.

The simplified stratigraphy together with the associated compressibility parameters and unit weights employed for the different soil types at the approach embankments are presented below and on Figure 1. Essentially, the upper crust, where present, and the lower portion of the deposit is over-consolidated and the middle firm portion of the deposit is normally or slightly overconsolidated with an OCR of approximately 1, meaning that any new load will result in consolidation settlement.

Soil Type	γ (kN/m^3)	σ_p' (kPa)	OCR	e_o	C_c	C_r
Upper stiff to very stiff clayey silt to clay crust	19	420 – σ_{vo}'	1 to 10	0.65	0.2	0.02
Firm clayey silt to clay	19	σ_{vo}'	1	0.75	0.2	0.02
Lower stiff to very stiff clayey silt to clay	19	σ_{vo}' - 475	1 to 10	0.75	0.2	0.02

6.7.3 Settlement Performance Requirements

The settlement performance criterion for design of high fill embankments is in accordance with MTO’s Guideline “Embankment Settlement Criteria for Design” (2010).

Where new embankments approach structural elements, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Maximum Limits During Pavement Design Life	
	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75
	>75 m	<100

The above criteria, and limiting differential settlement to 200:1, have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankment grade raise. The settlement criteria are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These settlement criteria form part of the overall design performance for the QEW embankments approaching the new replacement bridge. As such, the critical sections for the settlement analysis are at the abutments and at 20 m, 50 m and 75 m beyond the abutments, as well as over the Grassy Brook Culvert.

6.7.4 Results of Analysis

The results of the stability and settlement analyses for the embankment grade raise are provided in the following sections. In addition, recommendations for achieving the target factor of safety for slope stability for the required embankment geometry and for minimizing/mitigating the time dependent, post-construction settlements are also discussed.

The critical sections for stability and settlement are described above. The presence of the weak/firm cohesive deposits constitute zones of potential instability of the proposed embankment final geometry. In addition, given that the foundation soils at the site are comprised primarily of relatively thick compressible cohesive subsoils, time-dependent settlements under the raised embankments are expected. As such, mitigation measures will be required to maintain stability of the raised/widened embankments and to limit the post-construction settlements and reduce subsequent maintenance requirements on the highway.

6.7.4.1 Stability

Limit equilibrium analysis indicates a Factor of Safety of greater than 1.3 in the short-term (total stress analysis) and greater than 1.5 in the long-term (effective stress analysis) at the critical sections for the proposed raised embankment geometry, as shown on Figures 2 and 3 for the immediate north and south approaches, respectively, and at the Grassy Brook Culvert. The stability analyses were carried out assuming that the embankments are constructed with granular fill with 2H:1V embankment side slopes and 4H:1V slopes through the pavement structure as discussed above.

6.7.4.2 Settlement

A summary of the results of the settlement analysis at the critical sections, assuming embankment construction with granular fill, is presented below.

Approx. Station	Location	Proposed Grade Raise (m)	Settlement Criteria (mm)	Immediate Settlement	Estimated Factored Post-Construction Settlement (mm)		
					Primary	Creep in 20 Years Post Paving	Total Post-Construction (at 20 year design life)
21+475	75 m north of N. Abut.	1.8	100	<25	90	18	108
21+500	50 m north of N. Abut.	2.0	75	<25	90	16	106
21+530	20 m north of N. Abut.	2.4	50	<25	135	17	152

Approx. Station	Location	Proposed Grade Raise (m)	Settlement Criteria (mm)	Immediate Settlement	Estimated Factored Post-Construction Settlement (mm)		
					Primary	Creep in 20 Years Post Paving	Total Post-Construction (at 20 year design life)
21+550	North abutment	2.5	25	<25	105	22	127
10+200	South abutment	2.1	25	<25	85	22	107
10+220	20 m south of S. Abut.	2.0	50	<25	90	22	112
10+250	50 m south of S. Abut.	2.0	75	<25	70	22	92
10+275	75 m south of S. Abut.	2.0	100	<25	90	22	112
10+350	Grassy Brook Culvert	1.6	0	<25	75	22	97

At all critical locations, the magnitude of post-construction settlement is greater than MTO's settlement performance criteria and thus mitigation measures will be required. Because the embankment will be constructed in stages, the minimum differential settlement criteria of 1:200 in the transverse direction will also not be achieved resulting in the need for mitigation.

6.7.5 Settlement Mitigation

As per the RFP, lightweight fill has been specified to offset the grade raise to reduce post-construction settlement to meet the settlement criteria. Further to pre-design discussions, it is understood that the design-build team's preferred settlement mitigation option is the use of cellular concrete to construct the raised embankments. Using cellular concrete would reduce the loading on the subsoils, thereby improving the stability and reducing the post-construction settlement. Consideration has also been given to the use of lightweight rigid expanded polystyrene (EPS) blocks. However, as temporary protection systems will be required behind the abutments to facilitate staged construction, it may be difficult to install soil anchors through the fill. Further, it may be difficult to connect the EPS of the two stages in a manner that keeps the blocks acting as a solid mass. The benefit to using EPS over cellular concrete is that it is more lightweight and therefore results in lower settlement for the same grade raise/volume of material. Other types of settlement mitigation (such as preloading/surcharging or ground improvement) have not been considered at this point due to the complex staging requirements and overall construction schedule, although these options can be examined further as the detailed design is confirmed.

The required thickness of cellular concrete to meet the settlement criteria together with the required subexcavation depth to accommodate the thicker cellular concrete installation, as well as the proposed thickness of EPS for comparison, are presented in the table below.

Approximate Station	Location	Proposed Grade Raise (m)	Mitigation Option: Thicker Cellular Concrete Plus Subexcavation		Mitigation Option: Use of EPS
			Thickness of Cellular Concrete (m)	Approximate Subexcavaton Depth (m)*	Thickness of EPS (m)
21+475	75 m north of N. Abut.	1.8	0.3	-0.5	0.3
21+500	50 m north of N. Abut.	2.0	1.0	0	0.6
21+530	20 m north of N. Abut.	2.4	2.5	1.1	1.5
21+550	North abutment	2.5	3.3	1.8	2.1
10+200	South abutment	2.1	2.7	1.6	2.7
10+220	20 m south of S. Abut.	2.0	1.8	0.8	1.8
10+250	50 m south of S. Abut.	2.0	0.5	-0.5	0.5
10+275	75 m south of S. Abut.	2.0	0.4	-0.6	0.4
10+350	Grassy Brook Culvert	1.6	2.1	1.5	1.6

*Assuming an average pavement structure thickness of 1 m. Positive numbers indicate subexcavation is required.

Installation of cellular concrete should be in accordance with Special Provision for Cellular Concrete as provided in the RFP and attached in Appendix E.

The cellular concrete should be placed behind the abutments at the thicknesses noted above then tapering to 0 m thickness over a transition of 5H:1V beyond the limits noted above. At the Grassy Brook culvert, the cellular concrete thickness should be placed in a zone 10 m on either side of the culvert then tapering to a thickness of 0 m at 5H:1V south of the culvert and at 5H:1V north of the culvert to match the thickness of cellular concrete required at Station 10+275. The cellular concrete shall have a unit weight of no greater than 5 kN/m^3. The lateral extent of the cellular concrete should be between the outside crests of the full embankment width (which includes the shoulders) and stepped down at 1H:1V in steps not more than 650 mm thickness. Cover over the cellular concrete is limited to the pavement structure, which is understood to be an average of about 1 m, and slightly variable along the alignment and transversely. Immediately behind the abutment, the cellular concrete should be separated from the abutment concrete using a 75 mm thick layer of “Eva foam” (or equivalent) to act as a bond break.

6.8 Construction Considerations

6.8.1 Subgrade Preparation and Embankment Construction

Prior to construction of the raised embankments, it is recommended that all topsoil/organic soils and any loose/soft deleterious fill be stripped from within the footprint of the embankments and from the side slopes of the existing embankments.

It is recommended that fill for construction of the raised QEW embankments consist of granular fill or SSM plus cellular concrete; earth fill may be considered, although its suitability would be dependent on the quality of the source, including its plasticity and water content. Where granular fill is used, it should consist of OPSS.PROV 1010 (*Aggregates*) Granular ‘B’ Type I or II or Granular ‘A’; SSM should meet the requirements set out in OPSS.PROV 1010. Fill materials should be placed and compacted in accordance with OPSS.PROV 501 *(Compacting)* and OPSS.PROV 206 *(Grading)* and should be benched into the existing embankment side slopes in accordance with OPSD 208.010 (Benching of Earth Slopes). Embankments greater than 8 m should incorporate into the side slopes a minimum 2 m wide bench at mid-height for all fill heights greater than 8 m as suggested in OPSD 202.010 (Slope Flattening).

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 *(Rip Rap, Rock Protection and Granular Sheeting)*. Erosion protection should be placed on the slopes to at least 0.5 m above the design high water level. Subject to confirmation and modifications as necessary based on the hydrology assessment (by others), erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (180 mm size as per OPSS.PROV 1004 *[Aggregates Miscellaneous]*) or Rock Protection. The designer should address the potential for hydraulic scour below the pile caps in the design of the bridge foundations.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 *(Topsoil)* and OPSS.PROV 804 *(Seed and Cover)* should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 *(Rip Rap, Rock Protection and Granular Sheeting)*, and OPSS.PROV 1004 *(Aggregates – Miscellaneous)* will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.8.2 Excavations and Groundwater Control

The excavations for removal of the existing bridge structure and construction of the new pile caps will extend through existing embankment fill and/or native clayey silt to clay deposits. At the location of Pier 1, the pile cap will extend through native peat/organic silt material. These organic materials are recommended to be left in place, as the Pier will be supported on deep foundations, and any subexcavation and replacement of peat/organic soils will increase the effective stress and result in settlement and downdrag loading. However, some nominal subexcavation and replacement with a layer of granular fill is recommended to create a stable/clean base inside the excavation for pile driving; consideration will also need to be given to the working pad for the pile driving rig and other equipment in proximity to this pier location.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities (Ontario Regulation 213/91). The existing fill materials and native clayey silt to clay soils above the groundwater level are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V. Below the groundwater level, the native peat/organic silt and firm to stiff clayey silt to clay is classified as Type 4 soils requiring minimum 3H:1V side slopes or flatter.

Based on the elevations of the pile cap undersides provided on the GA drawing, it is anticipated that excavations for removal of the existing bridge structure and construction of the new pile caps at Piers 1 and 2 will extend below the local groundwater table, estimated to be at about Elevation 171.4 m. Depending on the time of year of construction, higher groundwater levels may exist and Piers 3 and 4 may also extend below the groundwater level. Further, perched water may be present within the embankment fill. As such, some form of groundwater control and dewatering, most likely in the form of a cofferdam, will be required to cut-off the potential for water inflows and maintain a dry excavation. Dewatering should be in accordance with Design Build Special Provision (DBSP) No. 0902 (*Excavation for Structure).* The design and construction of the groundwater control systems is the responsibility of the Contractor.

6.8.3 Construction Staging and Temporary Protection Systems

The existing twin bridges are proposed to be replaced using half-and-half staged construction. The Contractor has selected to remove the northbound (EBL) bridge first to allow for construction of the east half of the new replacement bridge while the traffic is routed to the existing southbound (WBL) bridge. Once the new east half is constructed and the traffic re-routed, the southbound bridge will be removed, and the west half of the new replacement bridge constructed. Temporary protection systems will be required along the QEW median to allow for excavation and removal of the existing abutments and pile caps and for construction of new abutment walls supported on new foundations. Temporary cofferdams will also be required to construct the piers. In order to limit excavation in the environmentally sensitive wetlands, the existing foundations (i.e. piles and pile caps) are to be cut off at the depth specified in the RFP

The temporary protection systems are to be designed, constructed and monitored in accordance with Design Build Special Provision (DBSP) No. 0539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 1a where the protection system face is located within a horizontal distance of ⅓ H of any part of a structure foundation or Performance Level 2 in other areas, as specified in DBSP No. 0539. Due to the nature of the firm to stiff compressible soils present at this site, consideration should be given to leaving the temporary protection systems in place and cut-off to avoid further loosening or softening of the native soils and existing embankment fill.

It is considered that driven sheet-piles would be suitable for the temporary excavation support at this site, based on the subsurface soil and groundwater conditions, and this type of system would also provide the necessary groundwater control/cut-off as described in Section 6.8.2. For construction of the abutments, a sheet-pile or a soldier pile and timber lagging system could be considered; however, some groundwater seepage should be anticipated through the fill if perched water conditions exist, as well as at the base of the non-cohesive fill, and it may be necessary to control seepage or include measures to mitigate loss of soil particles through the lagging boards. In this regard use of a sheet-pile wall would be advantageous.

The sheet-pile wall would have to penetrate to sufficient depth to provide the necessary passive resistance for the retained soil height. Additional lateral support to the sheet-pile wall (or soldier pile wall), if required, could be provided in the form of rakers or temporary anchors. The selection and design of the protection system will be the responsibility of the Contractor. The installation of anchors will likely not be impeded by the presence of cellular concrete within the embankment, however, the Contractors equipment should be sufficient to allow for advancement through the cellular concrete, if required.

The design and construction of the temporary support systems is the responsibility of the Contractor. The support systems may be designed using the following parameters. It should be noted that in the table below, the clayey silt to clay stratum is referred to as the 'silty clay' deposit for simplicity.

Foundation Element	Soil Type	Elevation Interval (m)	Unit Weight	Undrained Shear Strength	Internal Angle of Friction	Coefficient of Earth Pressure[1]		
			γ (kN/m³)	s_u (kPa)	ϕ' (°)	Active K_a	At Rest K_o	Passive[2] K_p
North Abutment (BHs 18-18, 18-19, 18-20)	Stiff to very stiff silty clay fill	180.0-171.5	19	75	30	0.333	0.500	3.000
	Soft to firm silty clay	171.5-160.6	19	30	29	0.347	0.515	2.882
	Dense silt and sand till	160.6-156.7	21	--	34	0.283	0.441	3.537
	Very dense silt and sand/silty sand	Below 157.0	20	--	35	0.271	0.426	3.690
Pier 1 (BHs 18-15, 18-16, 18-17)	Soft to firm silty clay fill	171.5-170.0	19	25	28	0.361	0.531	2.770
	Very soft organic silt	170.0-167.0	16	5	23	0.438	0.609	2.283
	Very soft to firm silty clay	167.0-157.5	19	20	29	0.347	0.515	2.882
	Compact to very dense silt and sand till	157.5-154.5	21	--	34	0.283	0.441	3.537
	Compact to very dense silty sand	Below 154.5	20	--	34	0.283	0.441	3.537
Pier 2 (BHs 18-12, 18-13, 18-14)	Firm silty clay	173.8-156.0	19	40	29	0.347	0.515	2.882
	Dense to very dense silt and sand till	Below 156.0	21	--	35	0.271	0.426	3.690
Pier 3 (BHs 18-09, 18-10, 18-11)	Stiff to very stiff silty clay	174.4-171.5	19	100	32	0.307	0.470	3.255
	Firm silty clay	171.5-153.4	19	40	29	0.347	0.515	2.882
	Loose to very dense silt and sand till	Below 153.4	21	--	33	0.295	0.455	3.392
Pier 4 (BHs 18-06, 18-07, 18-08)	Stiff to very stiff clayey silt fill	175.4-173.2	19	75	30	0.333	0.500	3.000
	Stiff to very stiff silty clay	173.2-172.0	19	100	32	0.307	0.470	3.255
	Firm silty clay	172.0-151.5	19	40	29	0.347	0.515	2.882
	Very dense silt and sand till	Below 151.5	21	--	35	0.271	0.426	3.690
South Abutment	Stiff to very stiff silty clay fill	184.3-175.5	19	75	30	0.333	0.500	3.000
	Firm silty clay	175.5-162.0	19	40	29	0.347	0.515	2.882

Found-ation Element	Soil Type	Elevation Interval (m)	Unit Weight	Undrained Shear Strength	Internal Angle of Friction	Coefficient of Earth Pressure[1]		
			γ (kN/m^3)	s_u (kPa)	ϕ' (°)	Active K_a	At Rest K_o	Passive[2] K_p
(BHs 18-03, 18-04, 18-05)	Very stiff silty clay/ silty clay till	162.0-155.0	19	100	32	0.307	0.470	3.255
	Hard silty clay till/ very dense silt and sand till	Below 155.0	21	200	33	0.295	0.455	3.392

1. The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected behind or in front of the walls, the coefficients should be corrected accordingly, as per the CFEM (2006) or other appropriate reference.
2. The total passive resistance below the base of the excavation (i.e. adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

The design groundwater level varies across the site and should be taken as the river high water level for the piers (as determined by others) and at Elevation 172.5 m and 172.9 m at the north and south abutment, respectively.

Design of the temporary support system should include an evaluation of base stability, soil squeezing stability and hydraulic uplift stability as defined in the CFEM (2006). The cohesive soils at this site are sensitive to disturbance from vibration and/or driving operations for sheet pile installation, which should be considered in the design and installation of the temporary protection systems.

6.8.4 Obstructions

As encountered in the boreholes advanced at the site, cobbles and/or boulders will be encountered within the native glacial till deposits, which may affect the installation of deep foundations and/or protection system elements. Boulders are commonly encountered in the overburden soils/tills of Southern Ontario. The presence of boulders can significantly affect the selection of equipment and progress of construction works. Boulders with maximum dimensions larger than 300 mm should be anticipated at this site. These boulders originate from the igneous and metamorphic rocks of the Canadian Shield and, therefore, boulders with uniaxial compressive strengths on the order of 250 MPa should be expected to be present at the site. It is recommended that bearing points (such as Titus standard “H” point or equivalent) be used on all steel H-piles to facilitate driving into the very dense, “100-blow” till. In addition, it is recommended that an NSSP or Notice to Contractor be included in the specifications to warn the Contractor of the possible presence of cobbles and/or boulders within the overburden soils.

6.8.5 Potential Mitigation of Water Seepage Along Piles

As presented in Section 4.2.10, pressurized groundwater conditions were encountered within the lower silt and sand till deposit at the site. At the location of Pier 1 and 2, the underside of pile cap is about 1.7 m to 2.3 m below the measured groundwater levels. As the piles will be driven through the silty clay and into the pressurized aquifer, mitigation measures (such as the use of a drainage/filter blanket comprised of OPSS.PROV 1002 concrete fine aggregate) may be required to minimize the risk of soil migration up along the pile shaft during/following pile driving. The contractor will be responsible for the selection and installation of an appropriate mitigation measure.

6.8.6 Monitoring

It is recommended that the existing bridge and new half bridge be monitored for settlement and lateral movement during the staged construction of the replacement structure especially during construction works adjacent to the existing structure, such as excavation operations, installation of temporary protection/cofferdams and installation of deep foundations, for the following reasons:

- The existing bridge is supported by timber piles founded at unknown depth and assumed to terminate with the firm to very stiff clayey silt to clay deposit and not the dense till deposit or bedrock; and,
- The existing structure is carrying traffic on the QEW and will have to be maintained in operation (or a portion thereof during the staging) throughout the construction.

In addition to monitoring of the bridge during construction, monitoring of post-construction settlement is required along the approach embankments in the areas where lightweight fill is used. Further, settlement monitoring of the CP Railway tracks is required during construction as per the requirements of the RFP. Consideration should also be given to monitoring on the nearby Montrose Road bridge and other areas in close proximity of the construction works to confirm no adverse affects on public/private property.

The monitoring program (i.e. types of instruments including temporary benchmarks, instrument installation details, frequency of monitoring, review/alert levels, the need for pre-/post-construction surveys, reporting requirements, etc.) for the site is provided under separate cover. Careful consideration will have to be given to the location of the instruments across the site due to avoid damage to instruments as a result of bridge demolition, construction access and staging.

6.8.7 Use of Heavy Equipment

In order to drive piles and construct the replacement bridge, the use of heavy construction equipment will be required. The impact of the heavy equipment loads on the underlying firm to stiff clayey silt to clay soils must be considered during selection of the scheduling, methodology and equipment employed for construction. Further, this equipment may also require the construction of temporary access roads to access the bridge foundation element locations. Based on the large central span of the bridge, very large cranes are anticipated to be required to lift the girders into place.

It is understood that these large cranes will be required adjacent to Piers 1 and 2. At the location of Pier 2, the subsurface conditions consist of a silty clay crust overlying the firm silty clay deposit, which will provide some support for the crane pad. At the location of Pier 1, the subsurface conditions consist of 1.5 m of silty clay fill overlying 3 m of peat/organic silt overlying firm silty clay. The presence of the peat/organic silt is problematic with respect to settlement and stability of the crane pad and equipment in this area. The placement of any amount of fill, even with a geogrid, will result in settlement of this layer, most likely not within a tolerable limit for a crane, and a deep foundation support (or potentially the application of ground improvement through the peat/organic soils) for the crane pad should be considered at this location. The design and construction of all heavy equipment/crane pads and access roads are the responsibility of the Contractor.

Heavy construction equipment, material stockpiles and the like should not be permitted within 2 m of the crest of any cut slopes or excavations.

6.8.8 Recommendations for Construction Materials Based on Analytical Testing

The results of analytical testing completed on four samples, one sample of the silty clay fill, two samples of the native silty clay and one sample of the native silt and sand, are summarized in Section 4.2.11 and presented in full in Appendix B. The potential for sulphate attack and corrosion are discussed in the following paragraphs. However, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1-24 Section 4.1.1 "*Durability Requirements*" are followed when designing concrete elements.

The potential for sulphate attack on concrete was determined by comparing analytical test results to CSA A23.1-14 Table 3 "*Additional Requirements for Concrete Subjected to Sulphate Attach*". The water-soluble sulphate concentration measured in the silty clay fill was 0.12 per cent, which is within the exposure class of S-3 (Moderate). Therefore, based on the test result for the sample of fill, when the designer is selecting the exposure class for the structure in contact with the fill the effects of the sulphates will need to be considered. The water-soluble sulphate concentration measured in the native silty clay and silt and sand ranged between 0.02 per cent and 0.03 per cent, which is below the exposure class S-3 (Moderate). Therefore, based on the test results for the samples of native silty clay and silt and sand, when the designer is selecting the exposure class for the structure in contact with the native silty clay or silt and sand the effects of the sulphates may not need to be considered. Additionally, given the location of the structure under the QEW, it may be exposed to de-icing salts and selection of the exposure class should consider this.

The silty clay fill has a pH of 7.7 and a resistivity of 741 ohm-cm. According to the MTO Gravity Pipe Guidelines, the pH is not considered detrimental to structure durability as it is less than a pH of 8.5 but greater than a pH of 5.5. The resistivity is less than 2,000 ohm-cm, which indicates that the soil corrosiveness is severe (R < 2,000 ohm-cm), as per Table 3.2 "*Soil Corrosiveness and Resistivity*" of the MTO Gravity Pipe Guidelines. The native silty clay and silt and sand have a pH ranging between 7.9 and 8.0 and a resistivity ranging between 2460 ohm-cm and 3020 ohm-cm. According to the MTO Gravity Pipe Guidelines, the pH is not considered detrimental to structure durability as it is less than a pH of 8.5 but greater than a pH of 5.5. The resistivity is less than 4,500 ohm-cm but greater than 2,000 ohm-cm, which indicates that the soil corrosiveness is moderate (4,500 ohm-cm > R < 2,000 ohm-cm), as per Table 3.2 "*Soil Corrosiveness and Resistivity*" of the MTO Gravity Pipe Guidelines.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Mo’oud Nasr, P.Eng., a geotechnical engineer with Golder and the technical aspects were reviewed by Ms. Sarah Poot, P.Eng., Associate of Golder and the Senior Foundation Engineer/Lead for this project. Ms. Lisa Coyne, P.Eng., a Principal of Golder and MTO Foundations Designated Contact, conducted an independent quality control review of the report.

Golder Associates Ltd.

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MTO Foundations Designated Contact, Principal

MN/MWK/SEMP/LCC/ljv

https://golderassociates.sharepoint.com/sites/33369g/deliverables/2 design/1 foundations/reporting/2draft fidr/18109622-r-fidr welland db 2019'07'16.docx

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ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

ASTM D7012 Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Ontario Provisional Standard Drawings:

OPSD 208.010	Benching of Earth Slopes
OPSD 202.010	Slope Flattening
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements

Ontario Provincial Standard Specifications:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specifications for Compacting
OPSS 511	Rip Rap, Rock Protection and Granular Sheeting
OPSS 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 1002	Material Specification for Aggregates – Concrete
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Design Build Special Provisions:

DBSP No. 0539	Temporary Protection Systems
DBSP No. 0902	Excavation for Structure
DBSP No. 0903	Deep Foundations

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects (as amended)

Ministry of Transportation, Ontario

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

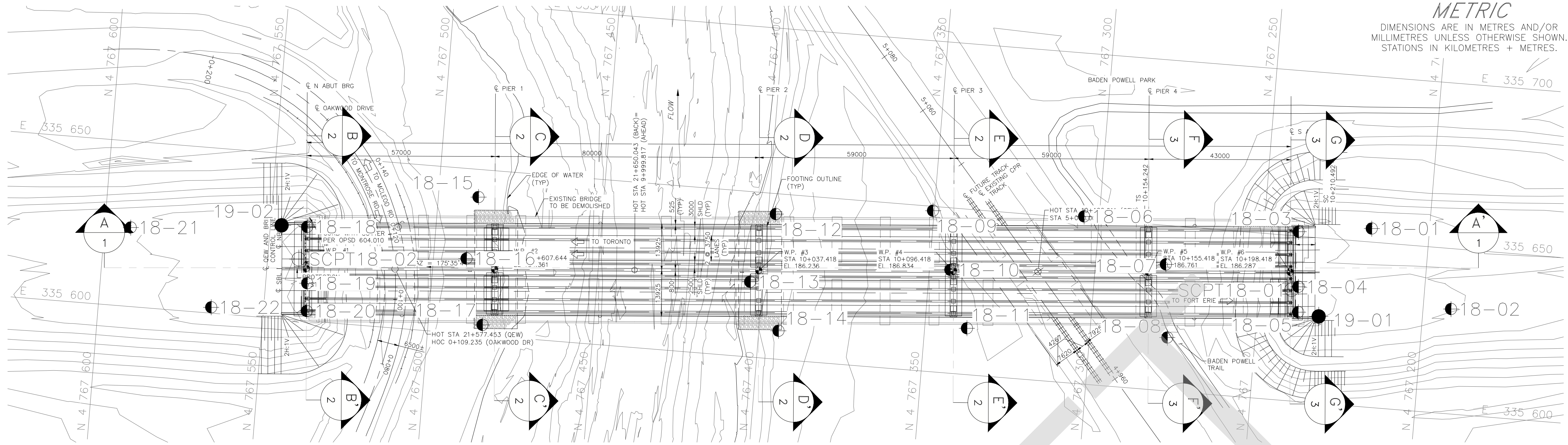
CONT No. DB 2018-2013
WP No. 2430-15-00

REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER

BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 43.046111 LONG. -79.121667

SHEET

GOLDER



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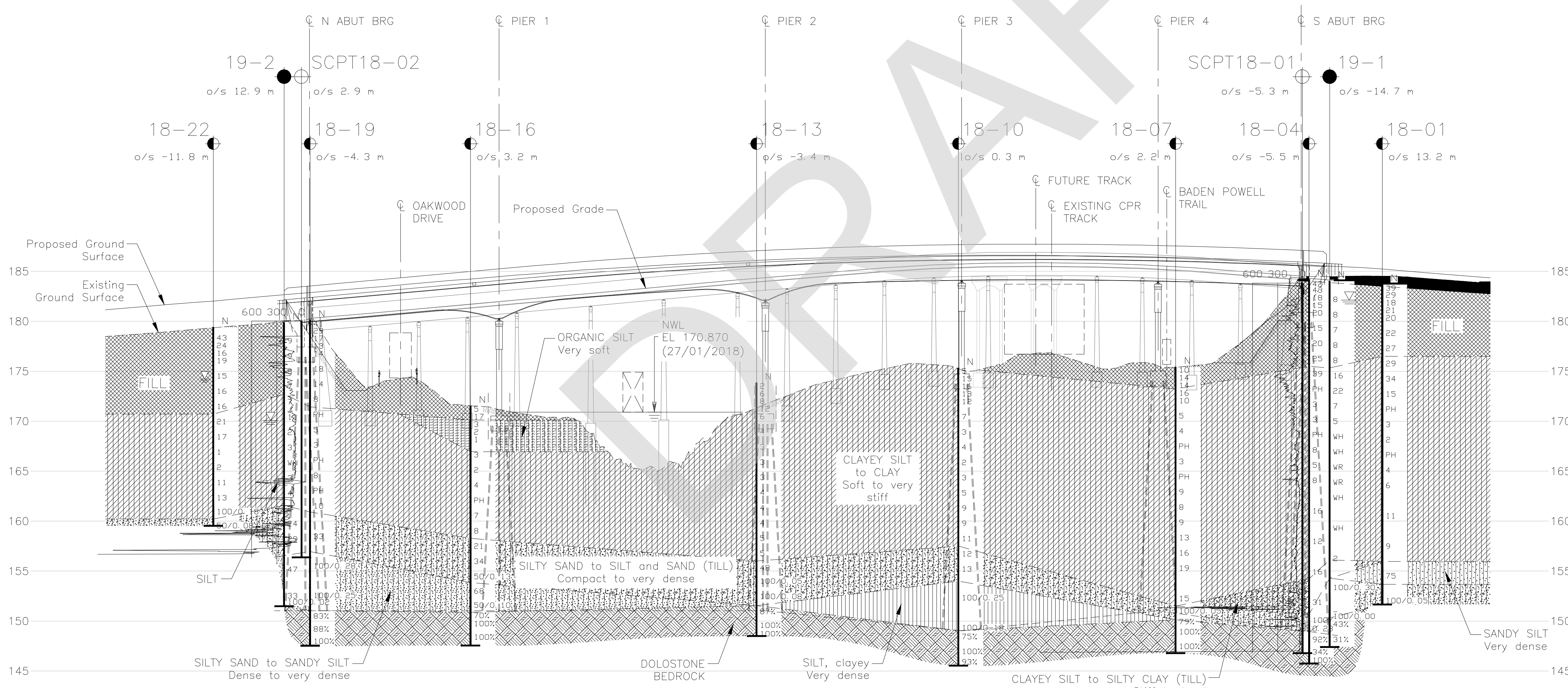

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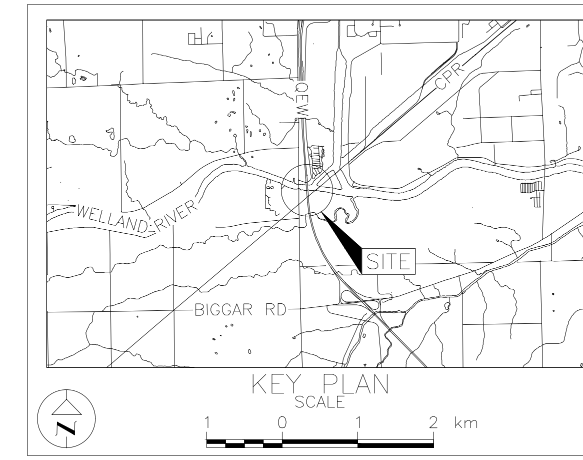

KEY PLAN
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LEGEND

- ● Borehole – Current Investigation
- ◐ Borehole – Previous Investigation (Geocres No. 30M03-307)
- ⊕ SCPT – Previous Investigation (Geocres No. 30M03-307)
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
19-1	184.0	4767231.3	335626.2
19-2	179.9	4767546.6	335628.3
18-01	183.7	4767217.4	335654.1
18-02	183.1	4767191.5	335631.7
18-03	184.3	4767239.4	335651.3
18-04	184.2	4767238.2	335634.7
18-05	184.2	4767237.6	335627.0
18-06	175.6	4767304.6	335650.7
18-07	175.4	4767278.7	335638.2
18-08	176.0	4767276.5	335616.1
18-09	175.0	4767350.3	335648.7
18-10	175.3	4767343.6	335631.3
18-11	175.3	4767337.2	335614.0
18-12	172.2	4767397.7	335643.8
18-13	173.8	4767403.8	335622.7
18-14	173.1	4767394.0	335608.6
18-15	171.4	4767487.9	335641.6
18-16	171.5	4767490.0	335622.8
18-17	171.3	4767483.6	335603.1
18-18	180.1	4767538.9	335628.4
18-19	180.1	4767537.5	335611.4
18-20	180.1	4767536.9	335603.0
18-21	178.9	4767592.1	335623.9
18-22	179.4	4767565.9	335601.8
SCPT18-01	183.8	4767240.1	335633.6
SCPT18-02	180.0	4767540.6	335618.5

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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 Parsons, drawing file no. 476943_00_BRGA_001.dwg, received JULY 4, 2019.

DRAFT

NO.	DATE	BY	REVISION

Geocres No. 30M03-307

HWY. QEW	PROJECT NO. 18109622	DIST. CENTRAL
SUBM'D. MN	CHKD. MN	DATE: 7/5/2019	SITE:
DRAWN: JM	CHKD. SEMP	APPD.	DWG. 1

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

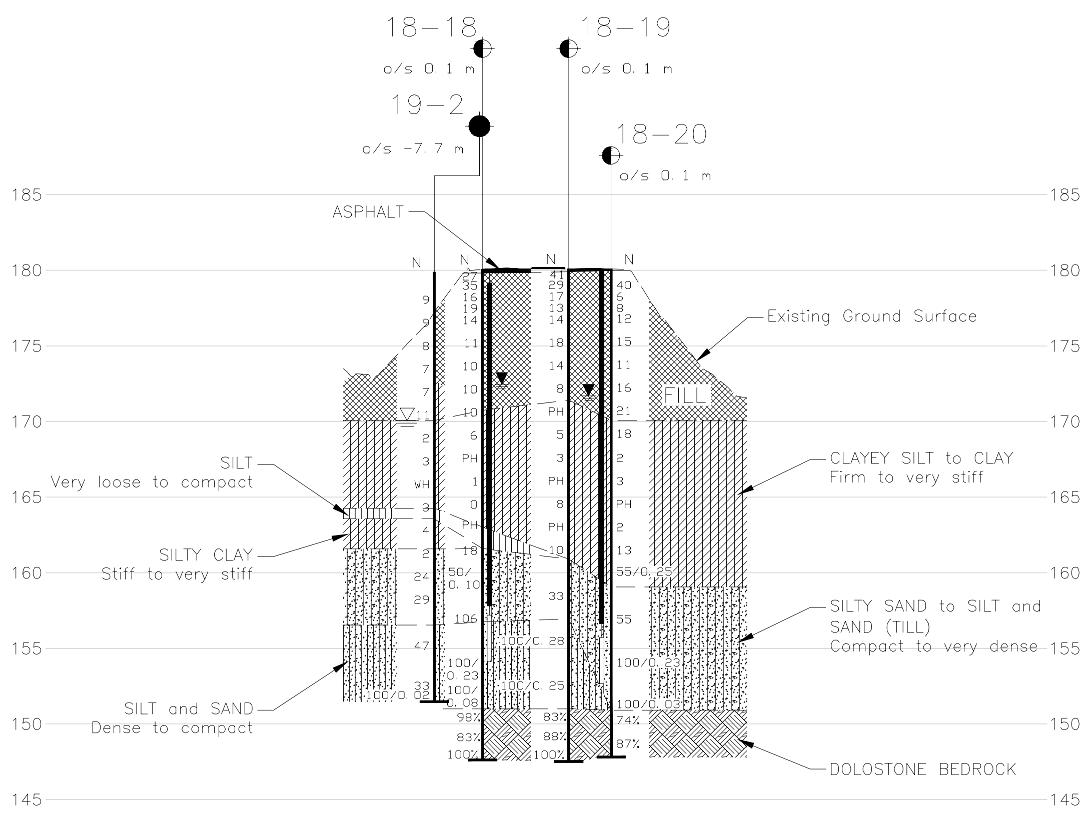
CONT No. DB 2018-2013
WP No. 2430-15-00

REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER

SOIL STRATA
LAT. 43.046111 LONG. -79.121667

SHEET

GOLDER



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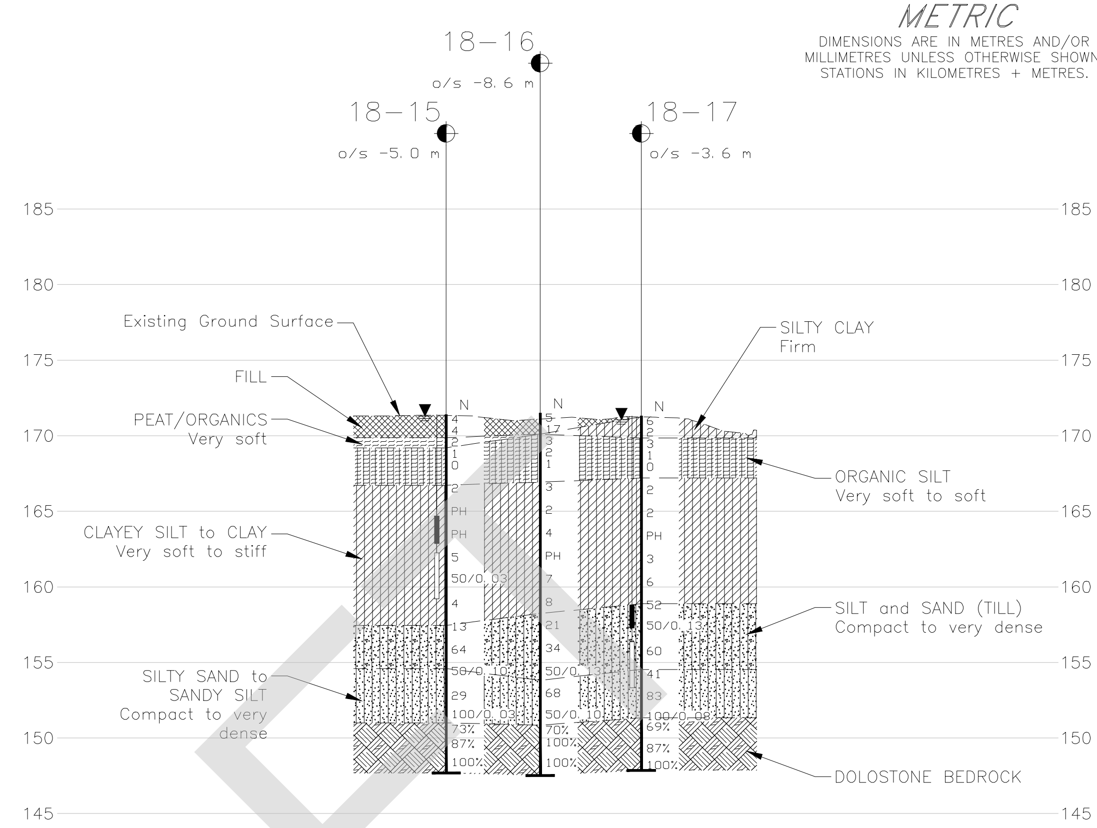
B-B / 2 CROSS-SECTION
NORTH ABUTMENT



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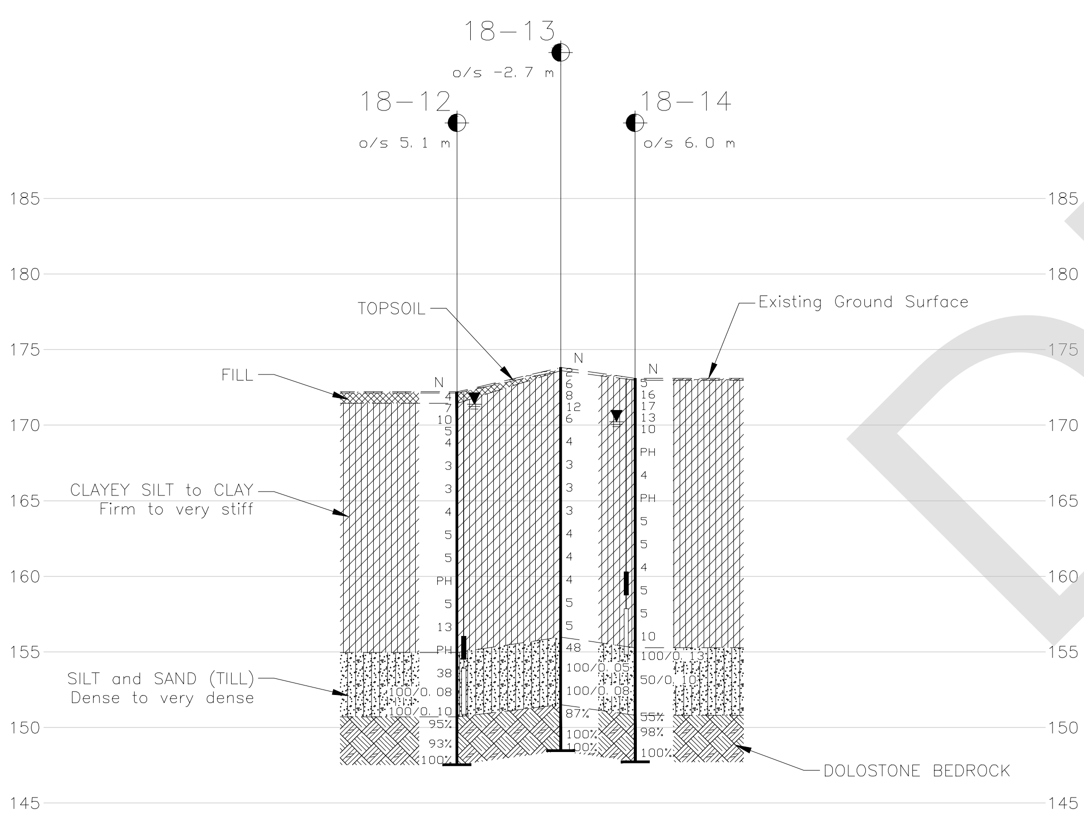
C-C / 2 CROSS-SECTION
PIER 1



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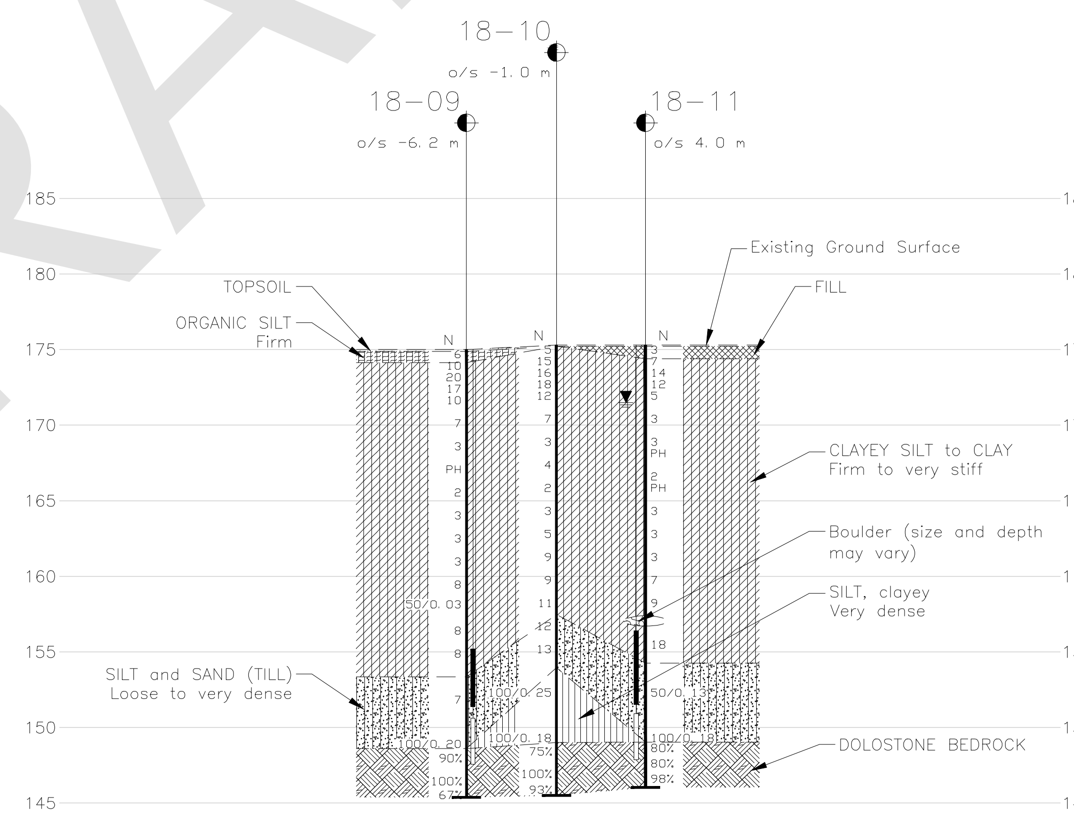
PIER 2



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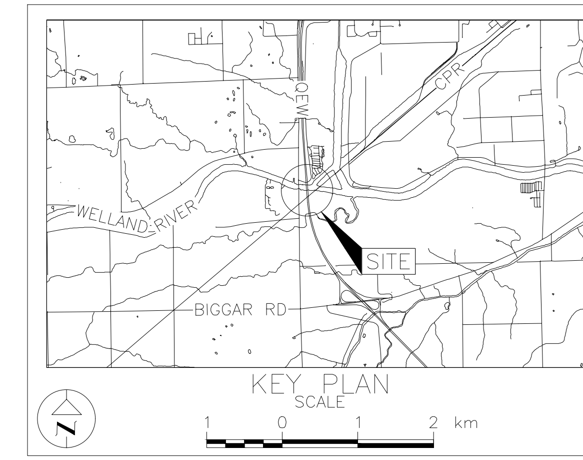
E-E / 2 CROSS-SECTION
PIER 3



KEY PLAN
SCALE

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 30M03-307)
- SCPTU – Previous Investigation (Geocres No. 30M03-307)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on JULY 24, 2018 and MAY 28, 2019
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

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18-11	175.3	4767337.2	335614.0
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18-22	179.4	4767565.9	335601.8
SCPT18-01	183.8	4767240.1	335633.6
SCPT18-02	180.0	4767540.6	335618.5

NO.	DATE	BY	REVISION

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DRAFT

Geocres No. 30M03-307

HWY. QEW	PROJECT NO. 18109622		DIST. CENTRAL
SUBM'D. MN	CHKD. MN	DATE: 7/5/2019	SITE:
DRAWN: JM	CHKD. SEMP	APPD.	DWG. 2

PLOT DATE: July 5, 2019
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METRIC
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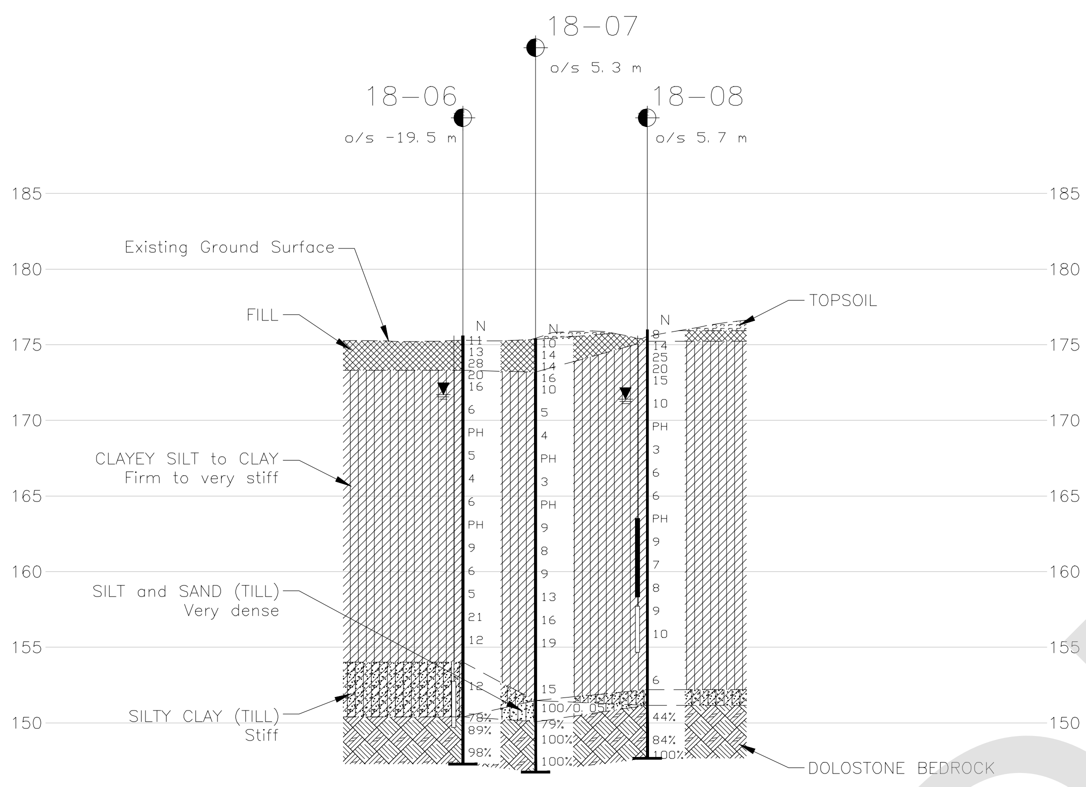
CONT No. DB 2018–2013
WP No. 2430–15–00

REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER

SOIL STRATA
LAT. 43.046111 LONG. –79.121667

SHEET

GOLDER



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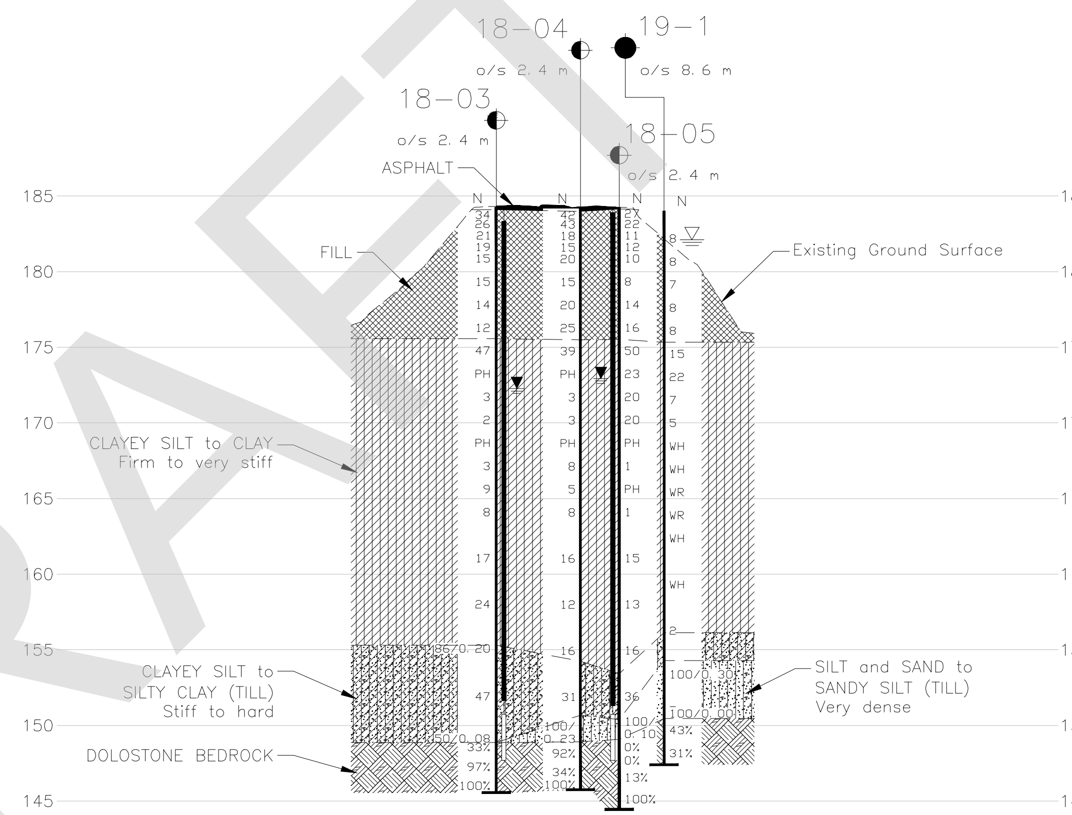

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G–G
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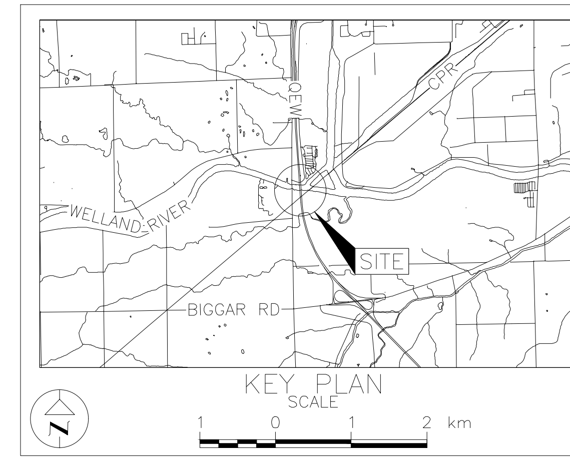
CROSS–SECTION SOUTH ABUTMENT



KEY PLAN
SCALE
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LEGEND

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- WL in piezometer, measured on JULY 24, 2018 and MAY 28, 2019
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BOREHOLE CO–ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
19–1	184.0	4767231.3	335626.2
19–2	179.9	4767546.6	335628.3
18–01	183.7	4767217.4	335654.1
18–02	183.1	4767191.5	335631.7
18–03	184.3	4767239.4	335651.3
18–04	184.2	4767238.2	335634.7
18–05	184.2	4767237.6	335627.0
18–06	175.6	4767304.6	335650.7
18–07	175.4	4767278.7	335638.2
18–08	176.0	4767276.5	335616.1
18–09	175.0	4767350.3	335648.7
18–10	175.3	4767343.6	335631.3
18–11	175.3	4767337.2	335614.0
18–12	172.2	4767397.7	335643.8
18–13	173.8	4767403.8	335622.7
18–14	173.1	4767394.0	335608.6
18–15	171.4	4767487.9	335641.6
18–16	171.5	4767490.0	335622.8
18–17	171.3	4767483.6	335603.1
18–18	180.1	4767538.9	335628.4
18–19	180.1	4767537.5	335611.4
18–20	180.1	4767536.9	335603.0
18–21	178.9	4767592.1	335623.9
18–22	179.4	4767565.9	335601.8
SCPT18–01	183.8	4767240.1	335633.6
SCPT18–02	180.0	4767540.6	335618.5

NO.	DATE	BY	REVISION

Geocres No. 30M03–307

HWY. QEW	PROJECT NO. 18109622		DIST. CENTRAL
SUBM'D. MN	CHKD. MN	DATE: 7/5/2019	SITE:
DRAWN: JM	CHKD. SEMP	APPD.	DWG. 3

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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REFERENCE

Base plans provided in digital format by Parsons, drawing file no. 476943_00_BRGA_001.dwg, received JULY 4, 2019.

DRAFT

MINISTRY OF TRANSPORTATION, ONTARIO

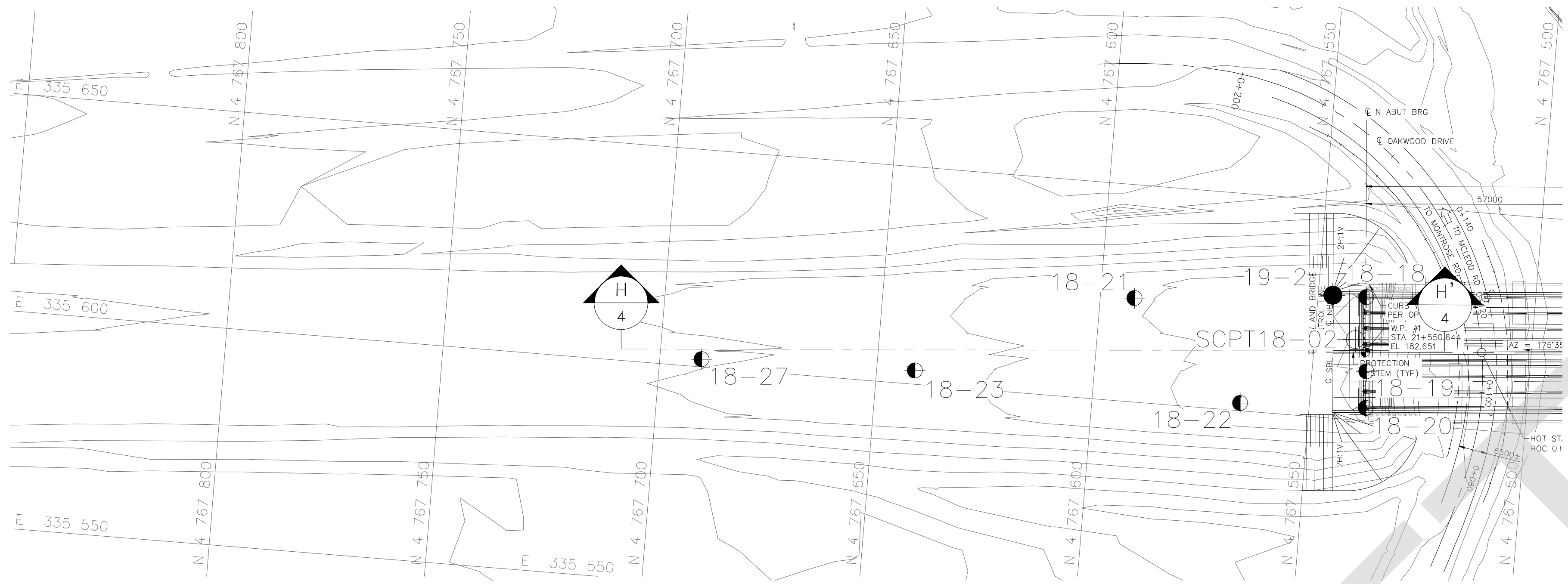
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. DB 2018–2013
WP No. 2430–15–00

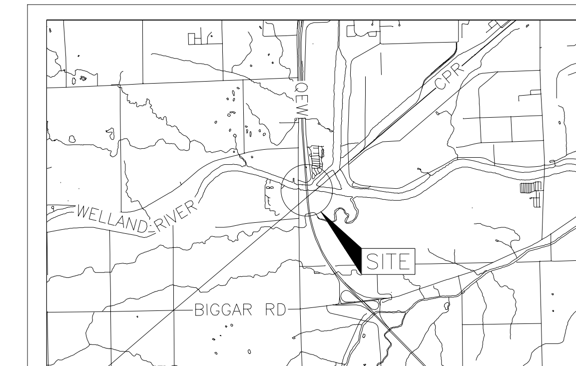
SHEET

REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 43.046111 LONG. –79.121667

GOLDER



SCALE 1:1500 4 PLAN



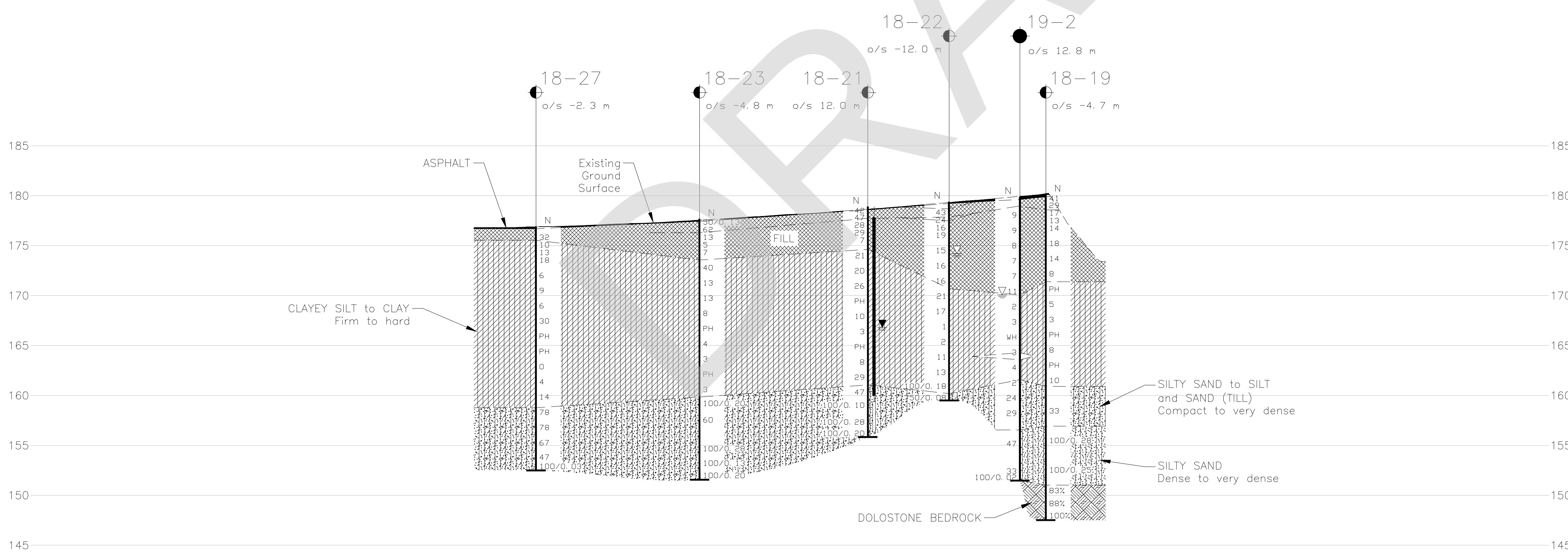
KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 30M03–307)
- SCPTU – Previous Investigation (Geocres No. 30M03–307)
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on MAY 28, 2019
- WL upon completion of drilling

BOREHOLE CO–ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
19–2	179.9	4767546.6	335628.3
18–18	180.1	4767538.9	335628.4
18–19	180.1	4767537.5	335611.4
18–20	180.1	4767536.9	335603.0
18–21	178.9	4767592.1	335623.9
18–22	179.4	4767565.9	335601.8
18–23	177.7	4767641.2	335603.2
18–27	176.9	4767690.4	335601.8
SCPT18–02	180.0	4767540.6	335618.5



HORIZ. SCALE 1:1500 H–H' PROFILE ALONG NORTH APPROACH
VERT. SCALE 1:500 4

NOTES

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REFERENCE

Base plans provided in digital format by Parsons, drawing file no. 476943_00_BRGA_001.dwg, received JULY 4, 2019.

DRAFT

NO.	DATE	BY	REVISION

Geocres No. 30M03–307

HWY. QEW	PROJECT NO. 18109622		DIST. CENTRAL
SUBM'D. MN	CHKD. MN	DATE: 7/5/2019	SITE:
DRAWN: JM	CHKD. SEMP	APPD.	DWG. 4

PLOT DATE: July 5, 2019

MINISTRY OF TRANSPORTATION, ONTARIO

CONT No. DB 2018–2013
WP No. 2430–15–00

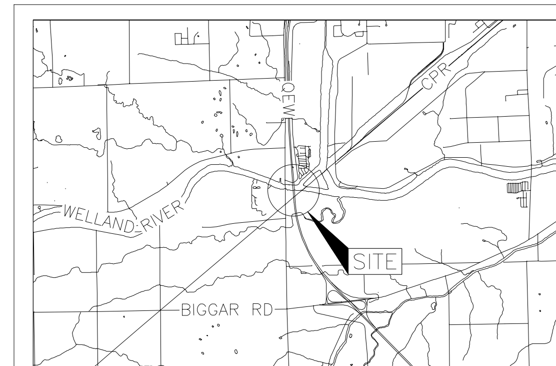
REPLACEMENT OF QEW TWIN BRIDGES OVER WELLAND RIVER
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 43.046111 LONG. –79.121667

SHEET

GOLDER

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

SCALE 1:1500 (5) PLAN



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (Geocres No. 30M03–307)
- SCPTU – Previous Investigation (Geocres No. 30M03–307)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on MAY 28, 2019
- WL upon completion of drilling

BOREHOLE CO-ORDINATES (MTM NAD 83 ZONE 10)

No.	ELEVATION	NORTHING	EASTING
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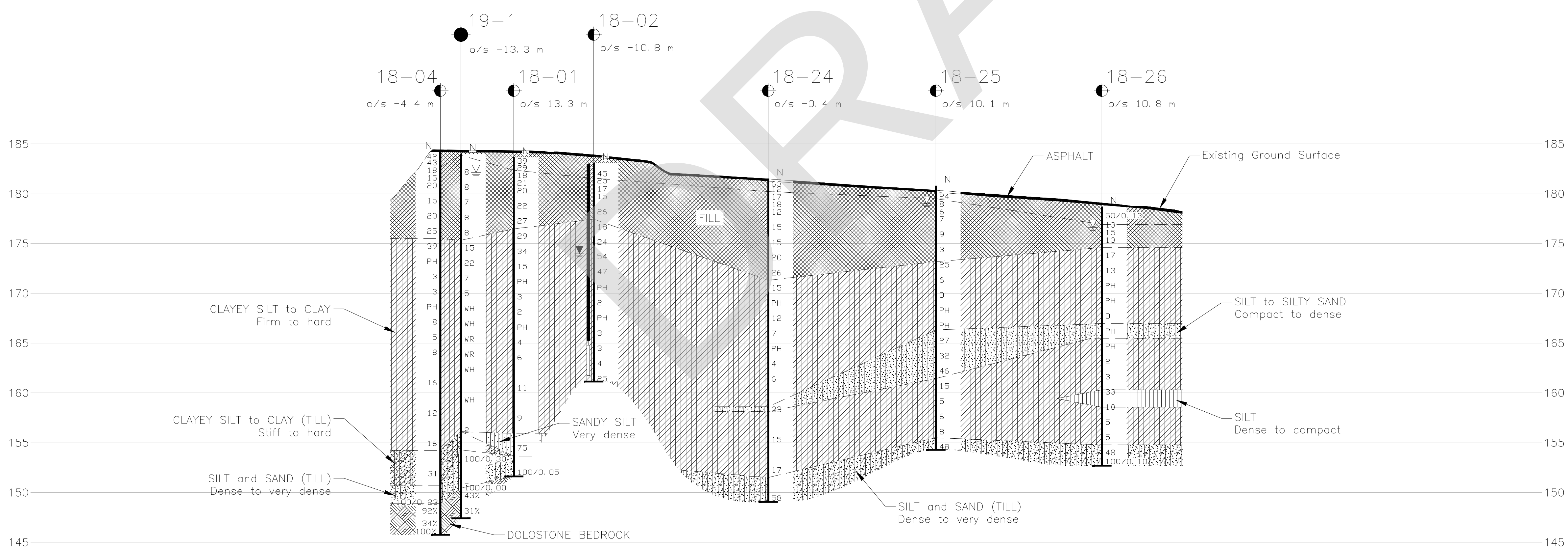
18–03	184.3	4767239.4	335651.3
18–04	184.2	4767238.2	335634.7
18–05	184.2	4767237.6	335627.0
18–24	181.5	4767139.8	335646.2
18–25	180.8	4767090.2	335660.7
18–26	178.7	4767040.3	335665.2
SCPT18–01	183.8	4767240.1	335633.6



HORIZ. SCALE 1:1500 (I–I' 5) PROFILE ALONG SOUTH APPROACH
VERT. SCALE 1:500

NOTES

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REFERENCE

Base plans provided in digital format by Parsons, drawing file no. 476943_00_BRGA_001.dwg, received JULY 4, 2019.

DRAFT

NO.	DATE	BY	REVISION

Geocres No. 30M03–307

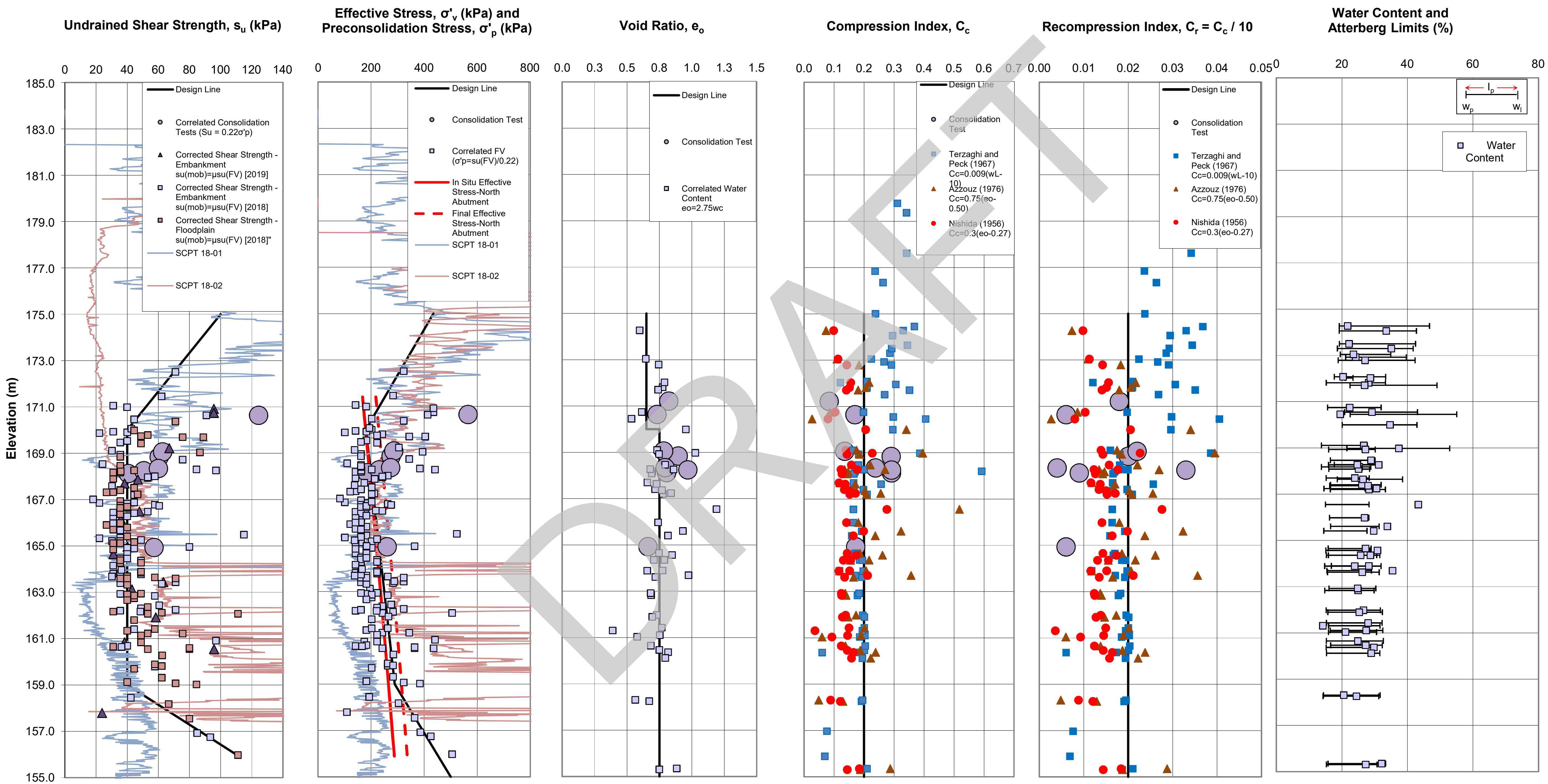
HWY. QEW	PROJECT NO. 18109622		DIST. CENTRAL
SUBM'D. MN	CHKD. MN	DATE: 7/5/2019	SITE:
DRAWN: JM	CHKD. SEMP	APPD.	DWG. 5

PLOT DATE: July 5, 2019
FILENAME: N:\Active\Spatial_IM\MTO\QEW\WellandRiverBridges\99_PROJ\18109622_Parsons_Foundations\40_PROD\0001_GeotechInvestigation\18109622-0001-BD-0005.dwg

C:\Users\mnasr\Golder Associates\18109622, Parsons MTO 2018 2013 Welland River Bridges - Technical Work\Foundation\Analysis\Detailed Design\Settlement\[18109622-Welland Cohesive Design Parameters.xlsx]Plots (2)

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSITS
Welland River Twin Bridge Replacement

FIGURE 1



Date: June 2019
Project No: 18109622

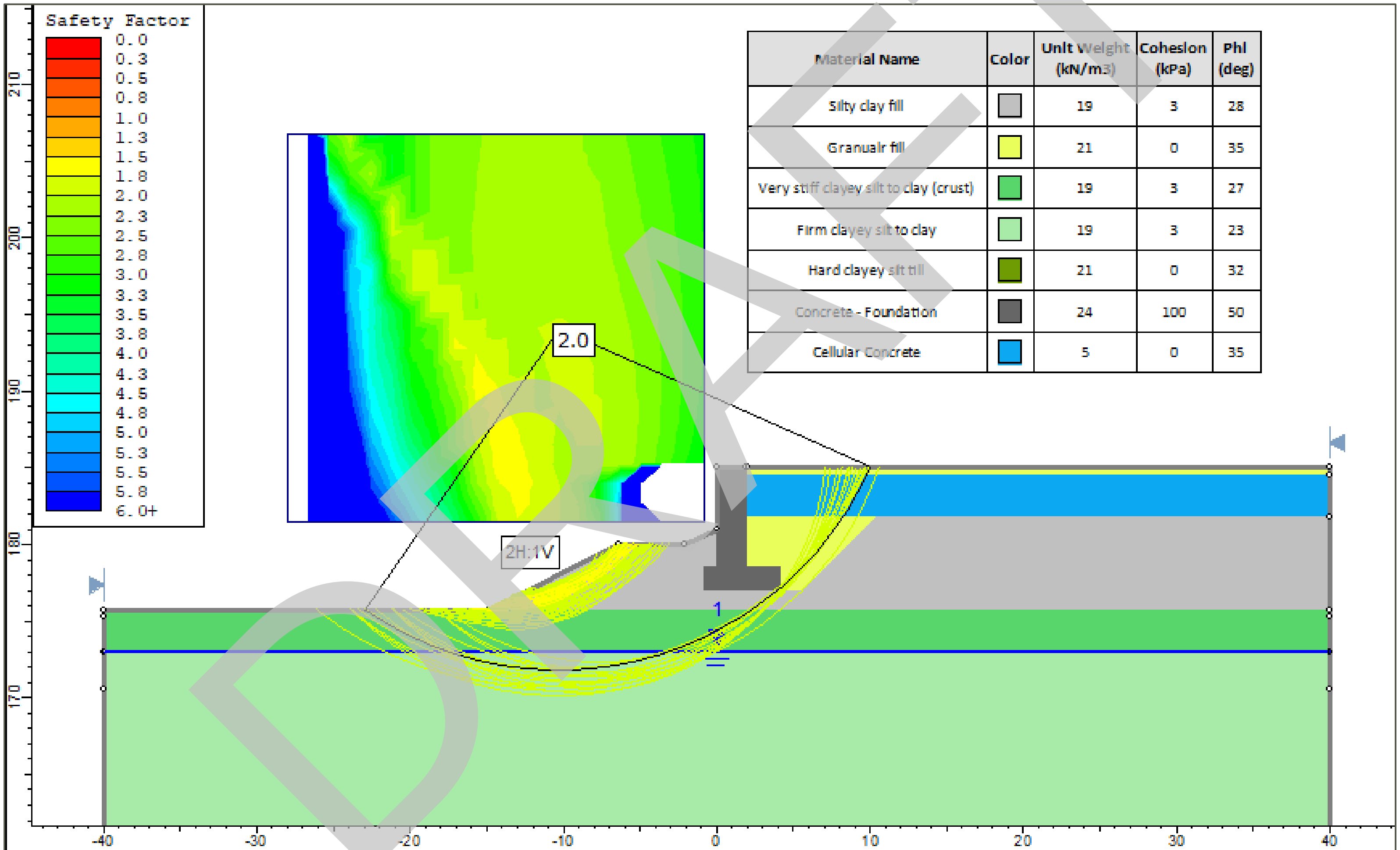
Prepared By: MN
Checked By: SEMP

Golder Associates

Global Stability Analysis

Figure 2

Replacement of QEW Twin Bridges over Welland River
South Abutment Front Slope (Sta. 10+220)
Long-Term (Drained) Analysis

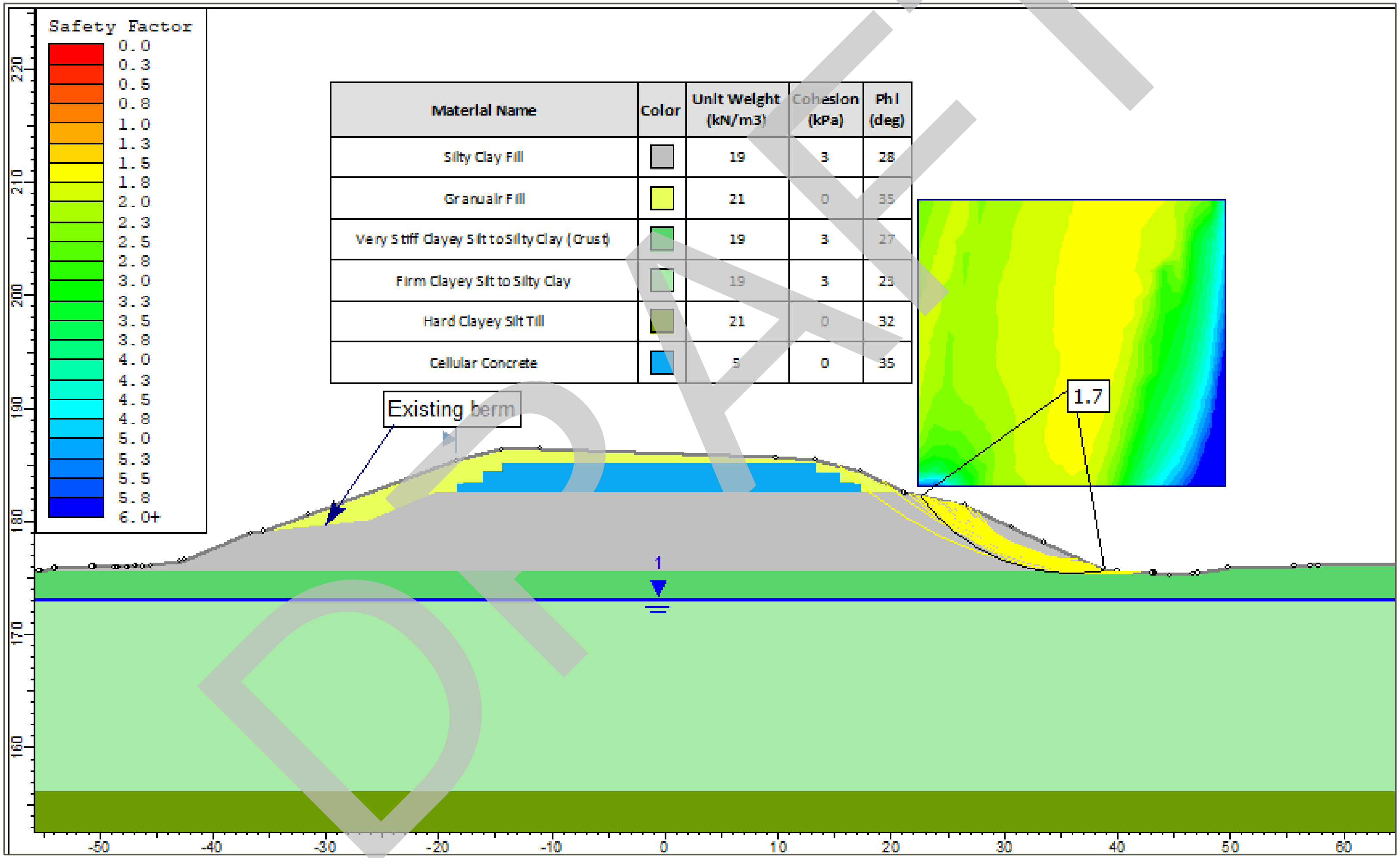


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty clay fill		19	3	28
Granualr fill		21	0	35
Very stiff clayey silt to clay (crust)		19	3	27
Firm clayey silt to clay		19	3	23
Hard clayey silt till		21	0	32
Concrete - Foundation		24	100	50
Cellular Concrete		5	0	35

Global Stability Analysis

Figure 3

Replacement of QEW Twin Bridges over Welland River
South Abutment Side Slope (Sta. 10+220)
Long-Term (Drained) Analysis

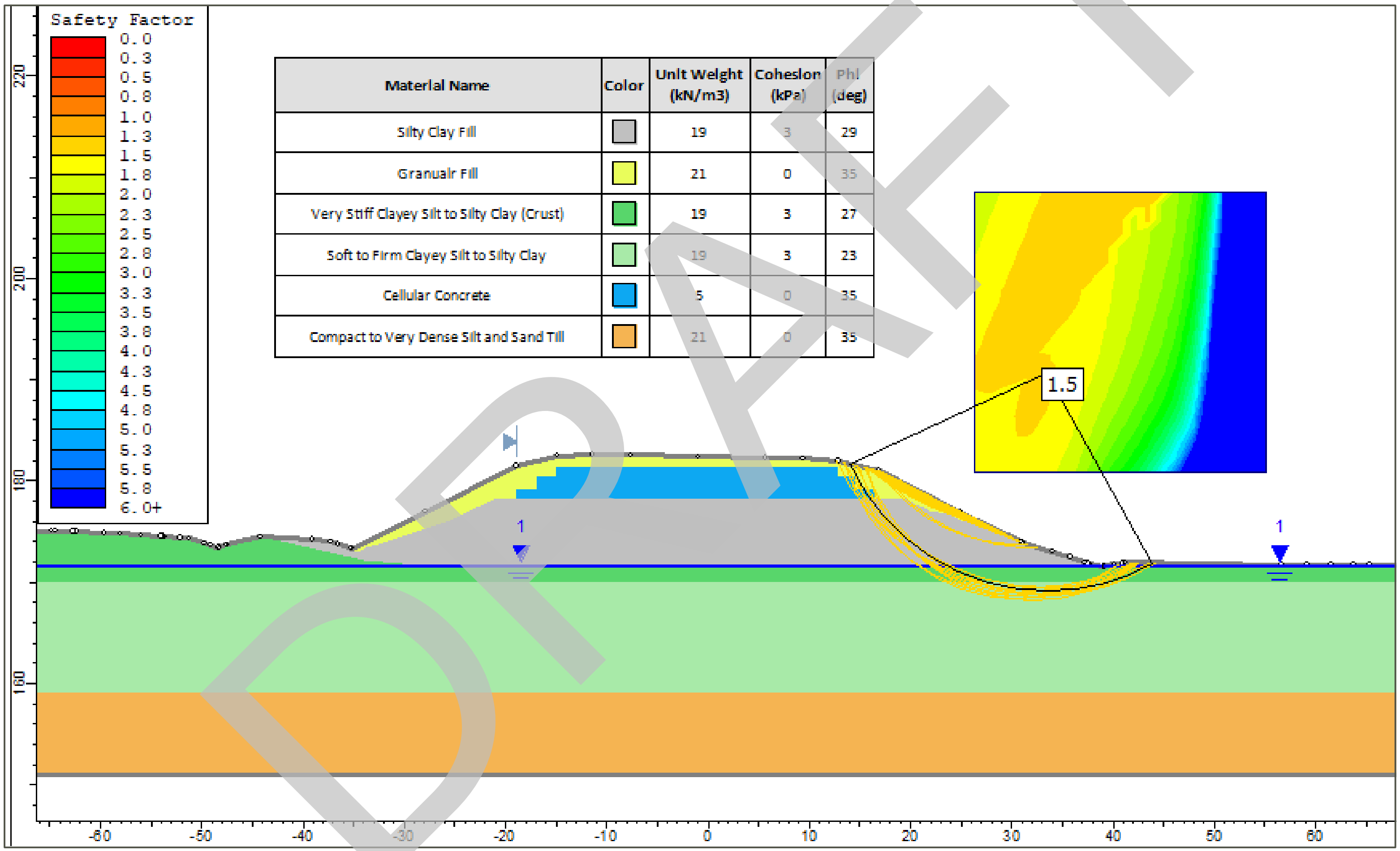


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Fill		19	3	28
Granualr Fill		21	0	35
Very Stiff Clayey Silt to Silty Clay (Crust)		19	3	27
Firm Clayey Silt to Silty Clay		19	3	23
Hard Clayey Silt Till		21	0	32
Cellular Concrete		5	0	35

Global Stability Analysis

Figure 4

Replacement of QEW Twin Bridges over Welland River
North Abutment Side Slope (Sta. 21+540)
Long-Term (Drained) Analysis



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Silty Clay Fill		19	3	29
Granualr Fill		21	0	35
Very Stiff Clayey Silt to Silty Clay (Crust)		19	3	27
Soft to Firm Clayey Silt to Silty Clay		19	3	23
Cellular Concrete		5	0	35
Compact to Very Dense Silt and Sand Till		21	0	35

APPENDIX A

Borehole Records – Previous Investigation (2018)

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction
1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

- SS Split Spoon Sample
- WS Wash Sample
- AS Auger (Grab) Sample
- TW Thin Wall Shelby Tube Sample
- TP Thin Wall Piston Sample
- PH Sampler Advanced by Hydraulic Pressure
- PM Sampler Advanced by Manual Pressure
- WH Sampler Advanced by Self Static Weight
- RC Rock Core
- SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

(1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	(psi)	Field Estimation of Hardness*
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

RECORD OF BOREHOLE No 18-01

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 217.4 E 335 654.1		ORIGINATED BY ES
DIST	HWY QEW	BOREHOLE TYPE NW Casing	COMPILED BY MP
DATUM Geodetic	DATE 2018.04.11 - 2018.04.11	LATITUDE 43.044147 LONGITUDE -79.121259	CHECKED BY GRL

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20, 40, 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
183.7	GROUND SURFACE							
0.0	**ASPHALT**							
0.2	**SAND** and **GRAVEL**, Dense to Compact, Grey, Moist (FILL)	1	SS	39				
		2	SS	29				
182.4 / 1.3	Silty **CLAY**, some sand, trace gravel, Very Stiff, Reddish Brown, Moist (FILL)	3	SS	18				
		4	SS	21				
		5	SS	20				
		6	SS	22	0	0	33	67
		7	SS	27				
176.5 / 7.2	Silty **CLAY**, trace to some sand, Very Stiff to Hard, Reddish Brown, Moist	8	SS	29				
		9	SS	34	0	0	35	65

Elevation scale: 183, 182, 181, 180, 179, 178, 177, 176, 175, 174

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-01

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 217.4 E 335 654.1		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.11 - 2018.04.11	LATITUDE 43.044147 LONGITUDE -79.121259	CHECKED BY GRL

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

Continued From Previous Page

Silty **CLAY**, trace to some sand
Very Stiff
Reddish Brown
Moist

Firm
Wet

Sample Number	Type	"N" Values	Field Vane (Sensitivity)	GR	SA	SI	CL
10	SS	15					
1	TW	PH					
			2.9				
11	SS	3					
			3.2				
12	SS	2		0	10	51	39
			8.0				
2	TW	PH					
13	SS	4					
			3.1				

Elevation scale: 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-01

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 217.4 E 335 654.1 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.11 - 2018.04.11 | LATITUDE 43.044147 | LONGITUDE -79.121259 | CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (field vane, sensitivity)	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace to some sand, trace gravel Firm to Stiff Reddish Brown Wet	14	SS	6			o	0	11	50	39
					163	4.3					
					162						
					161						
		15	SS	11			o				
					160	2.9					
					159						
					158						
		16	SS	9			o	2	9	47	42
					157	4.7					
155.9					156						
27.7	Sandy **SILT**, trace clay Very Dense Reddish Brown Wet										
					155						
		17	SS	75			o	0	24	71	5
153.7					154						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-01

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 217.4 E 335 654.1 | ORIGINATED BY ES

DIST ___ HWY QEW | BOREHOLE TYPE NW Casing | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.11 - 2018.04.11 | LATITUDE 43.044147 | LONGITUDE -79.121259 | CHECKED BY GRL

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
30.0	**SILT** and **SAND**, trace clay, trace gravel Very Dense Grey Moist (TILL)						153 152				
151.6 32.1	END OF BOREHOLE AT 32.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 1.2m, SAND TO 0.3m, CEMENT TO 0.1m THEN ASPHALT TO SURFACE.		18	SS	100/ 0.050				○		

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-02

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 191.5 E 335 631.7 ORIGINATED BY ES

DIST ____ HWY QEW BOREHOLE TYPE NW Casing COMPILED BY MP

DATUM Geodetic DATE 2018.04.08 - 2018.04.09 LATITUDE 43.043907 LONGITUDE -79.121663 CHECKED BY GRL

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.1	GROUND SURFACE				183	
0.0	**ASPHALT**					
0.1	**SAND** and **GRAVEL** Dense Grey Moist (FILL)	1	SS	45	182	
181.6 1.5	Silty **CLAY**, some sand, trace gravel Very Stiff Brown Moist (FILL)	2	SS	25	181	
		3	SS	17	180	
		4	SS	15	179	0 0 43 57
		5	SS	26	178	
177.4 5.6	Silty **CLAY**, trace sand Very Stiff to Hard Reddish Brown to Grey Moist	6	SS	18	177, 176	
		7	SS	24	175	
		8	SS	54	174	

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT; REMARKS & GRAIN SIZE DISTRIBUTION (%).

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-02

METRIC

GWP# 2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 191.5 E 335 631.7		ORIGINATED BY	ES
DIST	HWY QEW	BOREHOLE TYPE	NW Casing	COMPILED BY	MP
DATUM Geodetic	DATE 2018.04.08 - 2018.04.09	LATITUDE 43.043907	LONGITUDE -79.121663	CHECKED BY	GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	Silty **CLAY**, trace sand Hard Reddish Brown to Grey Moist				173							
		9	SS	47	172		○		0	0	37	63
					171							
	Soft Wet	1	TW	PH			○					
					170	+ 5.0						
		10	SS	2	169		├─○┤		0	0	58	42
	Firm					+ 3.2						
					168							
		2	TW	PH			○					
					167	+						
		11	SS	3	166		○					
	cobbles at 17.7m				165							
		12	SS	3			○		0	5	44	51
					164	+ 4.4						

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-02

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 191.5 E 335 631.7		ORIGINATED BY ES
DIST	HWY QEW	BOREHOLE TYPE NW Casing	COMPILED BY MP
DATUM Geodetic	DATE 2018.04.08 - 2018.04.09	LATITUDE 43.043907 LONGITUDE -79.121663	CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	Silty **CLAY**, trace sand Stiff to Very Stiff Reddish Brown to Grey Wet	13	SS	4	163		o					
						+ 4.0						
					162							
161.1		14	SS	25			o		0	0	51	49
21.9	END OF BOREHOLE AT 21.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.											

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.07.12	12.6	170.5

+ 3, × 3: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-03

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 239.4 E 335 651.3 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.09 - 2018.04.10 LATITUDE 43.044355 LONGITUDE -79.121260 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
184.3	GROUND SURFACE									
0.0	**ASPHALT**				184					
0.2	**SAND** and **GRAVEL**, Dense to Compact, Grey, Moist (FILL)	1	SS	34		o				
		2	SS	26	183	o				
182.7 / 1.5	Silty **CLAY**, some sand, trace gravel, Very Stiff to Stiff, Reddish Brown, Moist (FILL)	3	SS	21	182	o				
		4	SS	19		o				
		5	SS	15	181	o				
					180					
		6	SS	15	179	o				
		7	SS	14	178	o	0	0	40	60
					177					
		8	SS	12	176	o				
175.6 / 8.7	Silty **CLAY**, trace sand, Hard, Reddish Brown to Grey, Moist	9	SS	47	175	o				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity   20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

DRAFT

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-03

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 239.4 E 335 651.3		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.09 - 2018.04.10	LATITUDE 43.044355 LONGITUDE -79.121260	CHECKED BY GRL

SOIL PROFILE — Continued From Previous Page

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Firm Reddish Brown Wet					174	
			1	TW	PH		
						173	
						172	
			10	SS	3		0 4 42 54
						171	
			11	SS	2		
						170	
						169	
			2	TW	PH		
	Stiff						
						168	
			12	SS	3		0 0 64 36
						167	
						166	
			13	SS	9		
						165	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values plotted: 7.0, 4.5, 5.2, 4.0, 5.1

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

GROUND WATER CONDITIONS

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-03

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 239.4 E 335 651.3	ORIGINATED BY	ES
DIST		HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE 2018.04.09 - 2018.04.10	LATITUDE 43.044355 LONGITUDE -79.121260	CHECKED BY	GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Stiff Reddish Brown Wet	14	SS	8	164		○ ~25	0	9	40	51
						+ 5.3 (field vane)					
					163						
					162						
	Very Stiff occasional sand seams	15	SS	17	161		○ ~25				
					160						
					159						
		16	SS	24	158		○ ~25				
					157						
					156						
155.3 29.0	Silty **CLAY**, with sand, trace gravel Hard Reddish Brown Wet (TILL)	17	SS	86/ 0.200	155		○ ~15				

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-03

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 239.4 E 335 651.3 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.09 - 2018.04.10 LATITUDE 43.044355 LONGITUDE -79.121260 CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, with sand, trace gravel Hard Reddish Brown Wet (TILL)				154			
					153			
		18	SS	47	152			10 35 31 24 2 21 43 34
					151			
					150			
	rubble zone from 35.1m to 35.4m	19	SS	50/ 0.075			FI	
148.9 / 35.4	**DOLOSTONE BEDROCK** slightly weathered, strong to very strong, grey horizontal fracture at 35.6m, 35.7m, 35.8m, 35.9m and 36.0m	1	RUN		149 / 148		>25 >5 4 0	RUN #1 TCR=100% SCR=71% RQD=33% UCS=221.4MPa (average)
	clay seam at 36.7m and 36.8m horizontal fracture at 36.5m, 36.7m, 36.7m, 36.8m, 36.9m and 37.1m vertical fracture (25mm) at 36.8m	2	RUN		147		1 3 3 0	RUN #2 TCR=100% SCR=97% RQD=97% UCS=98.5MPa (average)
145.6 / 38.7	horizontal fracture at 38.5m and 38.6m	3	RUN		146		1 1 1 4	RUN #3 TCR=100% SCR=100% RQD=100% UCS=148.6MPa (average)

END OF BOREHOLE AT 38.7m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.07.12	5.1	179.2

+3, ×3: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-04

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 238.2 E 335 634.7 | ORIGINATED BY ES

DIST ______ HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.20 - 2018.04.20 | LATITUDE 43.044343 | LONGITUDE -79.121488 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.2	GROUND SURFACE										
0.0	**ASPHALT**						184				
0.2	**SAND** and **GRAVEL** Dense Grey Moist (FILL)		1	SS	42						
183.4 / 0.8	Sandy **GRAVEL**, trace silt Dense Brown Wet (FILL)		2	SS	43		183				64 30 6 (SI+CL)
182.7 / 1.5	Silty **CLAY**, some sand, trace gravel Very Stiff Brown Moist (FILL)		3	SS	18		182				
			4	SS	15						
			5	SS	20		181				
			6	SS	15		180				0 0 40 60
			7	SS	20		179, 178				
			8	SS	25		177, 176				
175.5 / 8.7	Silty **CLAY**, trace sand Hard Reddish Brown Moist		9	SS	39		175				

Continued Next Page

Legend: SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE. PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L.

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

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-04

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 238.2 E 335 634.7 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.20 - 2018.04.20 | LATITUDE 43.044343 | LONGITUDE -79.121488 | CHECKED BY GRL

ELEV DEPTH	DESCRIPTION (Continued From Previous Page)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	Field Vane (kPa) / Sensitivity	Water Content
	Silty **CLAY**, trace sand Stiff Reddish Brown Wet				174		
		1	TW	PH			○
					173		
						+ 3.6	
					172		
	Firm	10	SS	3			○
					171	+ 3.0	
		11	SS	3			○
					170		
						+ 5.8	
					169		
		2	TW	PH			○
					168	+ 3.4	
		12	SS	8			○
					167		
						+ 3.3	
					166		
		13	SS	5			○
					165	+ 3.3	

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-04

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 238.2 E 335 634.7	ORIGINATED BY	ES
DIST		HWY	QEW	BOREHOLE TYPE	NW Casing/NQ Coring
COMPILED BY	MP				
DATUM	Geodetic	DATE	2018.04.20 - 2018.04.20	LATITUDE	43.044343
LONGITUDE	-79.121488	CHECKED BY	GRL		

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE +)	NATURAL MOISTURE CONTENT (%)	GR	SA	SI	CL
	Continued From Previous Page										
	Silty **CLAY**, trace sand Stiff Reddish Brown Wet	14	SS	8	164		~31	0	0	64	36
						4.0 (+)					
					163						
					162						
	Very Stiff	15	SS	16	161		~29				
					160						
					159						
		16	SS	12	158		~28				
					157						
					156						
		17	SS	16	155		~29				
154.2											

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ○ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-04 METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 238.2 E 335 634.7 ORIGINATED BY ES
DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP
DATUM Geodetic DATE 2018.04.20 - 2018.04.20 LATITUDE 43.044343 LONGITUDE -79.121488 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page					
30.0	Silty **CLAY**, some sand, trace gravel, containing cobbles Hard Reddish Brown Wet (TILL)				154 153	
		18	SS	31	152	
	casing refusal, switch to coring gravel and cobbles (max. 150mm) from 32.6m to 35.1m				151	
150.7 33.5	**SILT** and **SAND**, some clay, some gravel, containing cobbles Very Dense Reddish Brown Moist (TILL)				150	
148.9		19	SS	100/ 0.225	149	11 33 37 19
35.3	**DOLOSTONE BEDROCK** slightly weathered, very strong, grey horizontal fracture at 35.4m, 35.5m, 35.8m, 36.1m, 36.4m, 36.6m and 36.7m sub vertical fracture at 35.5m and 35.6m	1	RUN		148	RUN #1 TCR=100% SCR=95% RQD=92% UCS=147.8MPa (average)
	horizontal fracture at 36.9m, 37.0m, 37.1m and 37.2m sub vertical fracture at 37.1m, (50mm) at 37.2m, 37.4m, (100mm) at 37.4m and (75mm) at 37.7m	2	RUN		147	RUN #2 TCR=100% SCR=100% RQD=34% UCS=123.6MPa (average)
145.7		3	RUN		146	RUN #3 TCR=100% SCR=100% RQD=100% UCS=180MPa (average)
38.5	END OF BOREHOLE AT 38.5m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.8m, SAND TO 0.2m, CEMENT TO 0.1m THEN ASPHALT TO SURAFCE.					

Plot data: Sample 18 water content ≈ 37%; Sample 19 water content ≈ 14%.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-05

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 237.6 E 335 627.0		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.07 - 2018.04.08	LATITUDE 43.044330 LONGITUDE -79.121719	CHECKED BY GRL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
184.2	GROUND SURFACE
0.0 / 0.1	**ASPHALT**
	SAND and **GRAVEL** Compact Grey Moist (FILL)
183.4 / 0.9	**SAND**, trace gravel, trace silt Compact Reddish Brown Moist (FILL)
182.7 / 1.5	Silty **CLAY**, some sand, trace gravel Stiff to Very Stiff Reddish Brown Moist (FILL)
175.5 / 8.7	Silty **CLAY**, trace sand Hard Reddish Brown Moist

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	27	
2	SS	22	
3	SS	11	
4	SS	12	
5	SS	10	
6	SS	8	
7	SS	14	
8	SS	16	0 0 43 57
9	SS	50	

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 184, 183, 182, 181, 180, 179, 178, 177, 176, 175

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-05

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 237.6 E 335 627.0		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.07 - 2018.04.08	LATITUDE 43.044330 LONGITUDE -79.121719	CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m3	GR	SA	SI	CL
	Silty **CLAY**, trace sand Very Stiff to Stiff Reddish Brown Moist				174							
		10	SS	23	173				0	0	49	51
	Wet	11	SS	20	172							
		12	SS	20	171 / 170							
		1	TW	PH	169							
					168	+						
	Firm	13	SS	1	167				0	0	53	47
					166	+						
		2	TW	PH	165	9.0 +						

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-05

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 237.6 E 335 627.0		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.07 - 2018.04.08	LATITUDE 43.044330 LONGITUDE -79.121719	CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page													
	Silty **CLAY**, trace sand Firm Reddish Brown Wet		14	SS	1		164		○					
								+ 4.0						
							163							
							162							
	Very Stiff		15	SS	15		161		⊢○⊣		0	9	41	50
							160							
							159							
			16	SS	13		158		○					
							157							
							156							
	Very Stiff		17	SS	16		155		○					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-05

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 237.6 E 335 627.0 | ORIGINATED BY ES

DIST ____ HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.07 - 2018.04.08 | LATITUDE 43.044330 | LONGITUDE -79.121719 | CHECKED BY GRL

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

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT	REMARKS
	Silty **CLAY**, trace sand Very Stiff Reddish Brown Wet				154			
153.4 30.8	Silty **CLAY**, some sand, trace gravel, containing cobbles Hard Reddish Brown Wet (TILL)				153			
		18	SS	36	152	○ (approx. 19%)		GR 3, SA 17, SI 37, CL 43
	casing refusal, switch to coring gravel and cobbles (max. 100mm) from 32.6m to 34.7m				151			
150.4 33.8	**SILT** and **SAND**, some gravel to gravelly, some clay, containing cobbles Very Dense Grey Moist (TILL)				150			
149.1 35.2	**DOLOSTONE BEDROCK**, slightly weathered, strong to very strong, grey	19	SS	100/ 0.100	149	○	FI	
	vertical fracture (25mm) at 35.7m and 35.9m sub vertical fracture (50mm) at 35.8m	1	RUN				>25 >25 >10	RUN #1 TCR=100% SCR=50% RQD=0% UCS=71.2MPa (average)
	sub vertical fracture (50mm) at 36.6m and (25mm) at 36.8m vertical fracture (25mm) at 36.9m	2	RUN		148		5 >25 8	RUN #2 TCR=67% SCR=38% RQD=0% UCS=226.5MPa (average)
	vertical fracture (50mm) at 36.9m and (125mm) at 37.8m sub vertical fracture (50mm) at 37.3m, (25mm) at 38.0m and (50mm) at 38.1m	3	RUN		147 146		>10 >25 >25 10 3	RUN #3 TCR=98% SCR=55% RQD=13% UCS=135.9MPa (average)
	highly broken zone (50mm) at 37.3m, (75mm) at 37.6m and (50mm) at 37.8m	4	RUN		145		0 0 1 0	RUN #4 TCR=100% SCR=100% RQD=100% UCS=167.8MPa (average)
144.4 39.8	END OF BOREHOLE AT 39.8m.							

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-05

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 237.6 E 335 627.0 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.07 - 2018.04.08 LATITUDE 43.044330 LONGITUDE -79.121719 CHECKED BY GRL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2018.07.12 12.6 171.6										

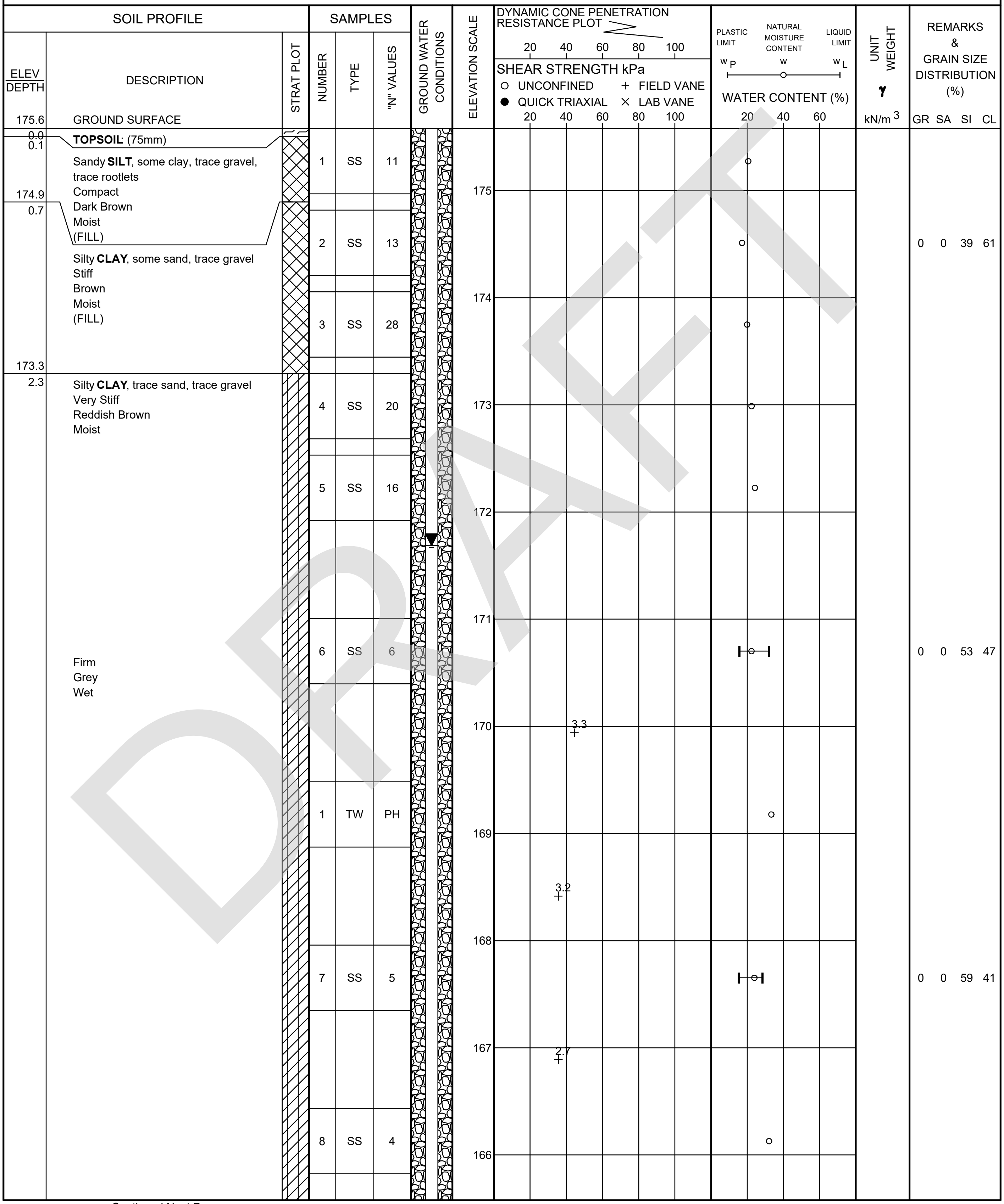
$+^3, \times^3$: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-06

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 304.6 E 335 650.6		ORIGINATED BY ES
DIST ______ HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.26 - 2018.03.27	LATITUDE 43.044934 LONGITUDE -79.121341	CHECKED BY GRL



SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
175.6	GROUND SURFACE
0.0 / 0.1	**TOPSOIL**: (75mm)
	Sandy **SILT**, some clay, trace gravel, trace rootlets Compact Dark Brown Moist (FILL)
174.9 / 0.7	Silty **CLAY**, some sand, trace gravel Stiff Brown Moist (FILL)
173.3 / 2.3	Silty **CLAY**, trace sand, trace gravel Very Stiff Reddish Brown Moist Firm Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	11	
2	SS	13	0 0 39 61
3	SS	28	
4	SS	20	
5	SS	16	
6	SS	6	0 0 53 47
1	TW	PH	
7	SS	5	0 0 59 41
8	SS	4	

Continued Next Page

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

(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-06

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 304.6 E 335 650.6		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.26 - 2018.03.27 LATITUDE 43.044934	LONGITUDE -79.121341	CHECKED BY GRL

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

Continued From Previous Page

Silty **CLAY**, trace sand, trace gravel
Firm to Stiff
Reddish Brown
Wet

very stiff

Sample No.	Type	"N" Values	GR	SA	SI	CL
9	SS	6				
2	TW	PH				
10	SS	9				
11	SS	6				
12	SS	5	2	6	54	38
13	SS	21				

Elevation scale: 165, 164, 163, 162, 161, 160, 159, 158, 157, 156

Field vane sensitivity values: 3.6, 3.1, 4.8, 4.4, 4.7

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-06

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 304.6 E 335 650.6 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.26 - 2018.03.27 LATITUDE 43.044934 LONGITUDE -79.121341 CHECKED BY GRL

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT	REMARKS / GR SA SI CL
	Silty **CLAY**, some sand Stiff Reddish Brown Wet	14	SS	12	○ (~21)		
154.0 / 21.6	Silty **CLAY**, some sand, trace gravel, containing cobbles Stiff Reddish Brown Wet (TILL)	15	SS	12	○ (~25)		4 13 52 31
	casing grinding at 23.6m						
	casing refusal, switch to coring gravel and cobbles (max. 75mm) from 24.4m to 25.2m					FI	
150.4 / 25.2	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey sub vertical fracture (25mm) at 25.2m	1	RUN			2 0	RUN #1 TCR=100% SCR=78% RQD=78% UCS=118.5MPa (average)
	horizontal fracture at 25.9m, 26.0m, 26.1m, 26.2m, 26.5m and 26.6m	2	RUN			3 3 2 2 0	RUN #2 TCR=100% SCR=100% RQD=89% UCS=188.7MPa (average)
	horizontal fracture at 27.1m, 27.7m and 27.9m sub vertical fracture at 28.2m	3	RUN			1 2 1 1	RUN #3 TCR=100% SCR=98% RQD=98% UCS=178.2MPa (average)
147.3 / 28.3	END OF BOREHOLE AT 28.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

Elevation scale: 155, 154, 153, 152, 151, 150, 149, 148

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.24	3.9	171.7

DRAFT

+³, ×³: Numbers refer to Sensitivity

20 / 15 ◆ 5 / 10 (%) STRAIN AT FAILURE

THURBER

RECORD OF BOREHOLE No 18-07

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 278.7 E 335 638.2		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.28 - 2018.03.28	LATITUDE 43.044702 LONGITUDE -79.121495	CHECKED BY GRL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
175.4	GROUND SURFACE
0.0	**TOPSOIL**: (50mm)
0.0	Silty **SAND**, some clay, trace gravel, trace roots, Compact, Dark Brown, Moist (FILL)
174.7 / 0.7	Clayey **SILT**, some sand, trace gravel, trace organics, Stiff to Very Stiff, Reddish Brown, Moist (FILL)
173.2 / 2.2	Silty **CLAY**, trace sand, Very Stiff to Stiff, Reddish Brown, Moist; Wet; Firm

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	10	
2	SS	14	
3	SS	14	
4	SS	16	
5	SS	10	0 0 61 39
6	SS	5	
7	SS	4	
1	TW	PH	
8	SS	3	0 0 57 43

GROUND WATER CONDITIONS

ELEVATION SCALE: 175, 174, 173, 172, 171, 170, 169, 168, 167, 166

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 2.7, 2.9, 3.2

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-07

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 278.7 E 335 638.2 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.28 - 2018.03.28 | LATITUDE 43.044702 | LONGITUDE -79.121495 | CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (field vane)	WATER CONTENT	GR	SA	SI	CL
	Silty **CLAY**, trace sand Firm Grey Wet					4.0					
					165						
		2	TW	PH							
					164						
		9	SS	9	163						
					162	7.0					
		10	SS	8				0	4	50	46
					161						
		11	SS	9	160						
					159	3.6					
	stiff to very stiff	12	SS	13							
					158						
		13	SS	16	157						
					156						

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Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-07

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 278.7 E 335 638.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.28 - 2018.03.28 LATITUDE 43.044702 LONGITUDE -79.121495 CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³ / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Very Stiff Grey Wet	14	SS	19	155			0 0 53 47
					154			
					153			
		16	SS	15	152			
151.5 23.9	**SILT** and **SAND**, trace clay, trace gravel, containing cobbles Very Dense Reddish Brown Wet (TILL) gravel and cobbles (max. 75mm) from 24.4m to 25.3m	17	SS	100/ 0.050	151			
150.1 25.3	**DOLOSTONE BEDROCK** slightly weathered, very strong, grey horizontal fracture at 25.4m and 25.6m	1	RUN		150		FI 4 0 0	RUN #1 TCR=100% SCR=79% RQD=79% UCS=171.5MPa (average)
	horizontal fracture at 26.9m and 27.1m sub vertical fracture (25mm) at 27.3m	2	RUN		149		0 0 0 2	RUN #2 TCR=100% SCR=100% RQD=100% UCS=168.8MPa (average)
146.7	horizontal fracture at 27.5m horizontal fracture at 28.4m	3	RUN		148 147		1 0 0 1	RUN #3 TCR=100% SCR=100% RQD=100% UCS=158.8MPa (average)
28.7	END OF BOREHOLE AT 28.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.							

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-08

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 276.5 E 335 616.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.27 - 2018.03.28 LATITUDE 43.044683 LONGITUDE -79.121766 CHECKED BY GRL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
176.0	GROUND SURFACE							
0.0	**TOPSOIL**: (75mm)							
0.1	Silty **SAND**, some clay, some gravel, trace rootlets, Loose, Dark Brown, Moist (FILL)	1	SS	8	11	53	22	14
175.2 / 0.8	Silty **CLAY**, trace to some sand, trace gravel, trace organics, Stiff to Very Stiff, Reddish Brown, Moist	2	SS	14				
		3	SS	25				
		4	SS	20	0	0	52	48
		5	SS	15				
		6	SS	10				
	Firm, Wet	1	TW	PH				
		7	SS	3	0	0	53	47
		8	SS	6				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m^3.

Elevation scale: 175, 174, 173, 172, 171, 170, 169, 168, 167, 166

Field vane sensitivity values: 4.0 (approx. El. 170.3), 3.2 (approx. El. 168.8), 3.5 (approx. El. 167.3)

Continued Next Page

+ ^{3}, × ^{3}: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-08

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 276.5 E 335 616.2 | ORIGINATED BY ES

DIST ___ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.27 - 2018.03.28 | LATITUDE 43.044683 | LONGITUDE -79.121766 | CHECKED BY GRL

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ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (FIELD VANE, SENSITIVITY)	WATER CONTENT	GR	SA	SI	CL
	Silty **CLAY**, some sand Firm Reddish Brown Wet					3.6					
		9	SS	6	165		o				
						3.7					
					164						
		2	TW	PH			o				
					163	3.6					
		10	SS	9	162		o				
						4.0					
					161						
		11	SS	7			⊢o⊣	0	10	46	44
					160	4.0					
		12	SS	8	159		o				
	Stiff					4.3					
					158						
		13	SS	9			o				
					157						
					156						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-08

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 276.5 E 335 616.2		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.27 - 2018.03.28	LATITUDE 43.044683 LONGITUDE -79.121766	CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION (Continued From Previous Page)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, some sand Stiff Reddish Brown Wet	14	SS	10	155			
		15	SS	6	154 153			0 16 48 36
152.2 23.8	probable silty clay till from 23.8m to 24.8m gravel and cobbles (max. 125mm) from 23.8m to 24.4m				152			
151.1 24.8	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey rubble zone from 25.0m to 25.2m horizontal fracture at 25.4m, 25.6m, 25.8m and 26.1m sub vertical fracture at 25.9m	1	RUN		151 150		FI >10 >10 3 2 1 1	RUN #1 TCR=92% SCR=73% RQD=44% UCS=126.7MPa (average)
	horizontal fracture at 26.5m, 26.7m, 26.7m, 27.0m and 27.3m sub vertical fracture at 27.5m and 27.7m	2	RUN		149		5 1 1 1	RUN #2 TCR=100% SCR=98% RQD=84% UCS=159.1MPa (average)
147.6 28.3		3	RUN		148		0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=193.9MPa (average)

END OF BOREHOLE AT 28.3m.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.24	4.6	171.4

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; UNIT WEIGHT γ kN/m³

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

DRAFT

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-09

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 350.3 E 335 648.7 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.17 - 2018.03.18 LATITUDE 43.045346 LONGITUDE -79.121363 CHECKED BY GRL

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
175.0	GROUND SURFACE					
0.0	**TOPSOIL**: (125mm)					
0.1	Organic **SILT**, some sand, trace gravel, trace roots Firm Dark Brown Moist	1	SS	6		
174.2 / 0.9	Silty **CLAY**, trace to some sand, trace gravel Stiff to Very Stiff Reddish Brown Moist	2	SS	10	174	
		3	SS	20	173	
		4	SS	17	172	
		5	SS	10	171	0 0 38 62
	Firm	6	SS	7	170	
					169	
		7	SS	3	168	
		1	TW	PH	167	
					166	
		8	SS	2		0 0 46 54

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane values (sensitivity): 2.7 (approx. elev. 169.5), 3.5 (approx. elev. 167.9), 4.0 (approx. elev. 166.4)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-09

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 350.3 E 335 648.7 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.17 - 2018.03.18 LATITUDE 43.045346 LONGITUDE -79.121363 CHECKED BY GRL

Continued From Previous Page

DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
Silty **CLAY**, trace sand Firm Reddish Brown Moist					
	9	SS	3	164	
	10	SS	3	163	
				162	
Stiff	11	SS	3	161	0 4 53 43
				160	
	12	SS	8		
silt seams at 15.8m casing grinding at 16.0m				159	
cobbles at 16.9m	13	SS	50/ 0.025	158	
				157	
	14	SS	8	156	

Field vane shear strength values: 3.0, 3.3, 4.0

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-09

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 350.3 E 335 648.7 | ORIGINATED BY ES

DIST ___ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.17 - 2018.03.18 | LATITUDE 43.045346 | LONGITUDE -79.121363 | CHECKED BY GRL

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

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT / FI	REMARKS / GRAIN SIZE (GR SA SI CL)
	Silty **CLAY**, trace sand Stiff Reddish Brown Moist	15	SS	8	154		
153.4 21.6	**SILT** and **SAND**, some clay, trace gravel, containing cobbles Loose to Very Dense Reddish Brown Wet (TILL)				153		
		16	SS	7	152		9 36 34 21
	casing refusal, switch to coring gravel and cobbles (max. 175mm) from 23.5m to 25.9m				151, 150		
		17	SS	100/ 0.200	149	FI	
148.6 26.4	**DOLOSTONE BEDROCK** slightly weathered, very strong, grey sub vertical fracture at 26.5m horizontal fracture at 27.2m	1	RUN		148	3, 1, 1, 0, 0	RUN #1 TCR=100% SCR=90% RQD=90% UCS=193.3MPa (average)
	sub vertical fracture (75mm) at 28.1m and (25mm) at 29.1m horizontal fracture at 28.3m, 28.6m and 28.8m	2	RUN		147, 146	0, 2, 0, 1, 1	RUN #2 TCR=100% SCR=100% RQD=100% UCS=136.0MPa (average)
		3	RUN			0	RUN #3 TCR=67% SCR=67% RQD=67% UCS=244.3MPa (average)
145.4 29.6	END OF BOREHOLE AT 29.6m. Piezometer installation consists of						

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-09

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 350.3 E 335 648.7 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.17 - 2018.03.18 | LATITUDE 43.045346 | LONGITUDE -79.121363 | CHECKED BY GRL

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.										

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

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

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Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-10

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 343.7 E 335 631.3		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.16 - 2018.03.17	LATITUDE 43.045286 LONGITUDE -79.121577	CHECKED BY GRL

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
175.3	GROUND SURFACE							
0.0 / 0.1	**TOPSOIL**: (75mm)							
	Silty **CLAY**, some sand, trace gravel, trace rootlets, occasional wood fibres Firm Dark Brown Moist	1	SS	5				
174.4 / 0.9	Silty **CLAY**, trace to some sand, trace gravel Very Stiff to Stiff Reddish Brown Moist	2	SS	15	0	0	67	33
		3	SS	16				
		4	SS	18				
		5	SS	12				
		6	SS	7	0	0	31	69
	Firm Wet	7	SS	3				
		8	SS	4	0	0	56	44
		9	SS	2				

Elevation scale: 175, 174, 173, 172, 171, 170, 169, 168, 167, 166

Field vane sensitivity: 3.1 (approx. El. 169.7), 2.8 (approx. El. 168.1), 2.7 (approx. El. 166.6)

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-10

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 343.7 E 335 631.3		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.16 - 2018.03.17	LATITUDE 43.045286 LONGITUDE -79.121577	CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Firm Reddish Brown Wet				165	+ 3.5					
		10	SS	3	164	+	o				
		11	SS	5	163		├o┤	0	0	57	43
	switch to casing cobbles from 13.0m to 13.9m				162						
		12	SS	9	161		o├┤				
	Stiff				160						
		13	SS	9	159		o				
		14	SS	11	158		o				
157.5 17.8	**SILT** and **SAND**, gravelly, trace clay Compact Reddish Brown Wet (TILL)				157						
		15	SS	12	156		o				

Continued Next Page

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-10

 METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 343.7 E 335 631.3		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.16 - 2018.03.17	LATITUDE 43.045286 LONGITUDE -79.121577	CHECKED BY GRL

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ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	UNIT WEIGHT / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILT and **SAND**, gravelly, trace clay Compact Reddish Brown Wet (TILL)	16	SS	13	○		30 34 27 9
154.0 / 21.3	**SILT**, clayey, trace sand, containing cobbles Very Dense Reddish Brown Moist	17	SS	100/ 0.250	○		0 5 69 26
	casing refusal, switch to coring gravel and cobbles (max. 75mm) from 23.1m to 25.9m	18	SS	100/ 0.175	○		
149.0 / 26.3	**DOLOSTONE BEDROCK** slightly weathered, very strong, grey horizontal fracture at 26.5m, 26.6m, 26.8m, 26.9m, 27.0m and 27.4m sub horizontal fracture (25mm) at 27.3m	1	RUN			FI: >10, 4, 3, 2, 1	RUN #1 TCR=94% SCR=83% RQD=75% UCS=153.8MPa (average)
	horizontal fracture at 27.9m, 28.0m, 28.4m and 28.5m	2	RUN			1, 1, 2, 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=148.1MPa (average)
145.5	sub vertical fracture (50mm) at 29.3m	3	RUN			1, 1, 0	RUN #3 TCR=100% SCR=100% RQD=93% UCS=113.0MPa (average)
29.8	END OF BOREHOLE AT 29.8m.						

Elevation scale: 155, 154, 153, 152, 151, 150, 149, 148, 147, 146

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DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity. 15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

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THURBER

RECORD OF BOREHOLE No 18-10

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 343.7 E 335 631.3		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.16 - 2018.03.17 LATITUDE 43.045286	LONGITUDE -79.121577	CHECKED BY GRL

ELEV DEPTH	SOIL PROFILE: DESCRIPTION (Continued From Previous Page)	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.										

+³, ×³: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-11

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 337.2 E 335 614.0 | ORIGINATED BY ES

DIST ______ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.14 - 2018.03.15 | LATITUDE 43.045230 | LONGITUDE -79.121789 | CHECKED BY GRL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
175.3	GROUND SURFACE
0.0 / 0.1	**TOPSOIL**: (75mm)
	Silty **CLAY**, some sand, trace rootlets Soft Brown Moist (FILL)
174.4 / 0.9	Silty **CLAY**, trace to some sand, trace gravel Firm to Stiff Reddish Brown Moist occasional sand pockets Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	3				
2	SS	7				
3	SS	14	0	0	52	48
4	SS	12				
5	SS	5				
6	SS	3				
7	SS	3	0	0	49	51
1	TW	PH				
8	SS	2	0	0	59	41
2	TW	PH				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (175, 174, 173, 172, 171, 170, 169, 168, 167, 166); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³.

Field vane sensitivity values: 2.8, 2.7

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+ 3, × 3: Numbers refer to Sensitivity — 20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-11

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 337.2 E 335 614.0 | ORIGINATED BY ES

DIST ____ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.14 - 2018.03.15 | LATITUDE 43.045230 | LONGITUDE -79.121789 | CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace to some sand Firm to Very Stiff Reddish Brown Wet				165	
		9	SS	3		0 0 53 47
					164	
		10	SS	3	163	
					162	
		11	SS	3	161	
					160	
		12	SS	7		
					159	
		13	SS	9	158	0 18 47 35
	casing refusal at 17.9m on boulder, switched to NQ Coring boulder (275mm) at 18.3m				157	
					156	
		14	SS	18		

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100. Plotted value: 5.1

Plastic limit w_P — Natural moisture content w — Liquid limit w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3 , × 3 : Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-11

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 337.2 E 335 614.0 | ORIGINATED BY ES

DIST ____ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.14 - 2018.03.15 | LATITUDE 43.045230 | LONGITUDE -79.121789 | CHECKED BY GRL

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT	REMARKS
					155		
154.3 / 21.0	**SILT** and **SAND**, clayey, trace gravel, containing cobbles Very Dense Grey Moist (TILL)				154		
					153		
		15	SS	50/ 0.125			
	gravel and cobbles (max. 125mm) from 23.1m to 25.9m				152		
					151		
					150		
		16	SS	100/ 0.175		FI	
149.0	rubble zone from 26.1m to 27.3m					>25	
26.3	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey				149	2	
	horizontal fracture at 26.4m and 27.0m	1	RUN			2	RUN #1 TCR=100% SCR=84% RQD=80% UCS=207.0MPa (average)
					148	2	
	horizontal fracture at 27.3m, 27.4m, 28.0m and 28.1m sub horizontal fracture at 27.8m	2	RUN			2 1 3	RUN #2 TCR=100% SCR=100% RQD=80% UCS=157.6MPa (average)
	horizontal fracture at 28.3m and 28.5m	3	RUN		147	2 1 0	RUN #3 TCR=100% SCR=100% RQD=98% UCS=208.1MPa (average)
146.0 / 29.3	END OF BOREHOLE AT 29.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.					0	

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+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-11

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 337.2 E 335 614.0			ORIGINATED BY ES
DIST	HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.14 - 2018.03.15	LATITUDE 43.045230	LONGITUDE -79.121789	CHECKED BY GRL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
	WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2018.05.24 3.8 171.5										

+ ³, × ³: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-12

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 397.7 E 335 643.8 | ORIGINATED BY ES

DIST ______ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.19 - 2018.03.20 LATITUDE 43.045773 LONGITUDE -79.121420 | CHECKED BY GRL

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
172.2	GROUND SURFACE					
0.0 / 0.1	**TOPSOIL**: (75mm)					
	Silty **CLAY**, trace sand, trace roots Firm Dark Brown Moist	1	SS	4	172	
171.5 / 0.8	(FILL)					
	Silty **CLAY**, trace sand, trace gravel, trace roots Firm to Stiff Reddish Brown to Grey Moist	2	SS	7	171	
		3	SS	10	170	0 0 51 49
	occasional wood fibre	4	SS	5		
	Wet	5	SS	4	169	0 0 53 47
					168	
		6	SS	3	167	
		7	SS	3	166	
					165	
		8	SS	4	164	0 0 68 32
		9	SS	5	163	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 3.2, 3.0, 2.8, 3.6

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-12

 METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 397.7 E 335 643.8 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.19 - 2018.03.20 LATITUDE 43.045773 LONGITUDE -79.121420 CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Stiff Grey Wet				162	3.0	
		10	SS	5	161		
						3.4	
		1	TW	PH	160		
	switch to casing				159		
		11	SS	5	158		0 7 45 48
	casing grinding at 14.5m						
		12	SS	13	157		
					156		
155.0		2	TW	PH	155		
17.2	**SILT** and **SAND**, some gravel, trace clay, containing cobbles Dense to Very Dense Reddish Brown Moist (TILL)				154		
		13	SS	38	153		19 41 31 9
		14	SS	100/			

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

WATER CONTENT (%): PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; 20 40 60

UNIT WEIGHT γ kN/m^3

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-12

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 397.7 E 335 643.8 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.19 - 2018.03.20 LATITUDE 43.045773 LONGITUDE -79.121420 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page			0.075	152		
150.7		15	SS	100/	151	FI	
21.5	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey horizontal fracture at 21.7m, 21.8m, 22.4m and 22.5m sub horizontal fracture at 21.6m, 21.8m and 22.3m horizontal fracture at 22.6m and 23.5m	1	RUN	0.100	150	>20 7 0 3	RUN #1 TCR=100% SCR=95% RQD=95% UCS=121.9MPa (average)
	sub horizontal fracture at 22.6m, 22.9m, 23.1m, 23.3m and 23.4m	2	RUN		149	3 2 1 1	RUN #2 TCR=100% SCR=95% RQD=93% UCS=233.9MPa (average)
147.6	sub horizontal fracture at 24.1m	3	RUN		148	0 1 1	RUN #3 TCR=100% SCR=100% RQD=100% UCS=157.1MPa (average)
24.7	END OF BOREHOLE AT 24.7m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2018.05.24 0.8 171.4						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%): 20 40 60

+³, ×³: Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-13

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 383.6 E 335 623.2 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.18 - 2018.03.19 | LATITUDE 43.045788 | LONGITUDE -79.121632 | CHECKED BY GRL

SOIL PROFILE

ELEV DEPTH	DESCRIPTION	STRAT PLOT
173.8	GROUND SURFACE	
0.0	**TOPSOIL**: (175mm)	
0.2	Silty **CLAY**, trace rootlets Soft to Firm Brown to Reddish Brown Wet occasional wood fibres	
172.3 1.5	Silty **CLAY**, trace sand Stiff to Firm Reddish Brown Moist	

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	FIELD VANE SENSITIVITY	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	2						
2	SS	6			0	0	32	68
3	SS	8						
4	SS	12						
5	SS	6						
				3.1				
6	SS	4			0	0	28	72
				2.7				
7	SS	3						
				2.6				
8	SS	3						
				2.6				
9	SS	3			0	4	53	43

ELEVATION SCALE: 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m3

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-13

METRIC

GWP#	2430-15-00
LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 383.6 E 335 623.2
ORIGINATED BY	ES
DIST	
HWY	QEW
BOREHOLE TYPE	Hollow Stem Augers/NW Casing/NQ Coring
COMPILED BY	MP
DATUM	Geodetic
DATE	2018.03.18 - 2018.03.19
LATITUDE	43.045788
LONGITUDE	-79.121632
CHECKED BY	GRL

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

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FIELD VANE (sensitivity)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Stiff Reddish Brown to Grey Wet					3.2				
		10	SS	4	163		0	0	51	49
					162	4.0				
		11	SS	4	161					
						3.3				
		12	SS	4	160					
					159	3.5				
		13	SS	5	158		0	4	51	45
						4.0				
	gravelly at 17.7m switch to casing	14	SS	5	157					
156.0 17.8	**SILT** and **SAND**, gravelly, some clay, containing cobbles Dense to Very Dense Grey Wet (TILL)				156					
		15	SS	48	155		22	35	30	13
		~~16~~	~~SS~~	~~100/~~	154					

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-13

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 383.6 E 335 623.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.18 - 2018.03.19 LATITUDE 43.045788 LONGITUDE -79.121632 CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page			0.050				
					153			
	casing refusal, switch to coring gravel and cobbles (max. 150mm) from 21.4m to 22.1m	17	SS	100/ 0.075		○		
					152		FI	
151.5	rubble zone from 22.1m to 22.3m						>25	
22.3	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey horizontal fracture at 22.4m, 22.5m, 22.6m, 23.0m, 23.1m and 23.3m	1	RUN		151		4, 1, 4, 1	RUN #1 TCR=98% SCR=87% RQD=87% UCS=169.2MPa (average)
	horizontal fracture (50mm) at 23.9m and 24.7m	2	RUN		150		2, 1, 0, 1	RUN #2 TCR=100% SCR=100% RQD=100% UCS=197.7MPa (average)
148.5		3	RUN		149		0, 0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=193.0MPa (average)
25.3	END OF BOREHOLE AT 25.3m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.							

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; GROUND WATER CONDITIONS; STRAT PLOT

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

15 20 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-14

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 394.0 E 335 608.7		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.20 - 2018.03.21	LATITUDE 43.045741 LONGITUDE -79.121851	CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
173.1	GROUND SURFACE										
0.0	**TOPSOIL**: (125mm)				173						
0.1	Silty **CLAY**, some sand, trace rootlets Firm Dark Brown Moist	1	SS	5							
172.5 / 0.6	Silty **CLAY**, trace sand, trace gravel Very Stiff to Stiff Reddish Brown Moist	2	SS	16	172						
		3	SS	17	171						
		4	SS	13	170						
		5	SS	10							
					169	+ 3.3					
	Firm Grey Wet	1	TW	PH	168			0	0	59	41
						+ 3.3					
		6	SS	4	167						
					166	+ 2.7					
		2	TW	PH	165						
						+ 3.2					
		7	SS	5	164						

Continued Next Page

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-14

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 394.0 E 335 608.7		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.20 - 2018.03.21	LATITUDE 43.045741 LONGITUDE -79.121851	CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (Field Vane, sensitivity)	WATER CONTENT	GR	SA	SI	CL
	Silty **CLAY**, trace sand Firm to Stiff Grey Wet				163	+ (3.1)					
		8	SS	5	162		o				
					161	+ (3.3)					
		9	SS	4	160		⊢o⊣	0	0	50	50
						+ (3.7)					
	sandy silt seams	10	SS	5	159		o				
		11	SS	5	158 / 157		o				
	some sand, trace gravel	12	SS	10	156		o	3	17	42	38
155.3 17.8	**SILT** and **SAND**, some clay, some gravel, containing cobbles Very Dense Reddish Brown Wet (TILL)	13	SS	100/ 0.125	155		o				
		14	SS	50/	154		o	17	40	30	13

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-14

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 394.0 E 335 608.7 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.20 - 2018.03.21 LATITUDE 43.045741 LONGITUDE -79.121851 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT (FI)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
	gravel and cobbles (max. 150mm) from 20.1m to 22.3m			0.100	153		
					152		
					151	FI	
150.8 / 22.3	**DOLOSTONE BEDROCK**, slightly weathered, strong to very strong, grey horizontal fracture at 22.4m	1	RUN			>10	RUN #1 TCR=100% SCR=74% RQD=55% UCS=130.0MPa (average)
						0	
					150	0	
		2	RUN			0	RUN #2 TCR=98% SCR=98% RQD=98% UCS=142.8MPa (average)
						0	
	horizontal fracture at 23.9m					1	
					149	2	
	horizontal fracture at 24.2m, 24.4m, 24.7m and 25.1m					1	
		3	RUN			1	RUN #3 TCR=100% SCR=100% RQD=100% UCS=106.3MPa (average)
147.7					148	0	
25.4	END OF BOREHOLE AT 25.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

Column headings (not filled in on this page): STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.24	2.9	170.2

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

20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-15

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 487.9 E 335 641.6 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.22 - 2018.03.23 LATITUDE 43.046585 LONGITUDE -79.121441 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH / WATER CONTENT NOTES	GR	SA	SI	CL
171.4	GROUND SURFACE								
0.0	**TOPSOIL**: (50mm)								
	Silty **CLAY**, sandy to some sand, trace gravel, trace rootlets, Firm, Brown, Wet (FILL)	1	SS	4					
		2	SS	4					
169.9 / 1.5	**PEAT**, roots and rootlets, Soft to Very Soft, Dark Brown, Wet	3	SS	2	256				
169.2 / 2.2	Organic **SILT**, Soft, Dark Brown, Wet	4	SS	1	127				
		5	SS	0	147				
					6.0				
166.7 / 4.7	Silty **CLAY**, trace sand, Very Soft to Firm, Grey, Wet	6	SS	2		0	0	53	47
		1	TW	PH					
					3.2				
		2	TW	PH					
					3.3				
		7	SS	5		0	7	47	46

Elevation scale: 171, 170, 169, 168, 167, 166, 165, 164, 163, 162

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

Continued Next Page

+³, ×³: Numbers refer to Sensitivity; 20 15 5 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-15

METRIC

Ministry of Transportation Ontario

THURBER

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 487.9 E 335 641.6		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.22 - 2018.03.23	LATITUDE 43.046585 LONGITUDE -79.121441	CHECKED BY GRL

SOIL PROFILE — Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Stiff Grey Wet				161	
	cobble (100mm) at 11.0m	8	SS	50/ 0.025	160	
		9	SS	4	159	
157.4					158	
13.9	**SILT** and **SAND**, some clay, trace gravel Compact to Very Dense Reddish Brown Moist (TILL)	10	SS	13	157	
		11	SS	64	156	10 35 36 19
154.6					155	
16.8	Silty **SAND**, with gravel, trace clay, containing cobbles Compact to Very Dense Grey Wet	12	SS	50/ 0.100	154	
	casing refusal, switch to coring cobble (75mm) at 17.7m	13	SS	29	153	
	gravel and cobbles (max. 150mm) from 19.8m to 20.3m	~~14~~	~~SS~~	~~100/~~	152	

Field vane shear strength plot: 4.9

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m3

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-15

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 487.9 E 335 641.6 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.22 - 2018.03.23 | LATITUDE 43.046585 | LONGITUDE -79.121441 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Continued From Previous Page			0.025			
151.0	rubble zone from 20.3m to 20.4m				151	FI	
20.4	**DOLOSTONE BEDROCK**, slightly weathered, strong to very strong, grey horizontal fracture at 20.4m, 20.6m, 20.8m, 21.1m and 21.2m	1	RUN			>10 2 3	RUN #1 TCR=95% SCR=85% RQD=73% UCS=78.4MPa (average)
	horizontal fracture at 21.4m, 21.6m, 21.7m, 21.9m and 22.0m	2	RUN		150	3 >5 3	RUN #2 TCR=100% SCR=100% RQD=87% UCS=222.3MPa (average)
147.7	horizontal fracture at 23.5m	3	RUN		149 148	0 0 0 0 1	RUN #3 TCR=100% SCR=100% RQD=100% UCS=160.1MPa (average)
23.7	END OF BOREHOLE AT 23.7m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.24	0.1	171.3

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60
GR SA SI CL

+³, ×³: Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-16

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 490.0 E 335 622.7 | ORIGINATED BY ES

DIST ______ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.23 - 2018.03.24 | LATITUDE 43.046605 | LONGITUDE -79.121673 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
171.5	GROUND SURFACE					
0.0 / 0.1	**TOPSOIL**: (75mm)					
	Silty **CLAY**, with sand, trace roots, occasional wood fibres, Firm to Soft, Brown, Moist (FILL)	1	SS	5	171	
		2	SS	17		0 38 35 27
170.1 / 1.4	Organic **SILT**, trace roots, Very Soft, Dark Brown, Wet	3	SS	3	170	
		4	SS	2	169	
		5	SS	1	168	
166.9 / 4.6	Silty **CLAY**, trace sand, Firm, Reddish Brown, Wet	6	SS	3	167	
		7	SS	2	166, 165	
		8	SS	4	164	0 6 48 46
		1	TW	PH	163, 162	

Shear strength (field vane) annotations: 6.0, 4.0, 3.3. Water content annotations: 125, 127.

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-16

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 490.0 E 335 622.7 | ORIGINATED BY ES

DIST ______ HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.23 - 2018.03.24 | LATITUDE 43.046605 | LONGITUDE -79.121673 | CHECKED BY GRL

Continued From Previous Page

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
	Silty **CLAY**, trace sand Firm to Stiff Reddish Brown Wet
158.2 / 13.3	**SILT** and **SAND**, clayey, trace gravel, containing cobbles Comapct to Very Dense Reddish Brown Moist (TILL) gravelly zone with cobbles casing refusal, switch to coring
153.8 / 17.7	Sandy **SILT**, trace clay, trace gravel Very Dense Reddish Brown Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
9	SS	7	
10	SS	8	
11	SS	21	10 40 28 22
12	SS	34	
13	SS	50/ 0.125	
14	SS	68	
15	SS	50/	

Field vane shear strength values: 3.0 (near elev. 161.5), 3.5 (near elev. 159.8)

ELEVATION SCALE: 161, 160, 159, 158, 157, 156, 155, 154, 153, 152

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-16

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 490.0 E 335 622.7 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.03.23 - 2018.03.24 LATITUDE 43.046605 LONGITUDE -79.121673 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT FI	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Continued From Previous Page			0.100			
150.9					151	FI	
20.7	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey horizontal fracture at 21.0m sub vertical fracture (100mm) at 21.1m	1	RUN			>5 1 2	RUN #1 TCR=100% SCR=88% RQD=70% UCS=149.2MPa (average)
		2	RUN		150	0 0 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=176.1MPa (average)
	quartz interbed at 23.3m horizontal fracture at 23.4m	3	RUN		149 148	0 0 0 1 0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=191.9MPa (average)
147.5 / 24.0	END OF BOREHOLE AT 24.0m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL

+³, ×³: Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-17

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 483.5 E 335 603.1		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.03.24 - 2018.03.25	LATITUDE 43.046547 LONGITUDE -79.121914	CHECKED BY GRL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
171.3	GROUND SURFACE
0.0	**TOPSOIL**: (50mm)
0.0	Silty **CLAY**, sandy to some sand, trace gravel, trace roots Firm Dark Brown Wet
169.9 / 1.4	Organic **SILT**, trace roots, occasional wood fibres Very Soft to Soft Dark Brown Moist to Wet
167.2 / 4.1	Silty **CLAY**, trace sand Firm Reddish Brown to Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	6
2	SS	2
3	SS	3
4	SS	1
5	SS	0
6	SS	2
7	SS	2
1	TW	PH
8	SS	3

ELEVATION SCALE: 171, 170, 169, 168, 167, 166, 165, 164, 163, 162

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 3.5 (approx. elev. 165.7), 3.0 (approx. elev. 162.6)

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

Water contents plotted off scale: 82, 120, 139

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
7 SS	0	5	53	42
8 SS	0	7	47	46

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-17

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 483.5 E 335 603.1		ORIGINATED BY ES
DIST	HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring	COMPILED BY MP
DATUM Geodetic	DATE 2018.03.24 - 2018.03.25	LATITUDE 43.046547 LONGITUDE -79.121914	CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Firm Reddish Brown to Grey Wet				161	+ 3.1 (field vane)					
		9	SS	6	160			0	0	58	42
158.9 / 12.4	**SILT**, clayey, some sand, trace gravel to gravelly, containing cobbles Very Dense Reddish Brown Moist (TILL) casing grinding at 12.8m	10	SS	52	159						
	cobble (150mm) at 14.0m	11	SS	50/ 0.125	158, 157						
		12	SS	60	156, 155			10	11	54	25
154.5 / 16.8	Silty **SAND**, trace clay Dense to Very Dense Reddish Brown Wet	13	SS	41	154						
		14	SS	83	153, 152			0	73	22	5
151.3		15	SS	100/							

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-17

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 483.5 E 335 603.1 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.03.24 - 2018.03.25 | LATITUDE 43.046547 | LONGITUDE -79.121914 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT (kN/m³) / FI	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Continued From Previous Page			0.075			
20.0	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey; rubble zone from 20.0m to 20.1m; horizontal fracture at 20.3m, 20.4m, 20.6m, 20.9m and 21.1m	1	RUN		151	>20, 2, 1, 2	RUN #1 TCR=87% SCR=79% RQD=69% UCS=145.2MPa (average)
	sub vertical fracture (25mm) at 21.2m; horizontal fracture at 21.3m, 21.7m, 21.9m, 22.0m and 22.1m	2	RUN		150, 149	3, 2, 4, 0	RUN #2 TCR=100% SCR=100% RQD=87% UCS=229.4MPa (average)
147.9		3	RUN		148	0, 0, 0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=187.6MPa (average)
23.4	END OF BOREHOLE AT 23.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.05.24	0.2	171.1

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; GR SA SI CL

+ ³, × ³: Numbers refer to Sensitivity; 15-20-5-10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-18

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 538.9 E 335 628.4 | ORIGINATED BY ES

DIST ______ HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.12 - 2018.04.12 | LATITUDE 43.047048 | LONGITUDE -79.121526 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT	GR	SA	SI	CL
180.1	GROUND SURFACE										
179.8 / 0.0	**ASPHALT**				180						
0.3	**SAND** and **GRAVEL**, Compact to Dense, Grey, Moist (FILL)	1	SS	27			○				
		2	SS	35	179		○				
178.5 / 1.5	Silty **CLAY**, trace sand, trace gravel, Very Stiff to Stiff, Brown to Reddish Brown, Moist (FILL)	3	SS	16			○				
					178						
		4	SS	19			○				
					177						
		5	SS	14			○				
					176						
		6	SS	11			○				
					175						
						+ 3.4					
					174						
		7	SS	10			○				
					173	+ 3.3					
		8	SS	10			○				
					172						
						+ 3.2					
170.9					171						
9.1 / 170.7	**ORGANICS**, trace rootlets, Dark Brown, Moist						○				
9.3	Silty **CLAY**, trace sand, Firm to Soft	9	SS	10			○	0	0	43	57

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-18

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 538.9 E 335 628.4		ORIGINATED BY ES
DIST	HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring	COMPILED BY MP
DATUM Geodetic	DATE 2018.04.12 - 2018.04.12	LATITUDE 43.047048 LONGITUDE -79.121526	CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Reddish Brown to Grey Wet	10	SS	6				
		1	TW	PH				
		11	SS	1				
		12	SS	0	0	5	52	43
		2	TW	PH				
163.0 17.0	**SILT**, trace sand Compact Reddish Brown Wet							
161.6 18.5	Silty **CLAY**, trace sand Very Stiff Reddish Brown Moist	13	SS	18	0	9	44	47
160.6 19.4	**SILT** and **SAND**, some clay, some gravel, trace gravel Very Dense Reddish Brown							

Continued Next Page

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (170, 169, 168, 167, 166, 165, 164, 163, 162, 161); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

Field vane sensitivity values plotted: 3.0, 2.7, 10.0, 7.0

+ 3, × 3 : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-18

METRIC

GWP# 2430-15-00		LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 538.9 E 335 628.4	ORIGINATED BY	ES
DIST	HWY QEW	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE 2018.04.12 - 2018.04.12	LATITUDE 43.047048 LONGITUDE -79.121526	CHECKED BY	GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Wet (TILL)	14	SS	50/ 0.100	160	o		
					159			
					158			
		15	SS	106	157	o		9 31 39 21
156.7 / 23.3	**SILT** and **SAND**, trace clay, containing cobbles Very Dense Reddish Brown Wet gravel and cobbles (max. 100mm) from 23.3m to 25.9m				156			
					155			
		16	SS	100/ 0.225	154	o		0 54 42 4
					153			
					152			
151.0		17	SS	100/ 0.075	151		FI 4	
29.1	**DOLOSTONE BEDROCK**, slightly weathered, very strong, grey Clay seam (25mm) at 29.1m and 29.3m	1	RUN				1 2	RUN #1 TCR=100% SCR=98% RQD=98% UCS=135.1MPa (average)

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DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-18

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 538.9 E 335 628.4 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.12 - 2018.04.12 LATITUDE 43.047048 LONGITUDE -79.121526 CHECKED BY GRL

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION (Continued From Previous Page)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Horizontal fracture (25mm) at 29.2m, 29.3m, 29.4m, 29.7mm 29.9m and 30.0m						150	1	
	Sub-horizontal fracture (25mm) at 30.5m and 30.9m		2	RUN				1 4	RUN #2 TCR=97% SCR=97% RQD=83% UCS=172.9MPa (average)
	Horizontal fracture (25mm) at 30.7m, 30.9m, 31.1m and 31.3m						149	2 1	
			3	RUN			148	3 1	RUN #3 TCR=100% SCR=100% RQD=100% UCS=141.9MPa (average)
147.6 / 32.4	Horizontal fracture (25mm) at 32.0m							0	
	END OF BOREHOLE AT 32.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.								

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)

+³, ×³: Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-19

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 537.5 E 335 611.4 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.18 - 2018.04.18 | LATITUDE 43.047036 | LONGITUDE -79.121755 | CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.1	GROUND SURFACE				
0.0	**ASPHALT**				
0.2	**SAND** and **GRAVEL** Dense Grey Moist (FILL)	1	SS	41	
179.2 / 0.9	Silty **SAND**, trace clay Compact Reddish Brown Moist (FILL)	2	SS	29	
178.6 / 1.5	Silty **CLAY**, trace sand, trace gravel Very Stiff to Stiff Reddish Brown Moist (FILL)	3	SS	17	
		4	SS	13	
		5	SS	14	
		6	SS	18	
		7	SS	14	
	Wet	8	SS	8	0 0 46 54
171.4 / 8.7	Silty **CLAY**, trace sand, trace rootlets Firm Reddish Brown Wet	1	TW	PH	

Field vane: 3.5 (at approx. elev. 171.4)

Elevation scale: 180, 179, 178, 177, 176, 175, 174, 173, 172, 171

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-19

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 537.5 E 335 611.4		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.18 - 2018.04.18	LATITUDE 43.047036 LONGITUDE -79.121755	CHECKED BY GRL

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

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION
	Silty **CLAY**, trace sand, trace rootlets Firm Reddish Brown Wet
	trace silt seams
	Stiff
160.9 19.2	**SILT** and **SAND**, some clay, some gravel, containing cobbles Dense Reddish Brown Wet

NUMBER	TYPE	"N" VALUES
9	SS	5
10	SS	3
2	TW	PH
11	SS	8
12	TW	PH
13	SS	10

Elevation scale: 170, 169, 168, 167, 166, 165, 164, 163, 162, 161

Field vane sensitivity values plotted: 4.7, 3.2, 4.5, 2.7, 2.8

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-19

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 537.5 E 335 611.4 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.18 - 2018.04.18 | LATITUDE 43.047036 | LONGITUDE -79.121755 | CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT	GR	SA	SI	CL
	(TILL) casing refusal, switch to coring boulder (350mm) at 19.6m				160						
					159						
		14	SS	33		o		11	36	34	19
					158						
156.9 / 23.2	Silty **SAND**, trace clay, Very Dense, Reddish Brown, Wet				157						
					156						
		15	SS	100/ 0.275		o		0	70	25	5
					155						
					154						
					153						
		16	SS	100/ 0.250		o					
	gravel and cobbles (max. 125mm) from 27.7m to 29.1m				152						
151.0 / 29.1	**DOLOSTONE BEDROCK** slightly weathered, very strong, grey; horizontal fracture at 29.2m, 29.4m, 29.6m, 29.8m, 30.0m and 30.1m	1	RUN		151		FI: 2, 1, 2	RUN #1 TCR=100% SCR=100% RQD=83% UCS=153.1MPa (average)			

Continued Next Page

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%).

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-19

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 537.5 E 335 611.4 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.18 - 2018.04.18 LATITUDE 43.047036 LONGITUDE -79.121755 CHECKED BY GRL

ELEV DEPTH	DESCRIPTION (Continued From Previous Page)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
						150	1	
	horizontal fracture at 30.4m, 30.5m, 30.7m, 30.9m, 31.4m and 31.6m sub vertical fracture at 30.7m	2	RUN			149	3 6 1 0	RUN #2 TCR=100% SCR=100% RQD=88% UCS=211.5MPa (average)
147.5	horizontal fracture at 31.8m	3	RUN			148	1 1 0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=172.4MPa (average)
32.6	END OF BOREHOLE AT 32.6m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG, CEMENT AND ASPHALT TO SURFACE.						0	

Dynamic cone penetration resistance plot: 20 40 60 80 100. Shear strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100. Plastic limit w_P, natural moisture content w, liquid limit w_L; WATER CONTENT (%) 20 40 60.

+3, ×3: Numbers refer to Sensitivity   20 / 15 5 / 10 (%) STRAIN AT FAILURE

THURBER

RECORD OF BOREHOLE No 18-20

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 536.9 E 335 603.0 | ORIGINATED BY ES

DIST ____ HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.05 - 2018.04.06 | LATITUDE 43.047024 | LONGITUDE -79.121986 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.1	GROUND SURFACE				180	
0.0	**ASPHALT**					
179.7 / 0.1	**CONCRETE**					
0.4	**SAND** and **GRAVEL**, Dense, Brown to Grey, Moist (FILL)	1	GS			
		1	SS	40	179	
178.8 / 1.2						
178.6 / 1.4	Silty **SAND**, trace gravel, Dense, Reddish Brown, Moist (FILL)					
		2	SS	6	178	
	Silty **CLAY**, trace sand, trace gravel, Firm to Very Stiff, Reddish Brown, Moist (FILL)	3	SS	8		
		4	SS	12	177	
					176	
		5	SS	15	175	0 0 48 52
		6	SS	11	174	
					173	
		7	SS	16	172	
					171	
	Wet, roots and rootlets	8	SS	21		
170.1						

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DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

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

(%) STRAIN AT FAILURE

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Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-20

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 536.9 E 335 603.0	ORIGINATED BY	ES
DIST		HWY	QEW		
		BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.04.05 - 2018.04.06	CHECKED BY	GRL
LATITUDE	43.047024	LONGITUDE	-79.121986		

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

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FIELD VANE (sensitivity)	GR	SA	SI	CL
10.0	Silty **CLAY**, trace sand Very Stiff Reddish Brown to Grey Wet				170					
		9	SS	18			0	0	45	55
					169					
					168					
	Firm	10	SS	2			0	0	35	65
					167	3.3				
		11	SS	3	166					
						3.2				
					165					
		1	TW	PH						
					164	5.8				
		12	SS	2	163		0	8	42	50
	Stiff					5.5				
					162					
		13	SS	13						
					161					
							0	7	50	43

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-20

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 536.9 E 335 603.0 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.05 - 2018.04.06 LATITUDE 43.047024 LONGITUDE -79.121986 CHECKED BY GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Hard Reddish Brown Moist	14	SS	55/ 0.250	160			
159.1 21.0	**SILT** and **SAND**, some clay, trace gravel, occasional cobbles Very Dense Reddish Brown Wet (TILL)				159			
					158			
		15	SS	55	157			8 37 35 20
					156			
					155			
		16	SS	100/ 0.225	154			
					153			
					152			
	spoon refusal, switch to coring gravel and cobbles from 29.0m to 29.2m	~~17~~	~~SS~~	~~100/~~ 0.025	151			
150.9 29.2	**DOLOSTONE BEDROCK**, slightly weathered, strong to very strong, grey clay seam at 29.5m	1	RUN				FI >5	RUN #1 TCR=87% SCR=81% RQD=74% UCS=179.0MPa (average)

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; γ kN/m^3

+ 3, × 3: Numbers refer to Sensitivity; (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-20

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 536.9 E 335 603.0 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.05 - 2018.04.06 | LATITUDE 43.047024 | LONGITUDE -79.121986 | CHECKED BY GRL

ELEV DEPTH	DESCRIPTION (Continued From Previous Page)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	horizontal fracture at 29.5m, 29.6m, 29.8m, 30.0m, 30.3m, 30.4m and 30.6m				150	3	
						2	
	sub vertical fracture (25mm) at 30.5m					1	
	horizontal fracture at 30.9m, 31.2m, 31.5m and 31.9m					4	
	sub vertical fracture (25mm) at 30.9m and 31.0m	2	RUN		149	6	RUN #2 TCR=95% SCR=95% RQD=87% UCS=139.6MPa (average)
						1	
						1	
147.8					148	1	
32.3	END OF BOREHOLE AT 32.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.						

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.07.12	6.6	173.5

Column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; STRAT PLOT; GROUND WATER CONDITIONS.

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-21

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 592.1 E 335 623.9		ORIGINATED BY ES
DIST ____ HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.17 - 2018.04.17	LATITUDE 43.047527 LONGITUDE -79.121586	CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
178.9	GROUND SURFACE							
0.0	**ASPHALT**							
0.2	**SAND** and **GRAVEL** Dense Grey Moist (FILL)	1	SS	42				
		2	SS	47				
177.7 / 1.2	Silty **CLAY**, trace sand, trace gravel Very Stiff Reddish Brown Moist (FILL)	3	SS	28				
		4	SS	29	0	0	44	56
		5	SS	7				
174.6 / 4.3	Silty **CLAY**, trace sand Very Stiff Reddish Brown to Grey Moist	6	SS	21	0	0	42	58
		7	SS	20				
		8	SS	26				
	Wet	1	TW	PH				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (178, 177, 176, 175, 174, 173, 172, 171, 170, 169); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-21

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 592.1 E 335 623.9		ORIGINATED BY ES
DIST	HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring	COMPILED BY MP
DATUM Geodetic	DATE 2018.04.17 - 2018.04.17	LATITUDE 43.047527 LONGITUDE -79.121586	CHECKED BY GRL

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

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION	FIELD VANE (Sensitivity)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Stiff to Firm Reddish Brown to Grey Wet					+ 3.8				
		9	SS	10	168					
					167	+ 3.3				
		10	SS	3			0	9	46	45
					166	+ 4.0				
		2	TW	PH	165					
					164	+ 4.7				
	trace gravel	11	SS	8	163					
	Very Stiff	12	SS	29	162					
161.1 17.8	**SILT** and **SAND**, some clay, trace gravel, occasional cobbles Very Dense Reddish Brown Moist (TILL)				161					
		13	SS	47	160		5	36	40	19
		14	SS	100/	159					

Continued Next Page

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

15–20–5–10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-21

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 592.1 E 335 623.9		ORIGINATED BY ES
DIST HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY MP
DATUM Geodetic	DATE 2018.04.17 - 2018.04.17	LATITUDE 43.047527 LONGITUDE -79.121586	CHECKED BY GRL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	Continued From Previous Page			0.100	
					158
		15	SS	100/ 0.275	
					157
155.8		16	SS	100/ 0.200	156
23.1	END OF BOREHOLE AT 23.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen.				

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2018.07.12	7.0	171.9

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-22 — 1 OF 3 — METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 565.9 E 335 601.8 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.06 - 2018.04.06 | LATITUDE 43.047284 | LONGITUDE -79.121990 | CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.4	GROUND SURFACE					
0.0	**ASPHALT**					
0.2	**SAND**, trace gravel Brown Moist (FILL)		GS			
178.7 / 0.7	**SAND** and **GRAVEL** Dense Brown Moist (FILL)	1	SS	43		
177.9 / 1.5	**SAND**, trace gravel Compact Brown Moist (FILL)	2	SS	24		
177.5 / 1.9	Silty **CLAY**, trace sand, trace gravel Very Stiff Grey Moist	3	SS	16		0 0 49 51
		4	SS	19		
	trace rootlets	5	SS	15	∇	
	Wet occasional wood fibre	6	SS	16		
		7	SS	16		0 0 53 47
170.7 / 8.7	Silty **CLAY**, trace sand Very Stiff Reddish Brown to Grey Wet	8	SS	21		

Elevation scale: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity — 20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-22

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 565.9 E 335 601.8 | ORIGINATED BY ES

DIST ____ HWY QEW | BOREHOLE TYPE NW Casing | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.06 - 2018.04.06 | LATITUDE 43.047284 | LONGITUDE -79.121990 | CHECKED BY GRL

Continued From Previous Page

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
	Silty **CLAY**, trace sand Very Stiff to Stiff Reddish Brown to Grey Wet Firm Hard
160.2 / 19.2	**SILT** and **SAND**, some clay, trace gravel, occasional cobbels Very Dense Reddish Brown Moist
159.5	

SAMPLES AND TEST RESULTS

NUMBER	TYPE	"N" VALUES	Approx. Elevation	Shear Strength (Field Vane, kPa sensitivity)	Water Content	GR	SA	SI	CL
9	SS	17	~168.4		~32%				
10	SS	1	~166.9		~25%				
			~165.9	+ 2.8					
11	SS	2	~165.4		wP–w	0	0	62	38
			~164.6	+ 3.0					
12	SS	11	~163.9		wP–w–wL	0	0	50	50
13	SS	13	~162.4		~15%				
14	SS	100/ 0.175	~161.0		~26%				
15	SS	50/	~159.5		~15%				

Elevation scale: 169, 168, 167, 166, 165, 164, 163, 162, 161, 160

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

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

WATER CONTENT (%): PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L (20, 40, 60)

UNIT WEIGHT γ kN/m^3

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

Continued Next Page

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

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-22

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 565.9 E 335 601.8 | ORIGINATED BY ES

DIST | HWY QEW | BOREHOLE TYPE NW Casing | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.06 - 2018.04.06 | LATITUDE 43.047284 | LONGITUDE -79.121990 | CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
19.9	(TILL) END OF BOREHOLE AT 19.9m. WATER LEVEL AT 5.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, CEMENT TO 0.07m THEN ASPHALT TO SURFACE.				0.075						

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

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-23

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 641.2 E 335 603.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.19 - 2018.04.19 LATITUDE 43.047968 LONGITUDE -79.121829 CHECKED BY GRL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL
177.7	GROUND SURFACE								
0.0	**ASPHALT**								
0.1	**SAND** and **GRAVEL**, some silt, Very Dense, Grey, Moist, (FILL)	1	SS	50/ 0.125	~7				
		2	SS	62	~8	35	48	17 (SI+CL)	
176.3 / 1.4	Silty **CLAY**, some sand, trace rootlets, Stiff to Firm, Dark Brown/Grey, Moist, (FILL)	3	SS	13	~47				
		4	SS	5	~34				
		5	SS	7	~24				
173.6 / 4.1	Silty **CLAY**, trace sand, trace gravel, Hard to Stiff, Reddish Brown, Moist	6	SS	40	~27				
		7	SS	13	~28	0	0	40	60
		8	SS	13	~26				
	Wet	9	SS	8	~30 (wP ~16)	0	4	48	48

Elevation scale: 177, 176, 175, 174, 173, 172, 171, 170, 169, 168

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 18-23

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 641.2 E 335 603.2	ORIGINATED BY	ES
DIST		HWY QEW	BOREHOLE TYPE NW Casing/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE 2018.04.19 - 2018.04.19	LATITUDE 43.047968 LONGITUDE -79.121829	CHECKED BY	GRL

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
	Silty **CLAY**, trace sand Firm Reddish Brown Wet				167				
		1	TW	PH	166				
		10	SS	4	165				
		11	SS	3	164	0	0	62	38
		2	TW	PH	163, 162				
		12	SS	3	161, 160				
159.8 17.8	**SILT** and **SAND**, some clay, some gravel, occasional cobbles Very Dense Grey Wet (TILL)	13	SS	100/ 0.200	159, 158				

Field vane sensitivity values plotted: 2.9, 2.8, 3.0, 2.7, 2.9

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

RECORD OF BOREHOLE No 18-23

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 641.2 E 335 603.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.19 - 2018.04.19 LATITUDE 43.047968 LONGITUDE -79.121829 CHECKED BY GRL

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	Continued From Previous Page				
		14	SS	60	
	Casing refusal, switch to coring gravel and cobbles (max. 100mm) from 20.4m to 22.9m				157
					156
					155
		15	SS	100/ 0.275	
					154
		16	SS	100/ 0.125	
	gravel and cobbles from 24.5m to 25.9m				153
					152
151.6		17	SS	100/ 0.200	
26.1	END OF BOREHOLE AT 26.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.8m, SAND TO 0.2m THEN CEMENT TO SURAFCE.				

Column headings: GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-24

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 139.8 E 335 646.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.21 - 2018.04.21 LATITUDE 43.043458 LONGITUDE -79.121369 CHECKED BY GRL

ELEV / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
181.5	GROUND SURFACE					
0.0	**ASPHALT**					
0.2	**SAND** and **GRAVEL** Very Dense to Compact Grey Moist (FILL)	1	SS	63	181	
		2	SS	12		
180.2 / 1.3	Silty **CLAY**, some sand, trace gravel Very Stiff Reddish Brown Moist (FILL)	3	SS	17	180	
		4	SS	18	179	0 0 44 56
178.5 / 3.0	Stiff	5	SS	12	178	
177.4 / 4.1					177	
		6	SS	15	176	
		7	SS	15	175	
					174	
		8	SS	20		0 0 34 66
					173	
		9	SS	26	172	

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3.

Continued Next Page

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-24

METRIC

GWP# 2430-15-00 | LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 139.8 E 335 646.2 | ORIGINATED BY ES

DIST ____ HWY QEW | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY MP

DATUM Geodetic | DATE 2018.04.21 - 2018.04.21 | LATITUDE 43.043458 | LONGITUDE -79.121369 | CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FIELD VANE (sensitivity)	WATER CONTENT (%)	GR SA SI CL
171.3 / 10.2	Silty **CLAY**, trace sand Very Stiff to Stiff Reddish Brown Moist				171			
		10	SS	15			~20	
					170			
		1	TW	PH	169		~23	
						+ 4.2		
					168			
	Wet	11	SS	12			~24	
					167			
166.7 / 14.8	Firm					+ 3.7		
		12	SS	7	166		~25	0 0 42 58
						+ 3.7		
					165			
		2	TW	PH			~24	
					164			
						+ 3.2		
		13	SS	4	163		~22	
						+ 3.2		
					162			

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DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

DRAFT

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-24

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 139.8 E 335 646.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MP

DATUM Geodetic DATE 2018.04.21 - 2018.04.21 LATITUDE 43.043458 LONGITUDE -79.121369 CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand, trace gravel Firm Reddish Brown Wet	14	SS	6	
158.6 / 22.9	**SILT**, trace clay Dense Reddish Brown Wet	15	SS	33	0 0 91 9
158.1 / 23.4	Silty **CLAY**, trace sand, trace gravel Very Stiff Reddish Brown Wet	16	SS	15	0 0 49 51
	possible cobbles	17	SS	17	
151.5					

Elevation scale: 161, 160, 159, 158, 157, 156, 155, 154, 153, 152

Field vane sensitivity reading at approx. elev. 160.7: 2.6

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-24

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 139.8 E 335 646.2	ORIGINATED BY	ES
DIST	HWY QEW	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.04.21 - 2018.04.21 LATITUDE 43.043458 LONGITUDE -79.121369	CHECKED BY	GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
30.0	**SILT** and **SAND**, some clay, trace gravel, containing cobbles Very Dense Reddish Brown Moist (TILL) casing refusal, switch to coring gravel and cobbles (max. 75mm) from 29.6m to 32.0m					151 150						
		18	SS	58			○ (approx. 25)		5	31	44	20
149.1 / 32.5	END OF BOREHOLE AT 32.5m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.9m, SAND TO 0.3m, CEMENT TO 0.1m, THEN ASPHALT TO SURAFCE.											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

15 ⌖ 5 (20 / 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-25

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 090.2 E 335 660.7 ORIGINATED BY ES/ISP

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY MP

DATUM Geodetic DATE 2018.07.10 - 2018.07.10 LATITUDE 43.043004 LONGITUDE -79.121229 CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.8	GROUND SURFACE						
0.0	**ASPHALT** (225mm)						
0.2	Gravelly **SAND**, trace silt Compact Brown Moist (FILL)	1	GS				
		1	SS	24		180	
179.5 1.3	Silty **CLAY**, trace sand, trace gravel Firm to Very Stiff Brown Moist (FILL)	2	SS	8	∇	179	
		3	SS	6		178	No recovery
		4	SS	7		177	
		5	SS	9		176	
175.2 5.6	Soft					175	
	occasional oxide staining	6	SS	3		174	0 0 33 67
							Field vane: 20.0
173.2 7.6	Silty **CLAY**, trace sand Very Stiff to Firm Reddish Brown Moist	7	SS	25		173	0 0 46 54
						172	
	Wet	8	SS	6		171	

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-25

METRIC

GWP# 2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 090.2 E 335 660.7		ORIGINATED BY	ES/ISP
DIST	HWY QEW	BOREHOLE TYPE	Hollow Stem Augers/NW Casing	COMPILED BY	MP
DATUM Geodetic	DATE 2018.07.10 - 2018.07.10	LATITUDE 43.043004	LONGITUDE -79.121229	CHECKED BY	GRL

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

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Stiff to Firm Reddish Brown Wet					+ 20.0	
					170		
		9	SS	0			
						+ 5.0	
					169		
		1	TW	PH			
					168		
						+ 2.5	
					167		
		2	TW	PH			No recovery
166.3 / 14.5	**SILT** and **SAND**, trace clay Compact to Dense Reddish Brown Wet				166		
		10	SS	27			
					165		
					164		
		11	SS	32			0 40 56 4
					163		
		12	SS	46			
					162		
161.4 / 19.4	Silty **CLAY**, trace sand Stiff to Firm Reddish Brown to Brown Wet				161		

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-25

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 090.2 E 335 660.7	ORIGINATED BY	ES/ISP
DIST	HWY QEW	BOREHOLE TYPE	Hollow Stem Augers/NW Casing	COMPILED BY	MP
DATUM	Geodetic	DATE	2018.07.10 - 2018.07.10 LATITUDE 43.043004 LONGITUDE -79.121229	CHECKED BY	GRL

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand Stiff to Firm Reddish Brown to Brown Wet	13	SS	15		
					160	
		14	SS	5	159	
					158	
		15	SS	6		
					157	
		16	SS	8	156	No recovery
155.5 25.3	**SILT** and **SAND**, some clay, trace gravel Dense Brown Moist (TILL)				155	
		17	SS	48		
154.3 26.5	END OF BOREHOLE AT 26.5m. WATER LEVEL AT 1.8m UPON COMPELTION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.1m, THEN ASPHALT TO SURAFCE.					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane: 2.8

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-26

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 040.3 E 335 665.2		ORIGINATED BY ES
DIST ____ HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing		COMPILED BY MP
DATUM Geodetic	DATE 2018.07.11 - 2018.07.11	LATITUDE 43.042555 LONGITUDE -79.121177	CHECKED BY GRL

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
178.7	GROUND SURFACE							
0.0	**ASPHALT** (200mm)							
0.2	Gravelly **SAND**, trace silt Very Dense Brown Moist (FILL)	1	GS					
		1	SS	50/ 0.125				
176.9 1.8	Silty **CLAY**, trace sand, trace gravel Stiff to Very Stiff Brown to Dark Brown Moist (FILL)	2	SS	13				
		3	SS	15	0	0	28	72
		4	SS	13				
174.6 4.1	Silty **CLAY**, trace sand, trace organics, trace rootlets Stiff to Very Stiff Dark Brown Moist	5	SS	17				
		6	SS	13				
	Firm	1	TW	PH				
		2	TW	PH				

Elevation scale: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169

Field vane: 7.0 (at approx. elev. 170)

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S2 MTO-18426.GPJ 2017TEMPLATE(MTO).GDT 10/2/18

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-26 METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 040.3 E 335 665.2 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY MP

DATUM Geodetic DATE 2018.07.11 - 2018.07.11 LATITUDE 43.042555 LONGITUDE -79.121177 CHECKED BY GRL

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (field vane, sensitivity)	WATER CONTENT (%)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Soft Reddish Brown Wet					+					
		7	SS	0	168		o				
167.0 / 11.7	**SILT**, some sand Reddish Brown Moist				167	+ 3.0					
		3	TW	PH	166		o				
165.4 / 13.3	Silty **CLAY**, trace sand Firm Reddish Brown Wet					+ 3.2					
		4	TW	PH	165		o				
					164	+ 6.0					
	Stiff	8	SS	2	163		o				
					162	+ 10.0					
		9	SS	3			o				
					161	+ 8.0					
160.3 / 18.4	**SILT**, some sand, some clay, trace gravel Dense to Compact Reddish Brown Wet	10	SS	33	160		o	6	11	72	11
					159						

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-26

METRIC

GWP# 2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 040.3 E 335 665.2		ORIGINATED BY	ES
DIST	HWY QEW	BOREHOLE TYPE	Hollow Stem Augers/NW Casing	COMPILED BY	MP
DATUM Geodetic	DATE 2018.07.11 - 2018.07.11	LATITUDE 43.042555	LONGITUDE -79.121177	CHECKED BY	GRL

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100) / PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 20, 40, 60 / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
158.6	Continued From Previous Page	11	SS	18	
20.1	Silty **CLAY**, trace sand, trace gravel Firm to Stiff Reddish Brown Wet				158
		12	SS	5	157
					156
		13	SS	5	155
154.8 / 23.9	**SILT** and **SAND**, gravelly Dense to Very Dense Brown Moist (TILL)	14	SS	48	154
					153
152.7	occasional cobble	15	SS	100/ 0.100	
26.0	END OF BOREHOLE AT 26.0m. WATER LEVEL AT 2.1m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.1m, THEN ASPHALT TO SURAFCE.				

+3, ×3: Numbers refer to Sensitivity

15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-27

METRIC

GWP#	2430-15-00	LOCATION	Welland River Bridge Replacement, MTM NAD83-10: N 4 767 690.4 E 335 601.8	ORIGINATED BY	ES
DIST		HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing	COMPILED BY	MP
DATUM	Geodetic	DATE 2018.07.12 - 2018.07.12	LATITUDE 43.048409 LONGITUDE -79.121919	CHECKED BY	GRL

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) / Shear strength	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
176.9	GROUND SURFACE												
0.0	**ASPHALT**: (200mm)												
0.2	Gravelly **SAND**, trace silt Dense Brown Moist (FILL)		1	GS									
			1	SS	32		176						
175.5 / 1.4	Silty **CLAY**, trace sand, trace gravel, trace organics Stiff to Very Stiff Brown Moist		2	SS	10		175						
			3	SS	13		174						
			4	SS	18		173						
			5	SS	6		172						
			6	SS	9		171 / 170			0	0	37	63
			7	SS	6		169						
							168	Field vane + (sensitivity 2.7)					
			8	SS	30		167						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 18-27

METRIC

GWP# 2430-15-00	LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 690.4 E 335 601.8		ORIGINATED BY ES
DIST ___ HWY QEW	BOREHOLE TYPE Hollow Stem Augers/NW Casing		COMPILED BY MP
DATUM Geodetic	DATE 2018.07.12 - 2018.07.12	LATITUDE 43.048409 LONGITUDE -79.121919	CHECKED BY GRL

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

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH (field vane, sensitivity)	GR	SA	SI	CL
	Silty **CLAY**, trace sand Stiff to Firm Brown Moist					+ (5.1)				
		1	TW	PH	166					
						+ (2.5)				
					165					
		2	TW	PH						
					164	+ (3.3)				
		9	SS	0	163		0	5	52	43
					162	+ (4.0)				
		10	SS	4						
					161	+				
		11	SS	14	160					
158.8					159					
18.1	**SILT** and **SAND**, trace clay, trace gravel, occasional cobbles Very Dense Brown Moist (TILL)	12	SS	78						
					158					
					157					

Continued Next Page

+ 3, × 3 : Numbers refer to Sensitivity

15 ⊕ 5 (20 / 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 18-27

METRIC

GWP# 2430-15-00 LOCATION Welland River Bridge Replacement, MTM NAD83-10: N 4 767 690.4 E 335 601.8 ORIGINATED BY ES

DIST HWY QEW BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY MP

DATUM Geodetic DATE 2018.07.12 - 2018.07.12 LATITUDE 43.048409 LONGITUDE -79.121919 CHECKED BY GRL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
	Continued From Previous Page					
		13	SS	78		
					156	
		14	SS	67		
					155	
					154	
	Dense	15	SS	47		
					153	
	possible cobbles and boulders					No recovery
152.5		16	SS	100/0.025		
24.4	END OF BOREHOLE AT 24.4m. BOREHOLE BACKFILLED WITH CUTTINGS TO 0.1m, THEN ASPHALT TO SURAFCE.					

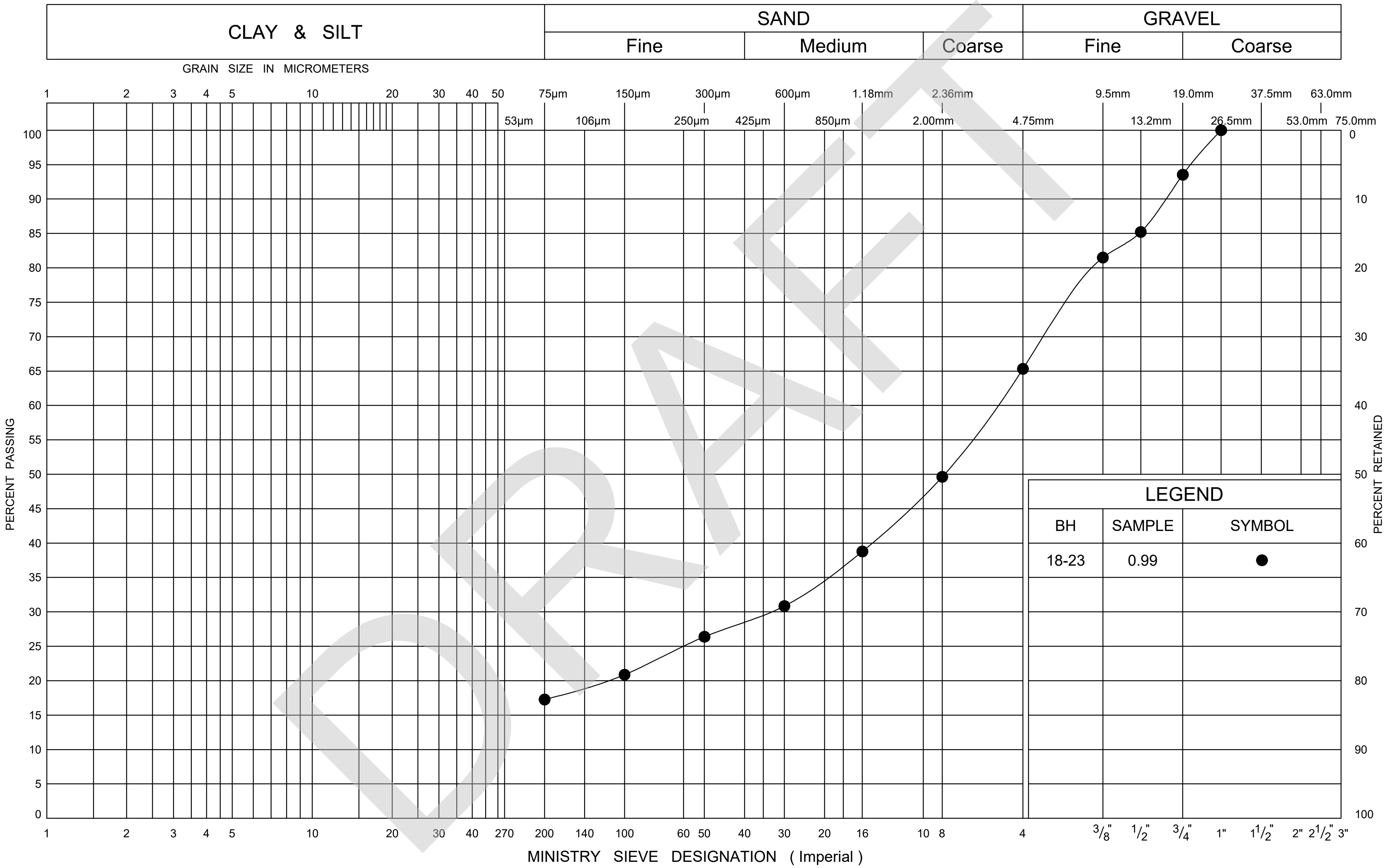
Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL

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

20 / 15 5 / 10 (%) STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results and Cone Penetration Testing – Previous Investigation (2018)



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GRAIN SIZE DISTRIBUTION

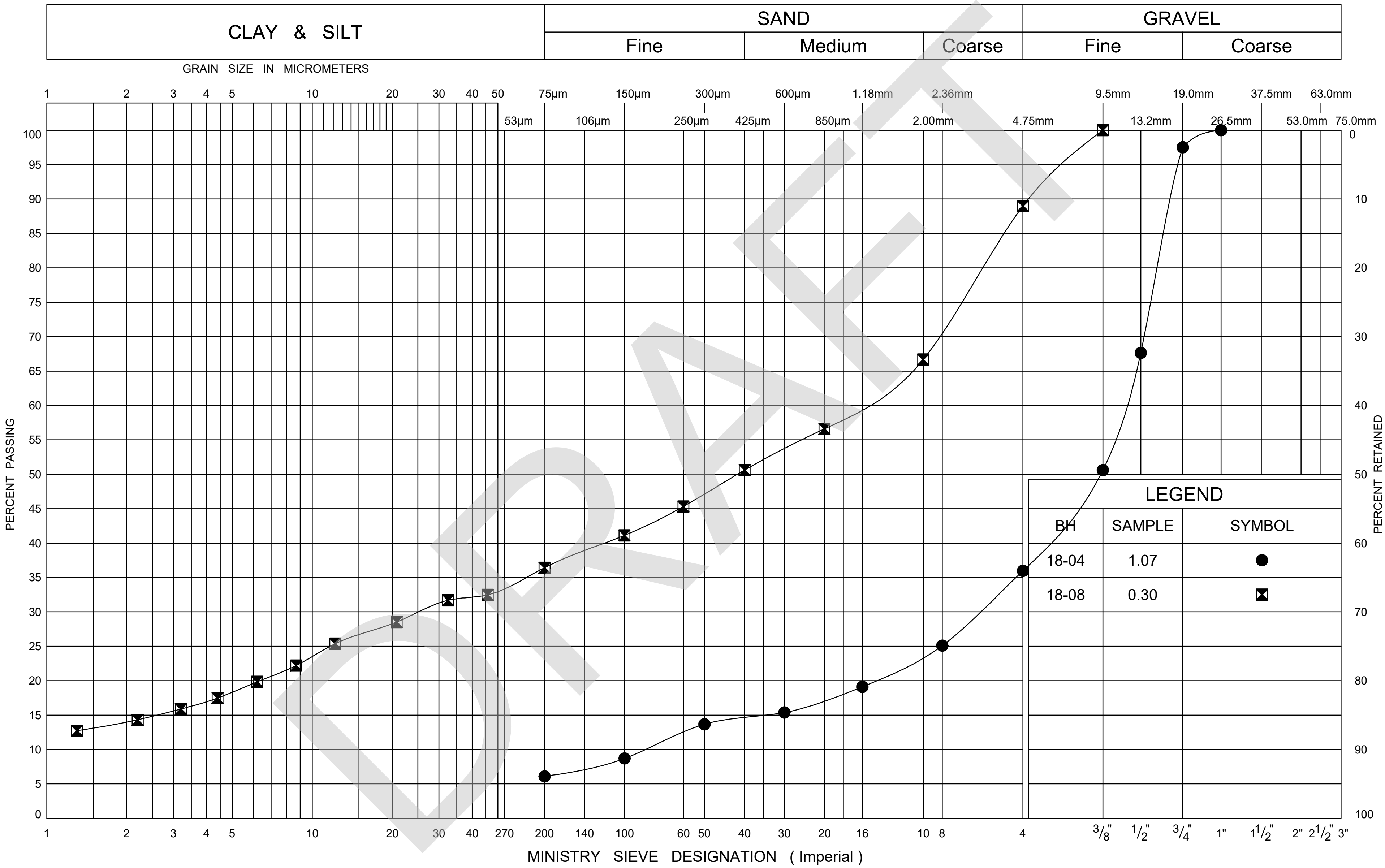
SAND and GRAVEL (Pavement Granulars)

FIG No B1

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/21/18



ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

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GRAIN SIZE DISTRIBUTION

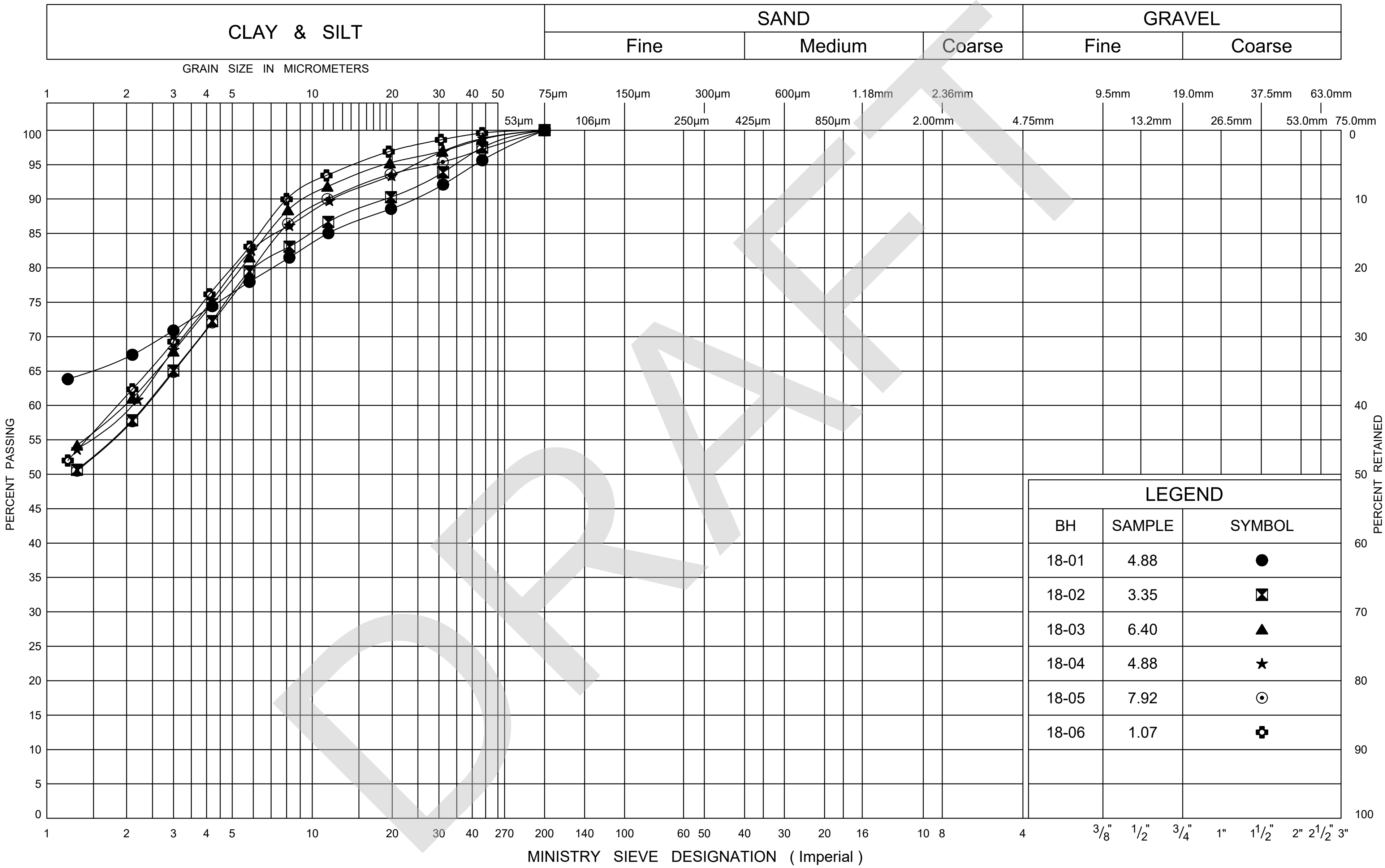
Cohesionless FILL

FIG No B2

GWP 2430-15-00

Welland River Bridge Replacement

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
18-01	4.88	●
18-02	3.35	⧗
18-03	6.40	▲
18-04	4.88	★
18-05	7.92	⊙
18-06	1.07	✿

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Ontario

GRAIN SIZE DISTRIBUTION

Cohesive FILL

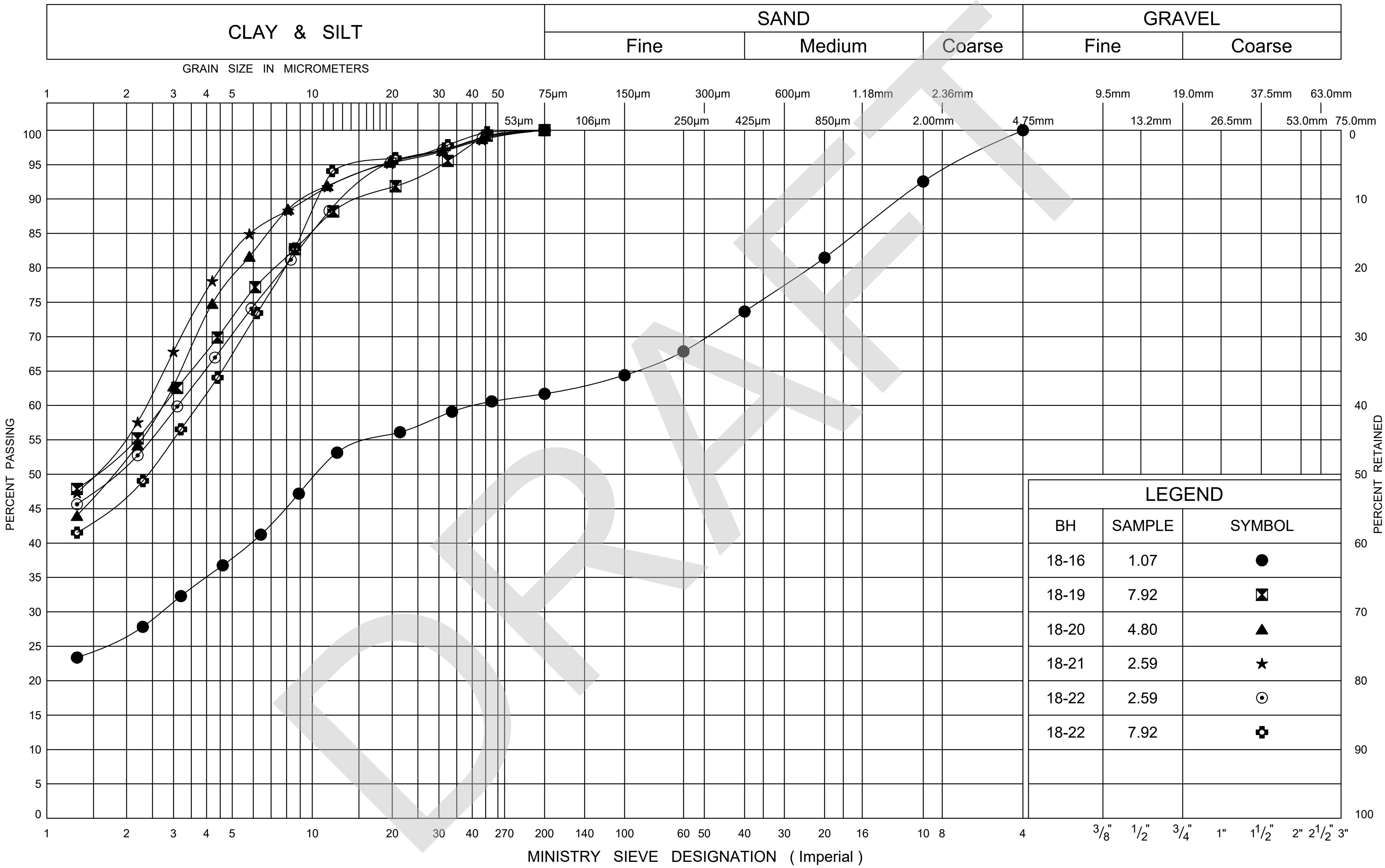
FIG No B3

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



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GRAIN SIZE DISTRIBUTION

Cohesive FILL

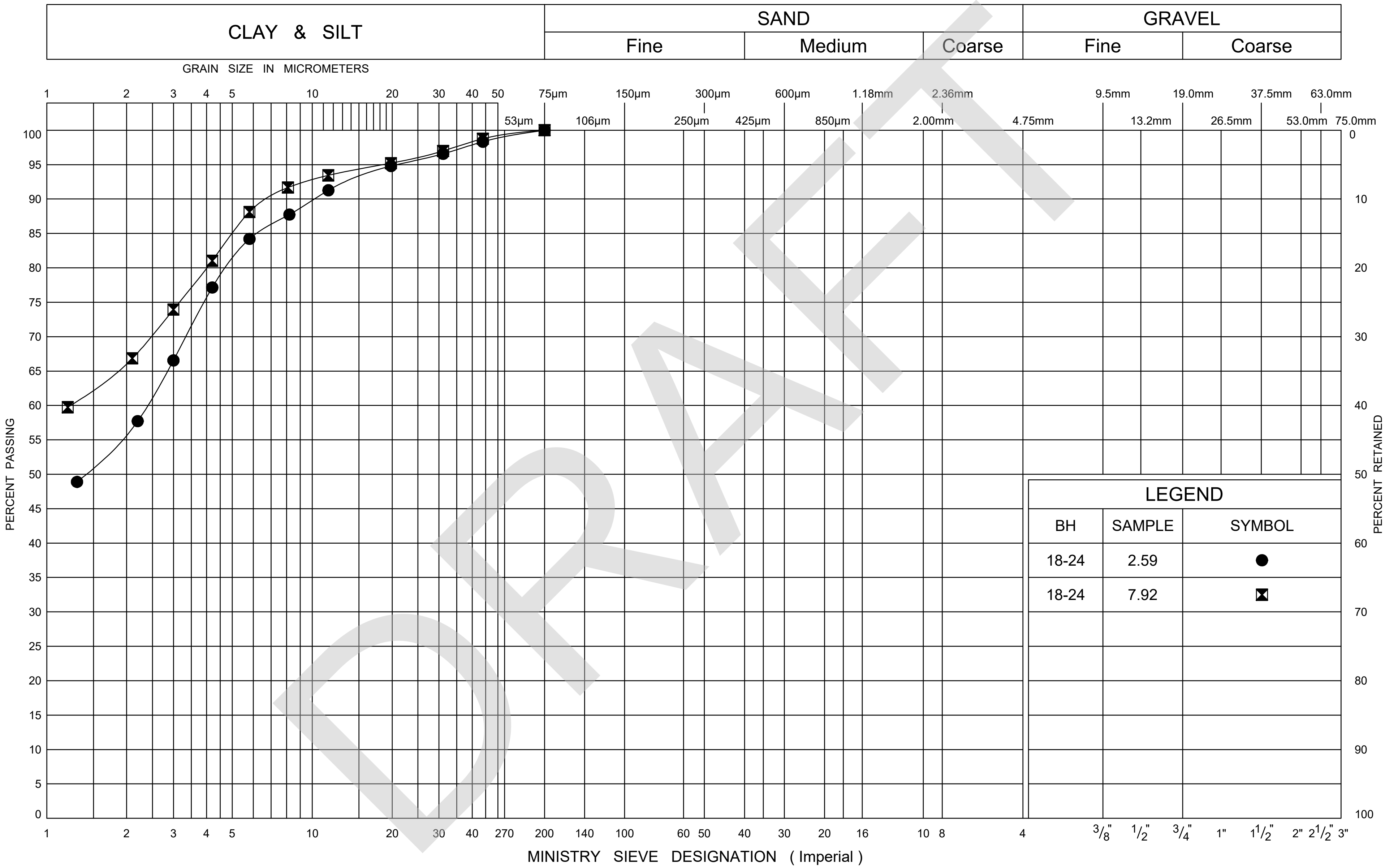
FIG No B4

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



LEGEND		
BH	SAMPLE	SYMBOL
18-24	2.59	●
18-24	7.92	⊠

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GRAIN SIZE DISTRIBUTION

Cohesive FILL

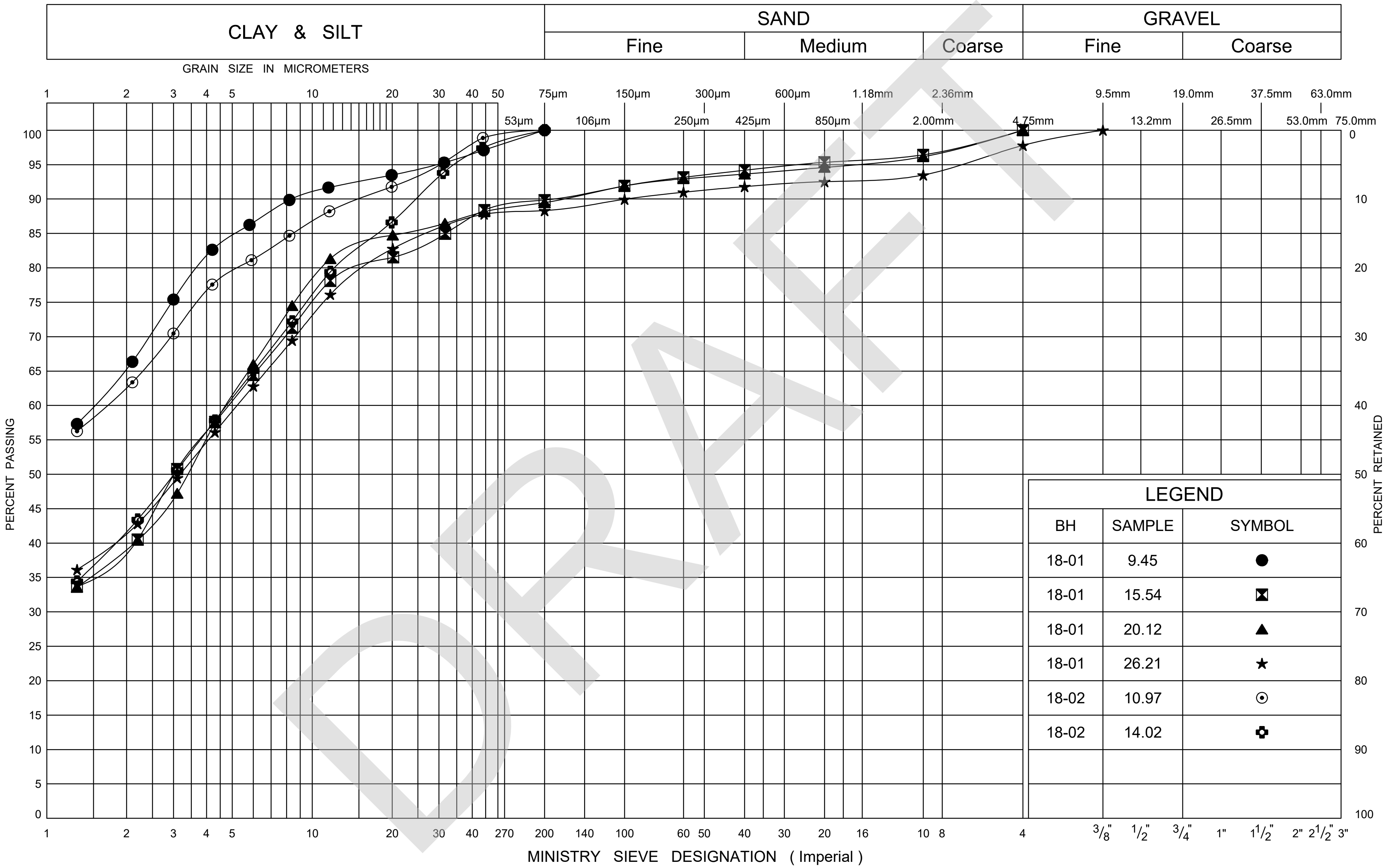
FIG No B5

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



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GRAIN SIZE DISTRIBUTION

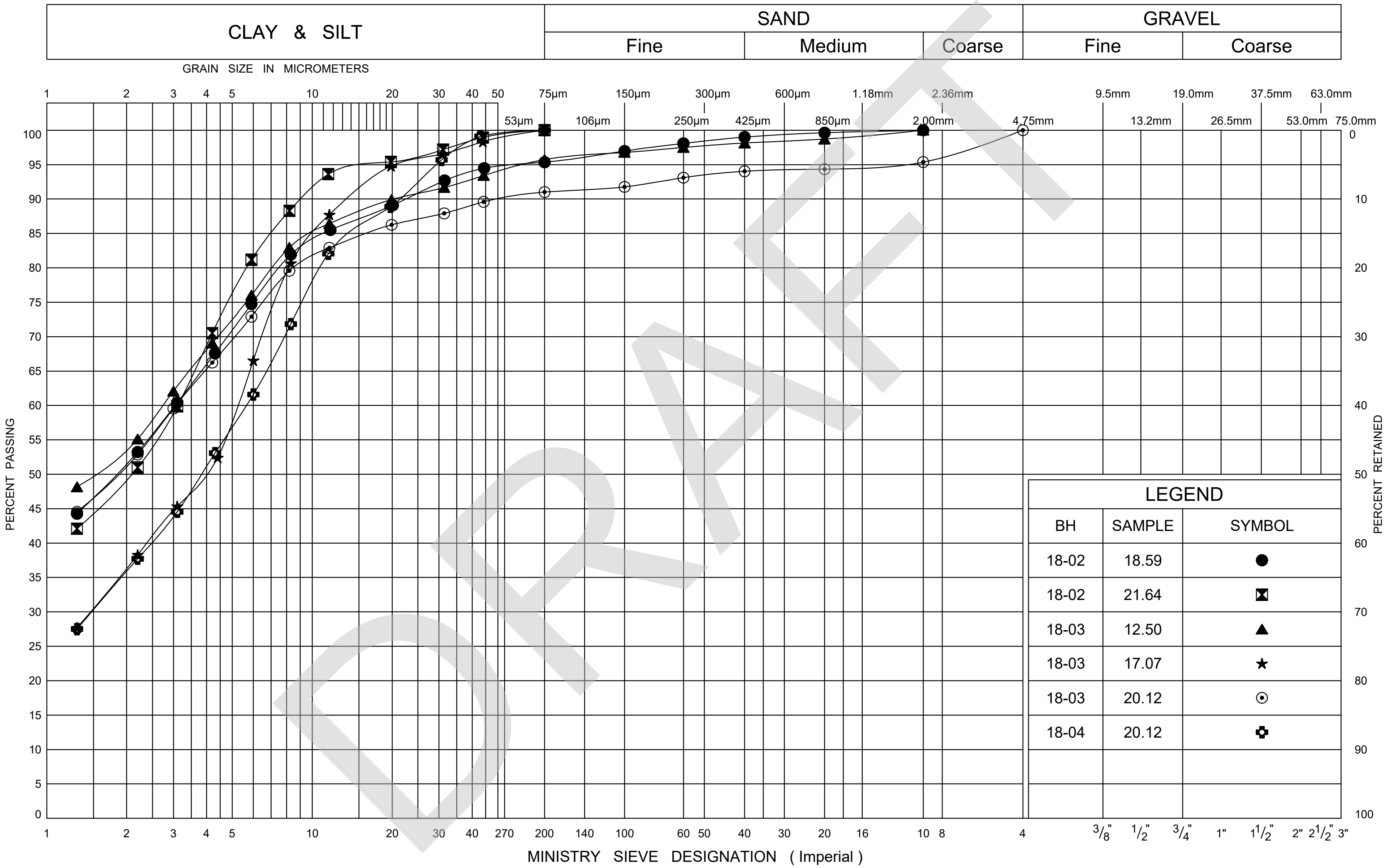
Silty CLAY

FIG No B6

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18



LEGEND

BH	SAMPLE	SYMBOL
18-02	18.59	●
18-02	21.64	⧗
18-03	12.50	▲
18-03	17.07	★
18-03	20.12	⊙
18-04	20.12	✿

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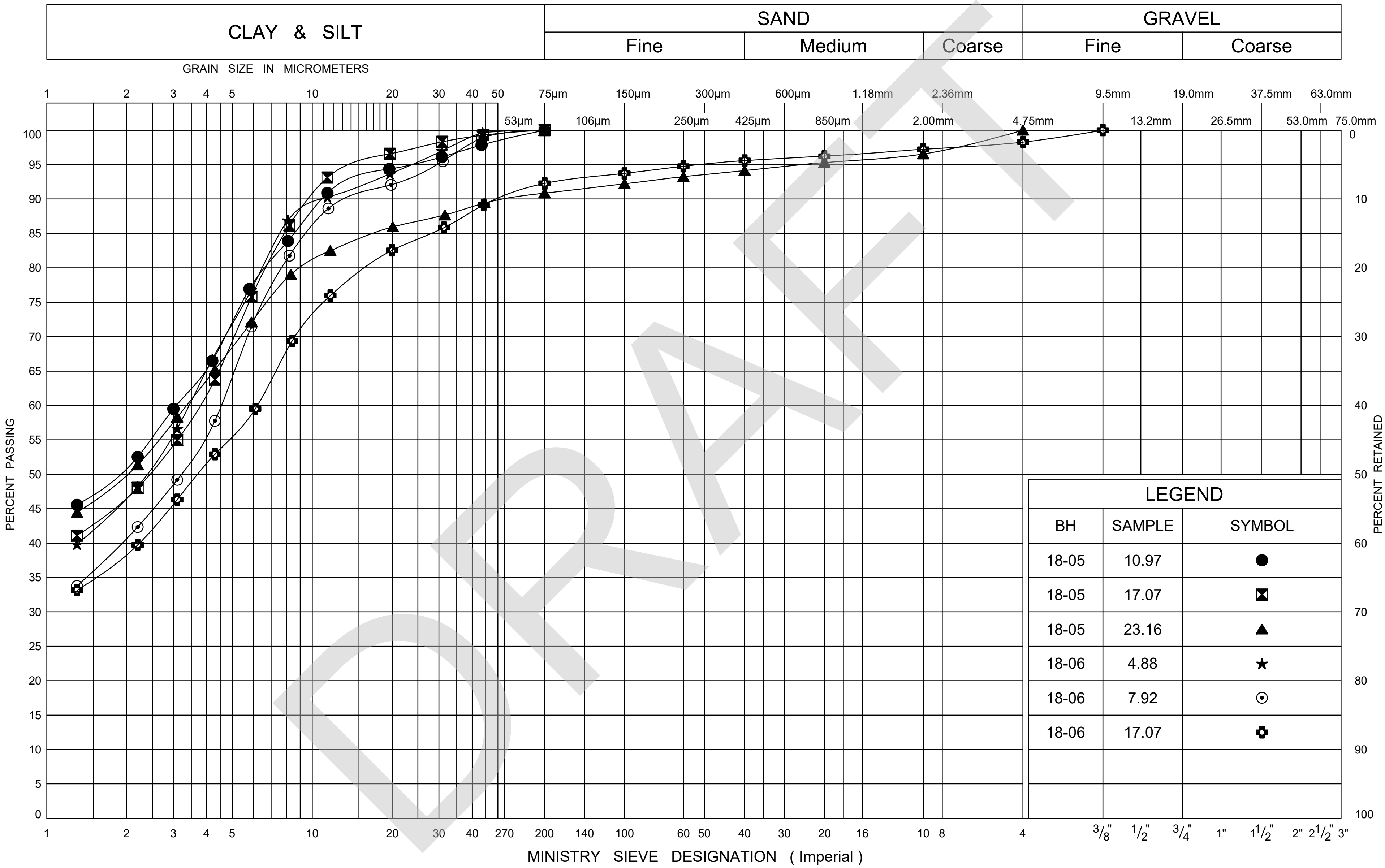
GRAIN SIZE DISTRIBUTION

Silty CLAY

FIG No B7

GWP 2430-15-00

Welland River Bridge Replacement



LEGEND		
BH	SAMPLE	SYMBOL
18-05	10.97	●
18-05	17.07	⊠
18-05	23.16	▲
18-06	4.88	★
18-06	7.92	⊙
18-06	17.07	✤

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GRAIN SIZE DISTRIBUTION

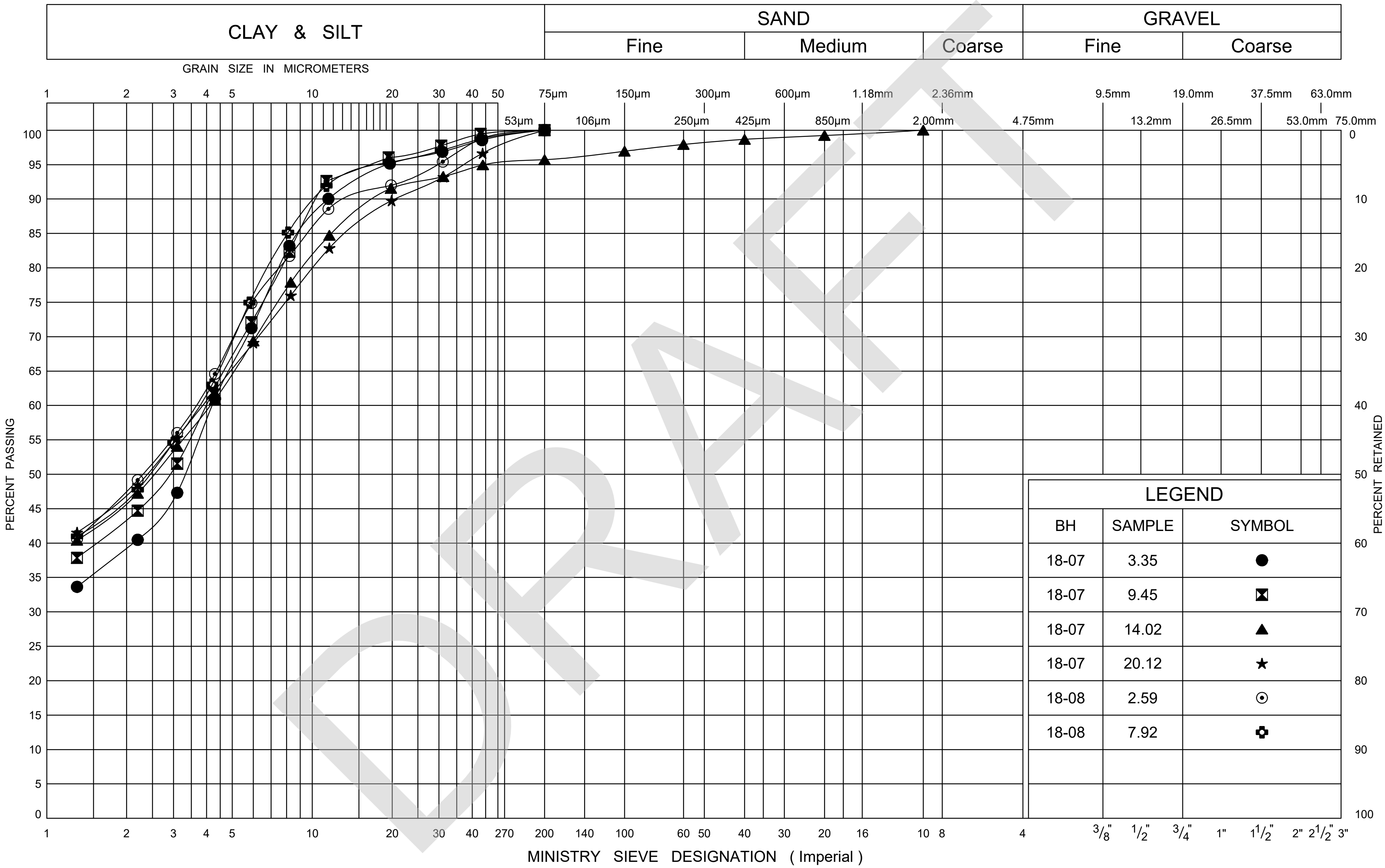
Silty CLAY

FIG No B8

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18



LEGEND		
BH	SAMPLE	SYMBOL
18-07	3.35	●
18-07	9.45	⊠
18-07	14.02	▲
18-07	20.12	★
18-08	2.59	⊙
18-08	7.92	✪

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GRAIN SIZE DISTRIBUTION

Silty CLAY

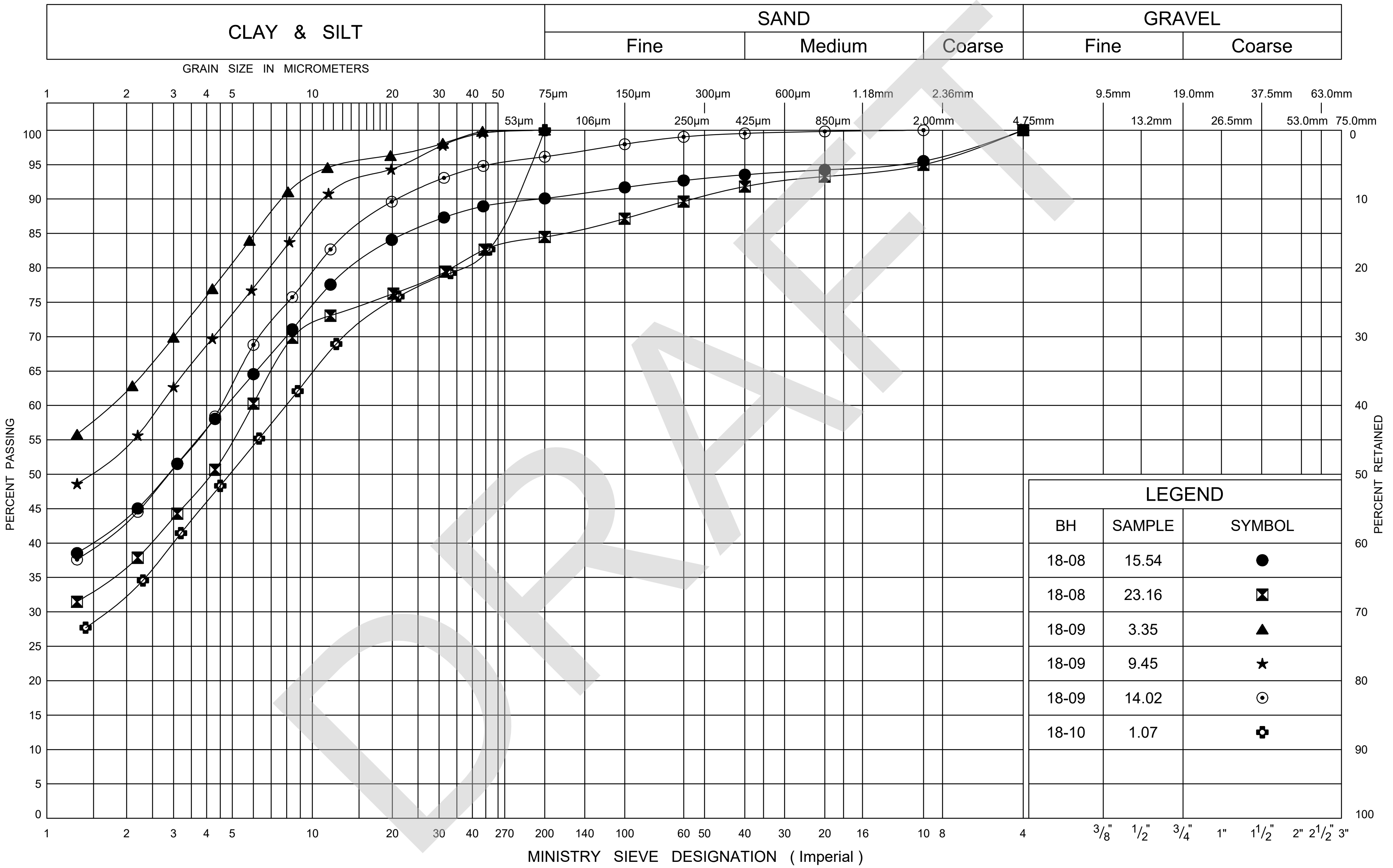
FIG No B9

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



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GRAIN SIZE DISTRIBUTION

Silty CLAY

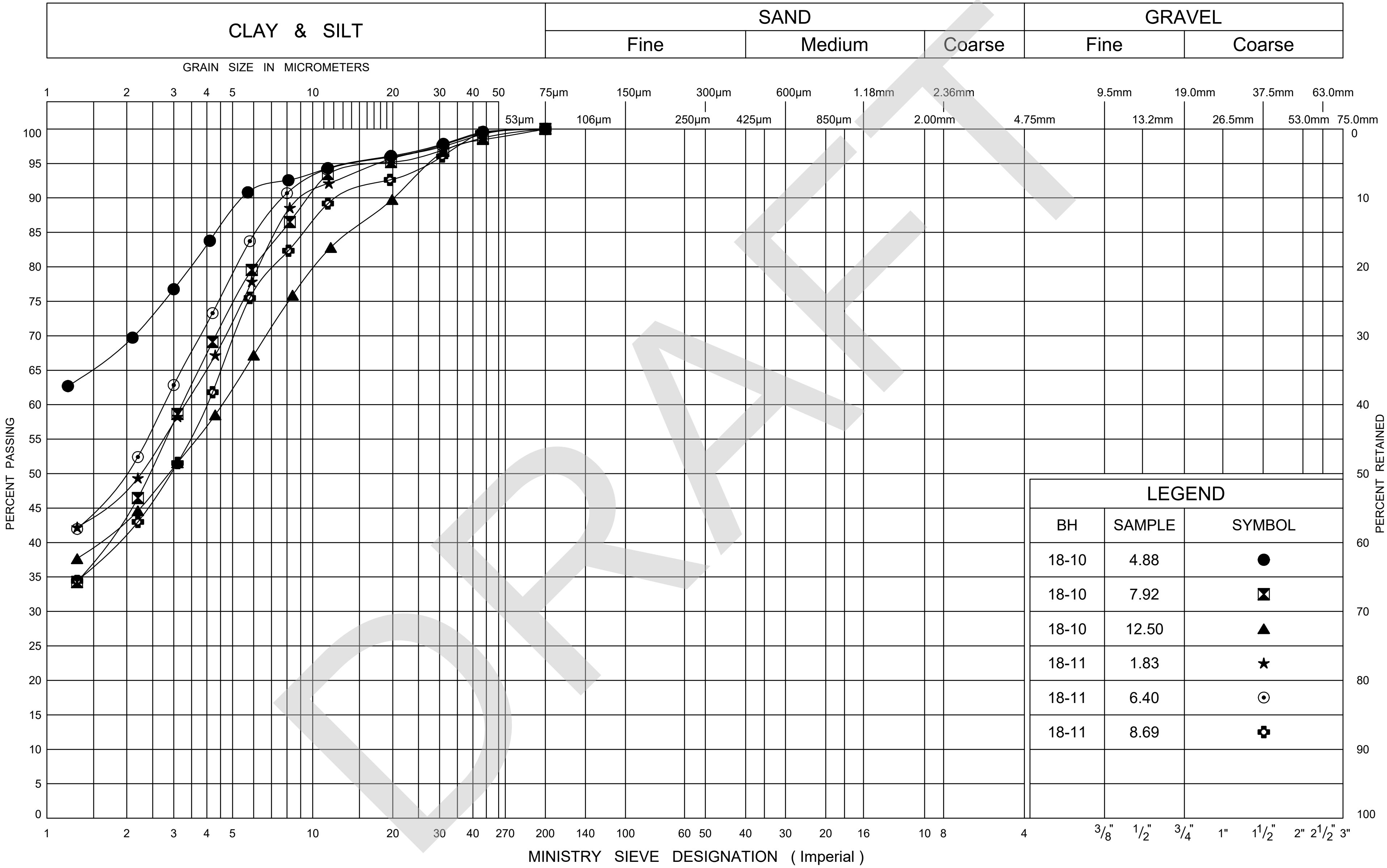
FIG No B10

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
18-10	4.88	●
18-10	7.92	⊠
18-10	12.50	▲
18-11	1.83	★
18-11	6.40	⊙
18-11	8.69	✿

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GRAIN SIZE DISTRIBUTION

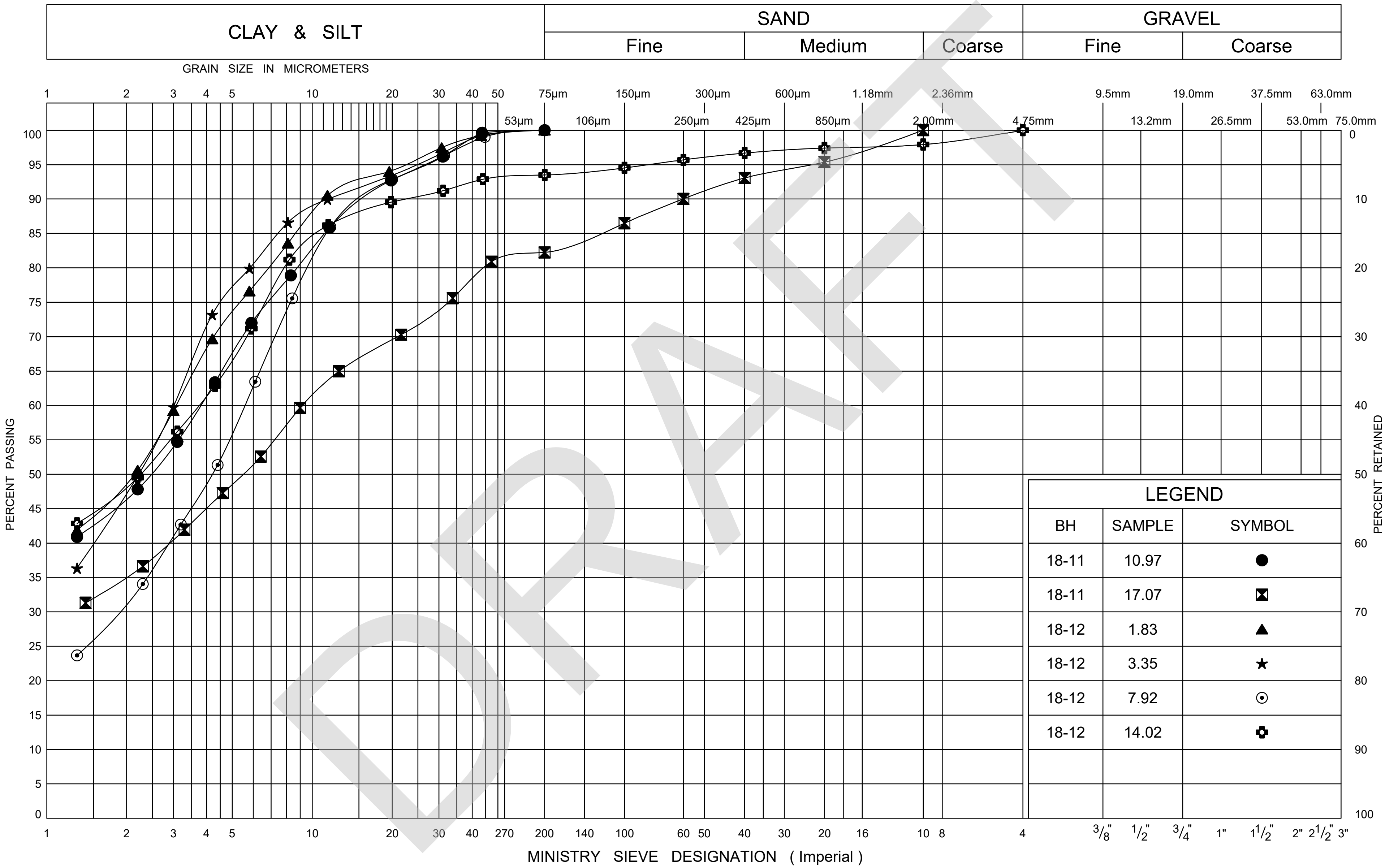
Silty CLAY

FIG No B11

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18



LEGEND		
BH	SAMPLE	SYMBOL
18-11	10.97	●
18-11	17.07	⧗
18-12	1.83	▲
18-12	3.35	★
18-12	7.92	⊙
18-12	14.02	✿

Ministry of Transportation
Ontario

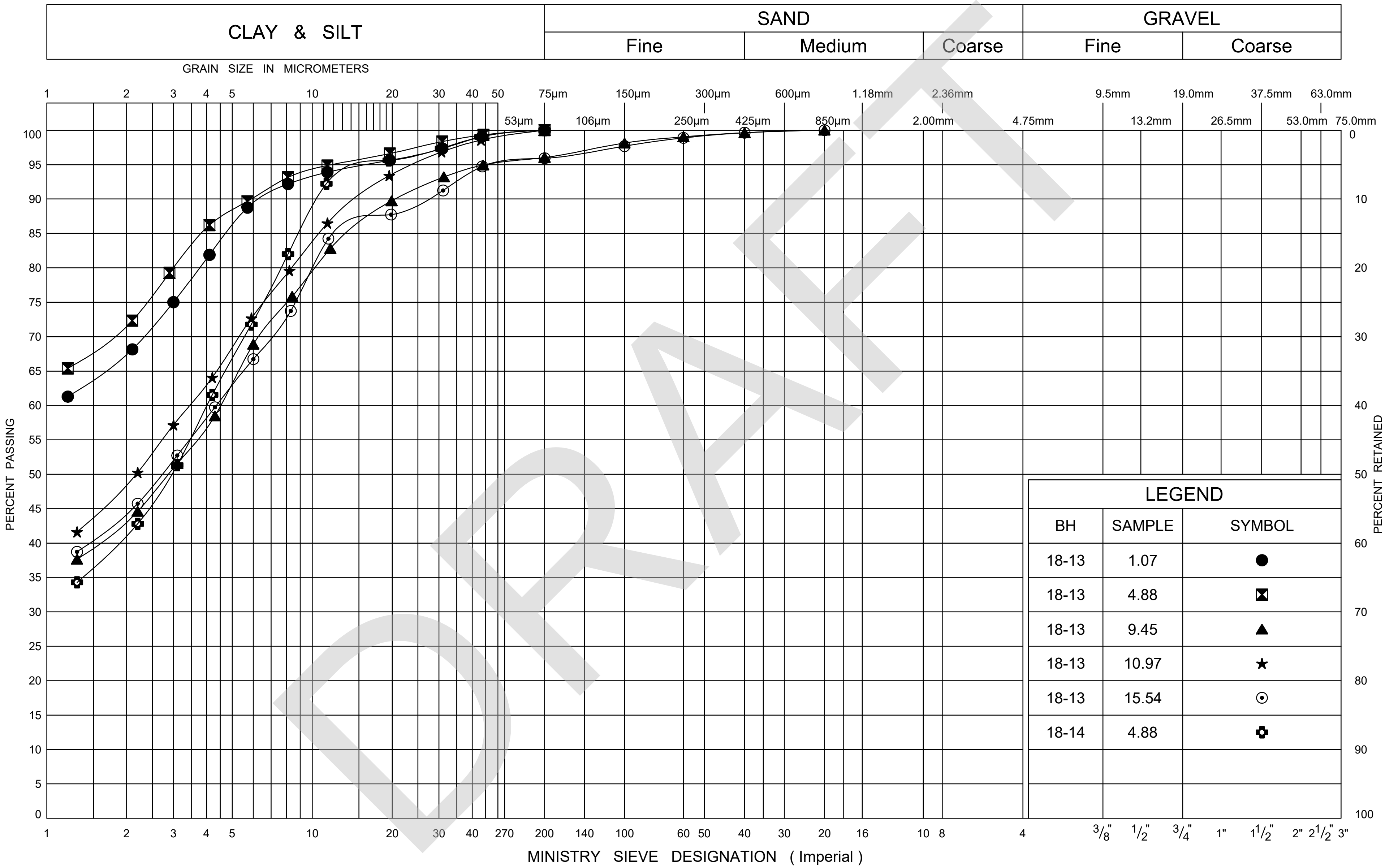
GRAIN SIZE DISTRIBUTION

Silty CLAY

FIG No B12

GWP 2430-15-00

Welland River Bridge Replacement



LEGEND

BH	SAMPLE	SYMBOL
18-13	1.07	●
18-13	4.88	⧗
18-13	9.45	▲
18-13	10.97	★
18-13	15.54	⊙
18-14	4.88	✽

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Ontario

GRAIN SIZE DISTRIBUTION

Silty CLAY

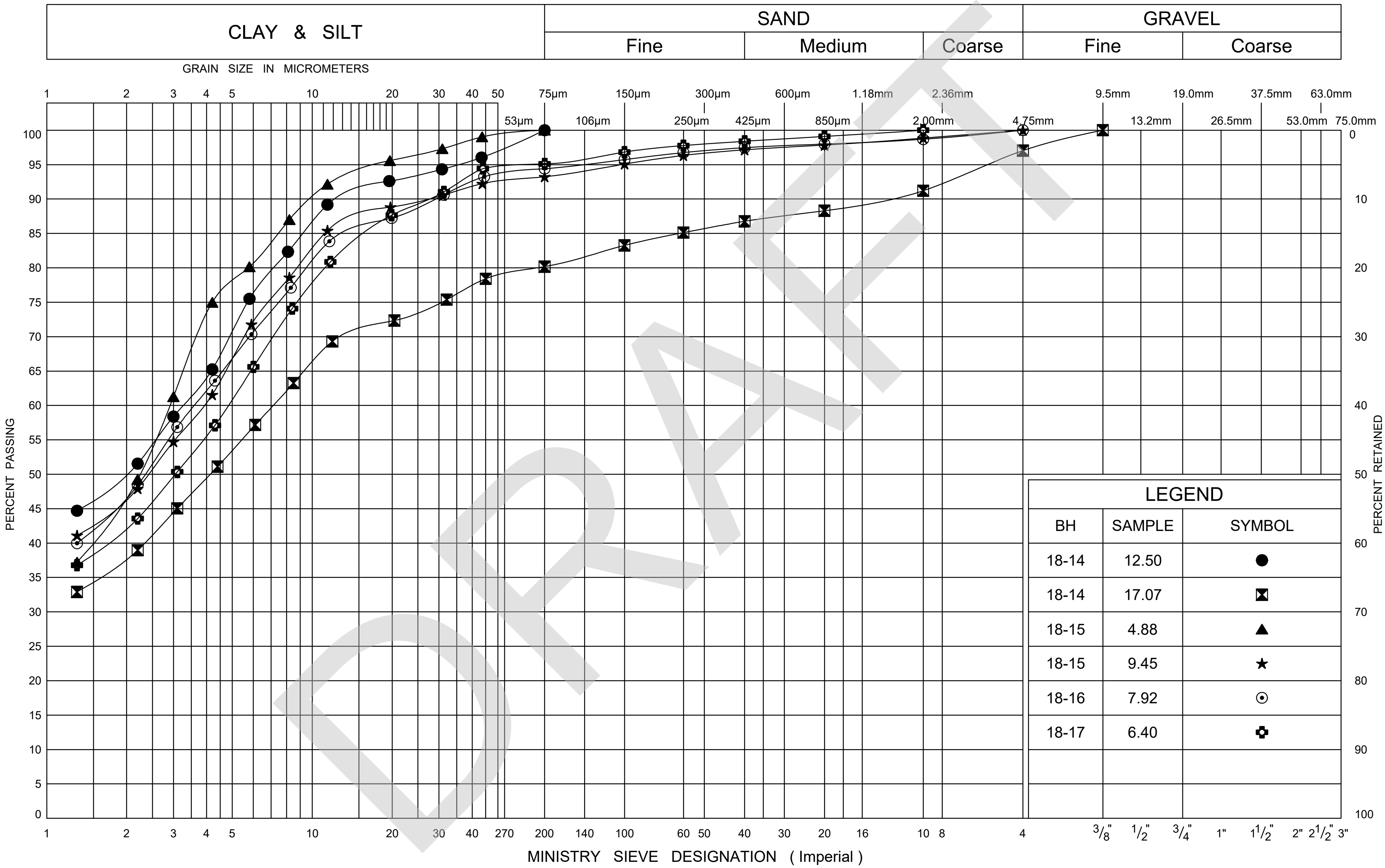
FIG No B13

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

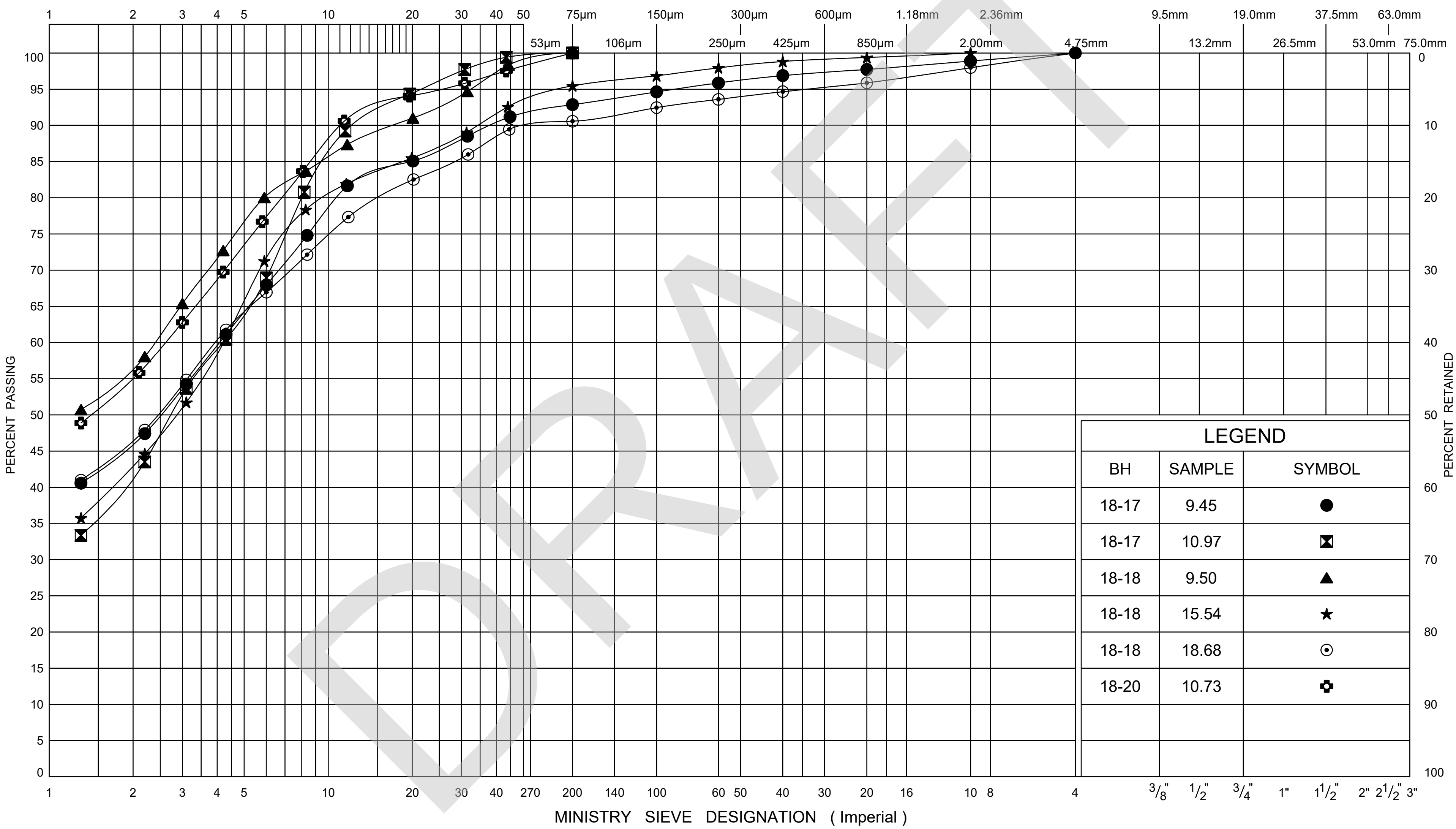
Silty CLAY

FIG No B14

GWP 2430-15-00

Welland River Bridge Replacement

78 12 M



LEGEND		
BH	SAMPLE	SYMBOL
18-17	9.45	●
18-17	10.97	⊠
18-18	9.50	▲
18-18	15.54	★
18-18	18.68	⊙
18-20	10.73	✿

Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

Silty CLAY

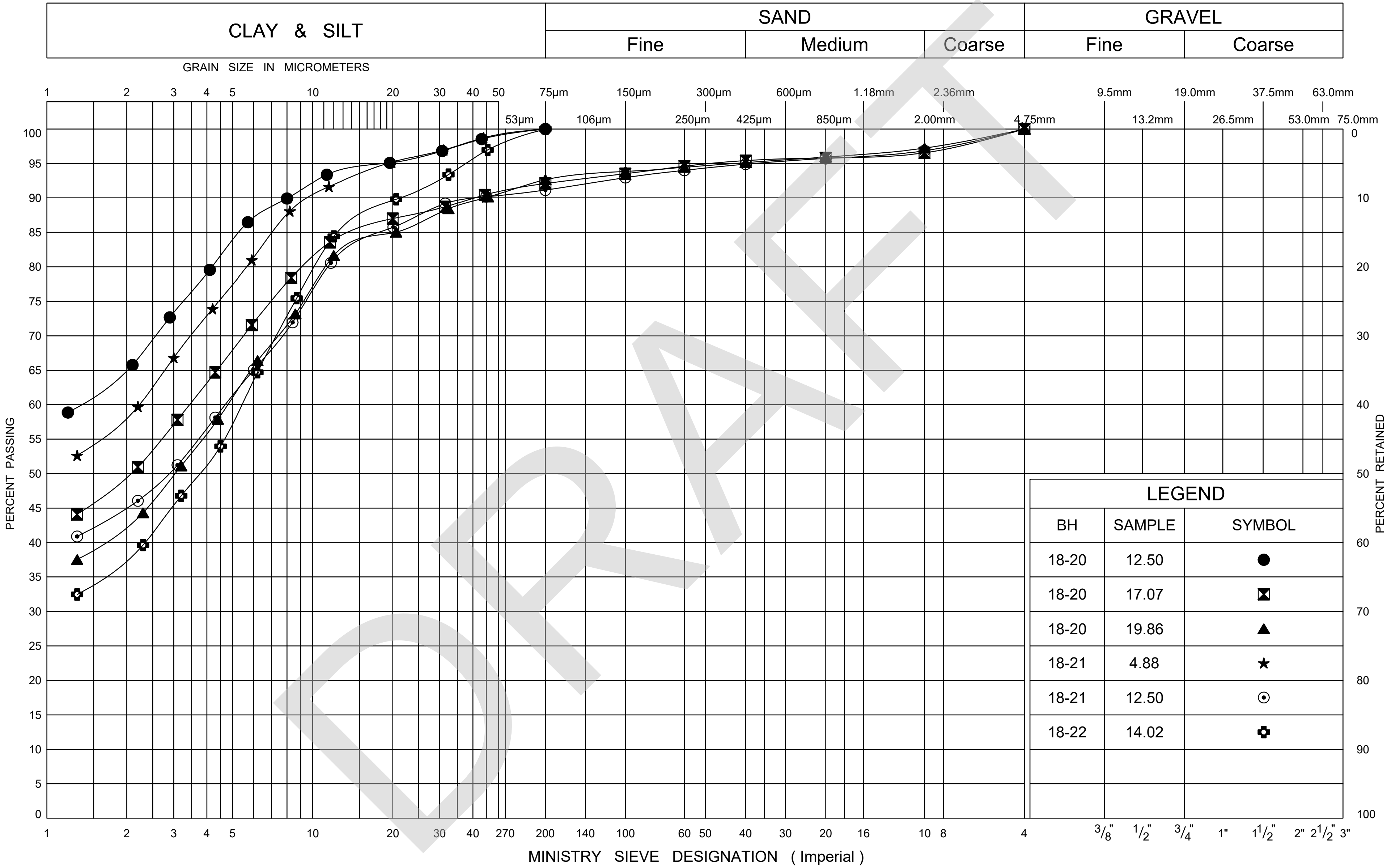
FIG No B15

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
18-20	12.50	●
18-20	17.07	⊠
18-20	19.86	▲
18-21	4.88	★
18-21	12.50	⊙
18-22	14.02	✤

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Silty CLAY

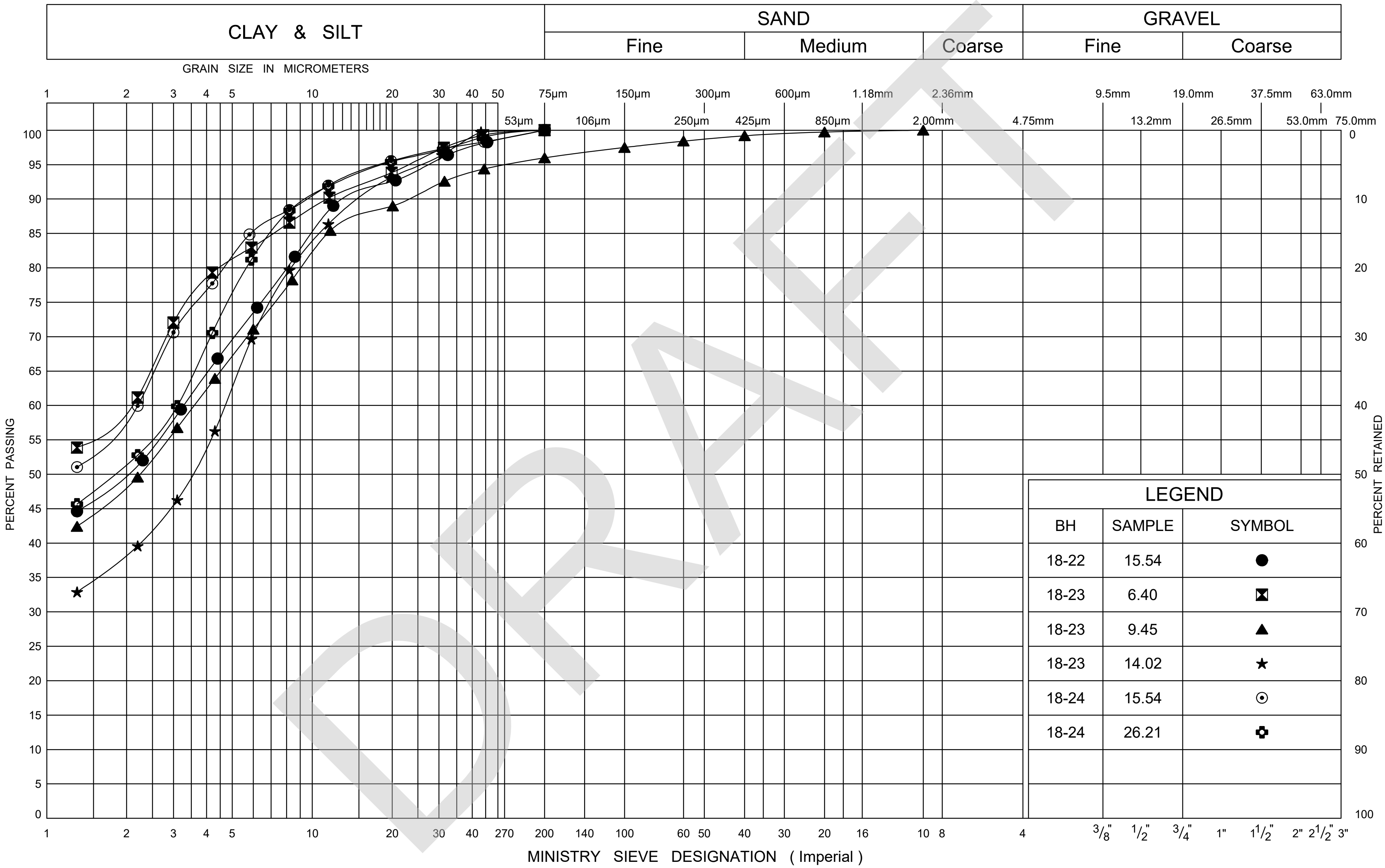
FIG No B16

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
18-22	15.54	●
18-23	6.40	⧗
18-23	9.45	▲
18-23	14.02	★
18-24	15.54	⊙
18-24	26.21	✿

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

Silty CLAY

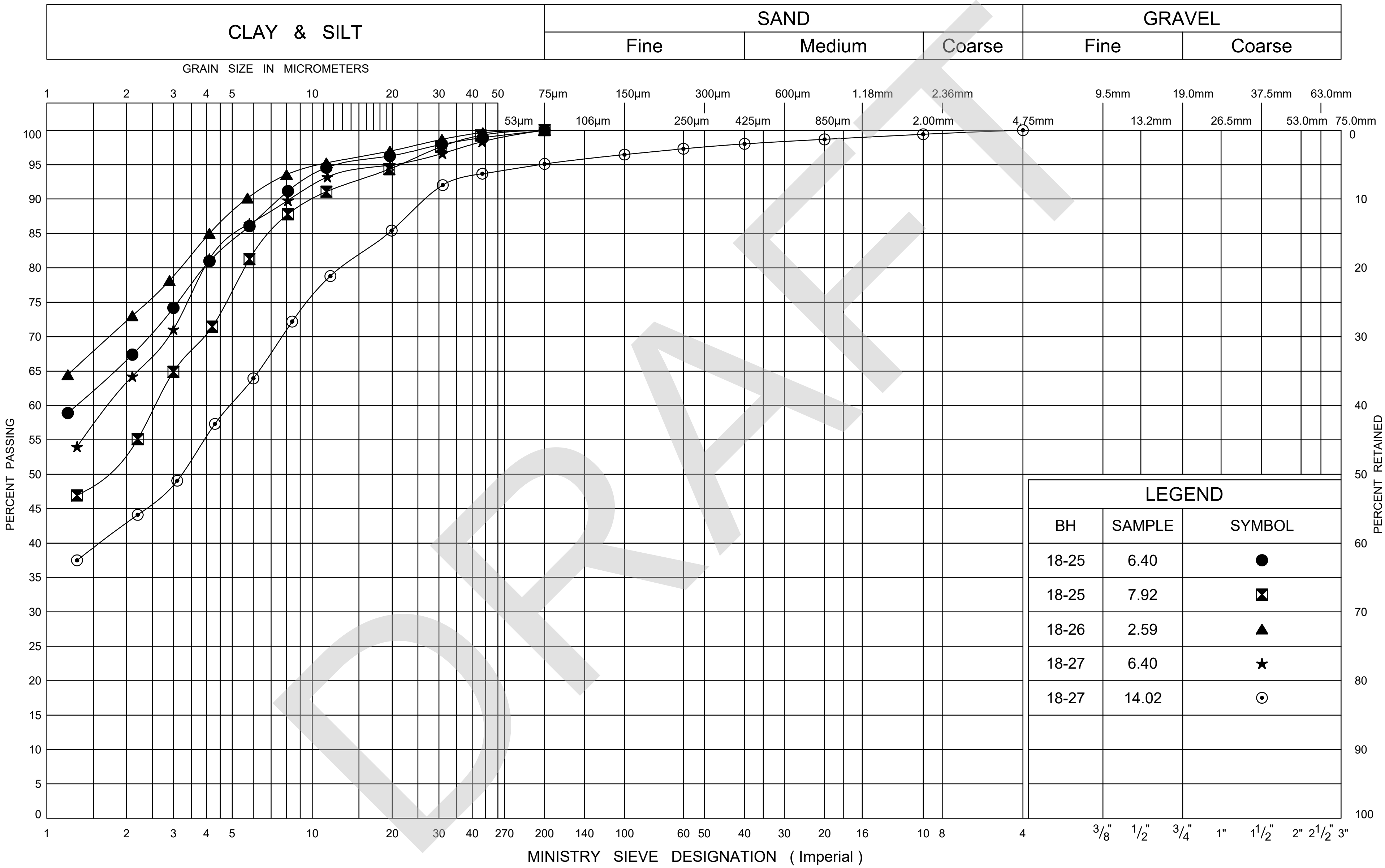
FIG No B17

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

Silty CLAY

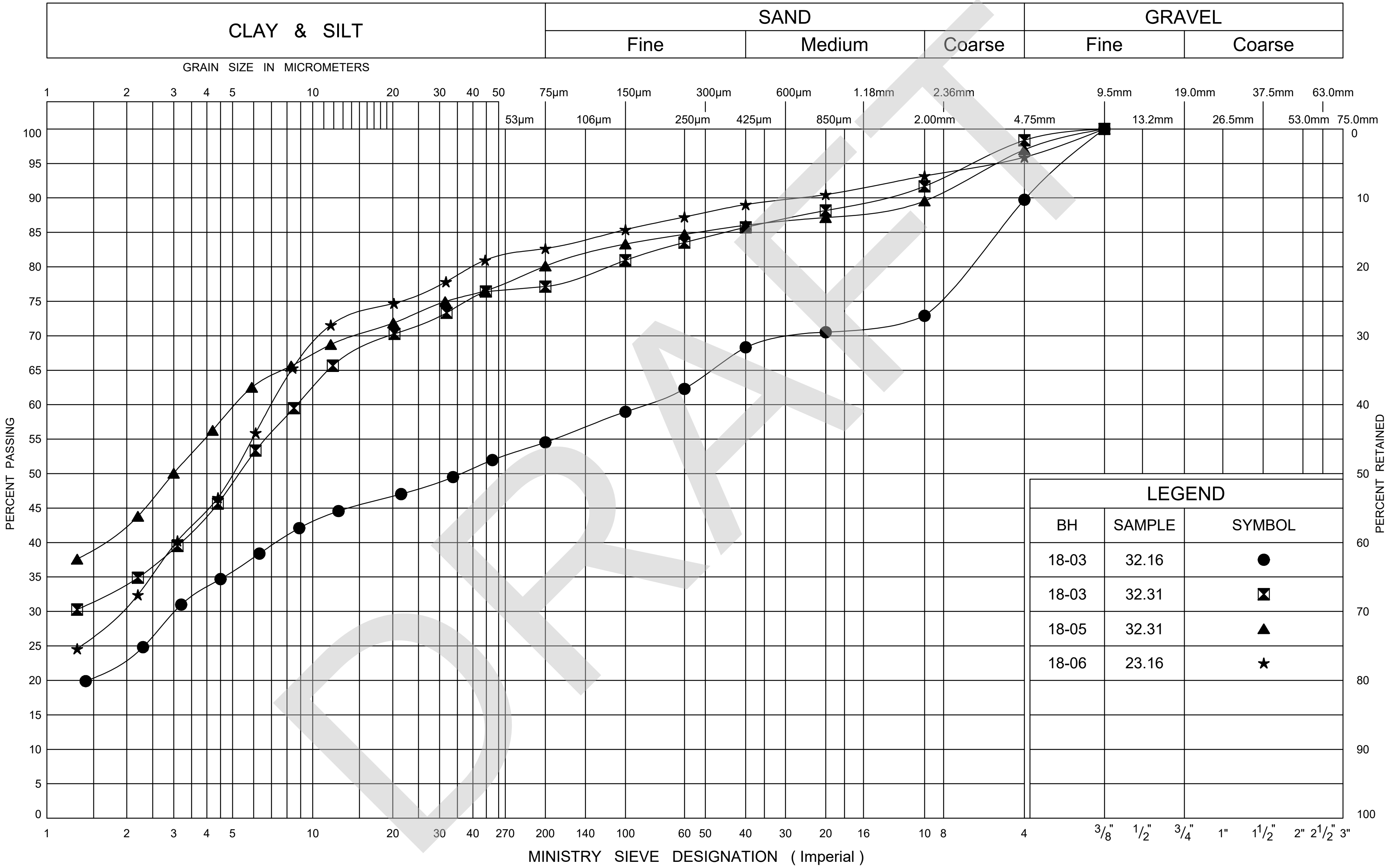
FIG No B18

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
18-03	32.16	●
18-03	32.31	⊠
18-05	32.31	▲
18-06	23.16	★

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

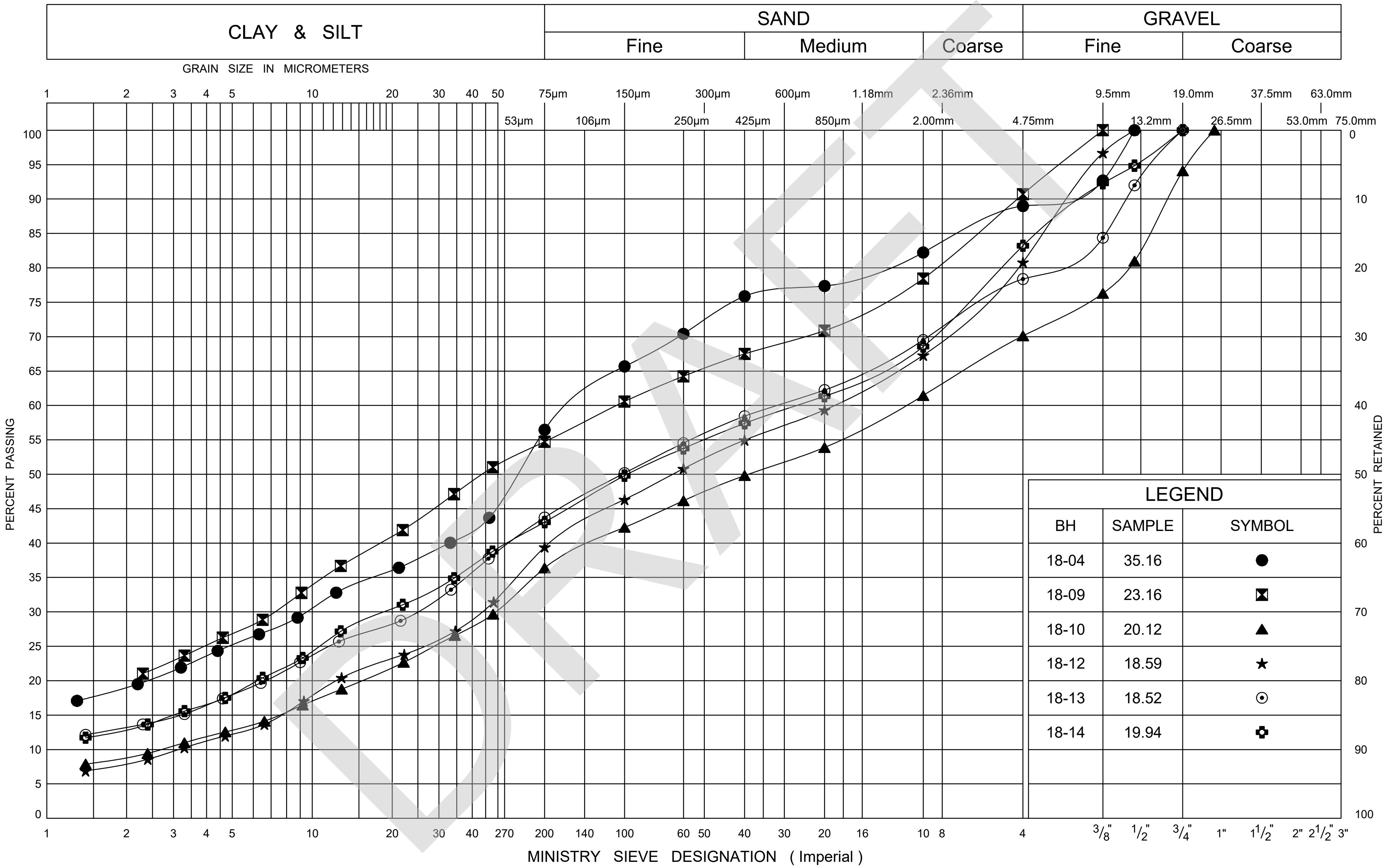
Silty CLAY TILL

FIG No B19

GWP 2430-15-00

Welland River Bridge Replacement

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILT and SAND TILL

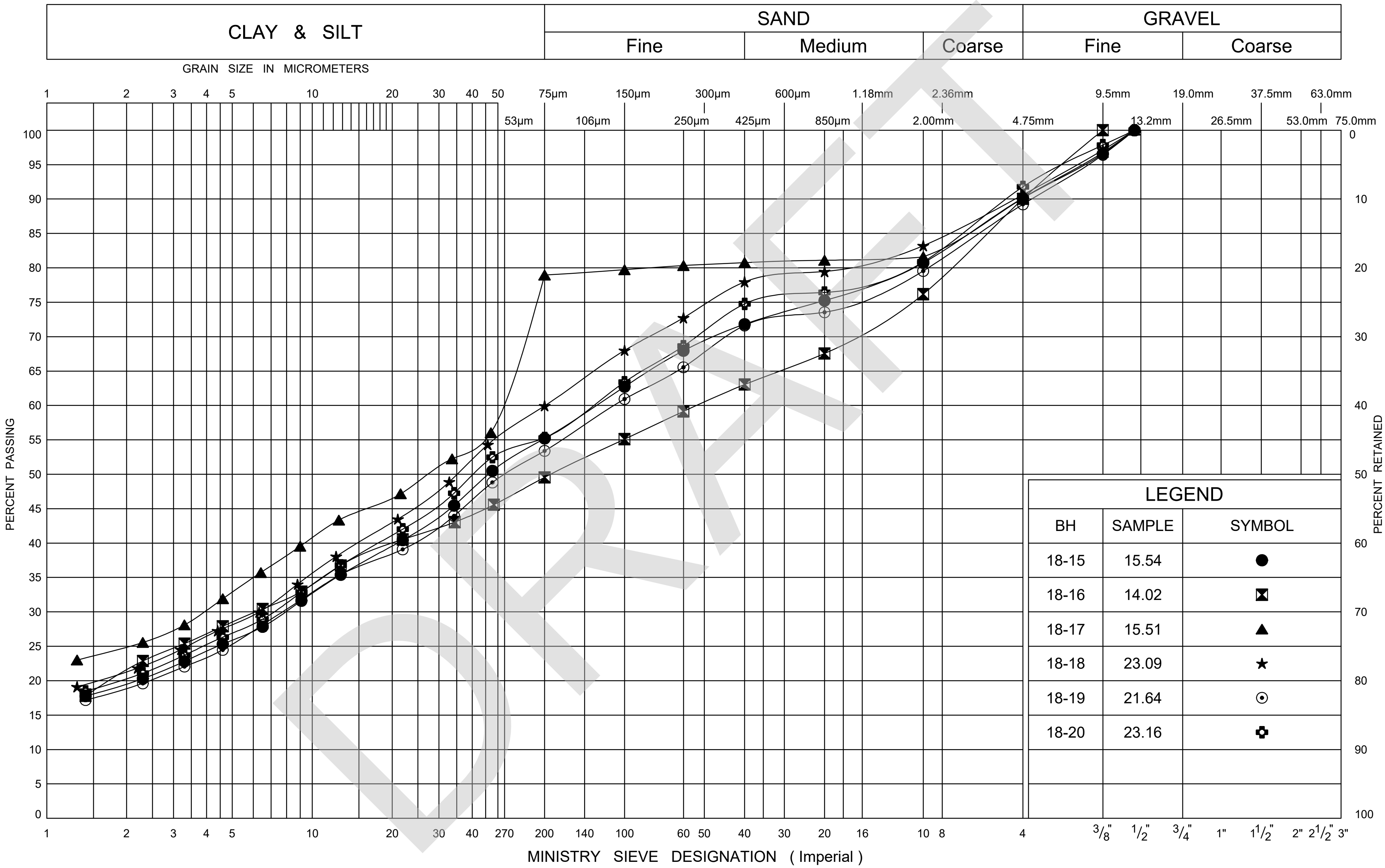
FIG No B20

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

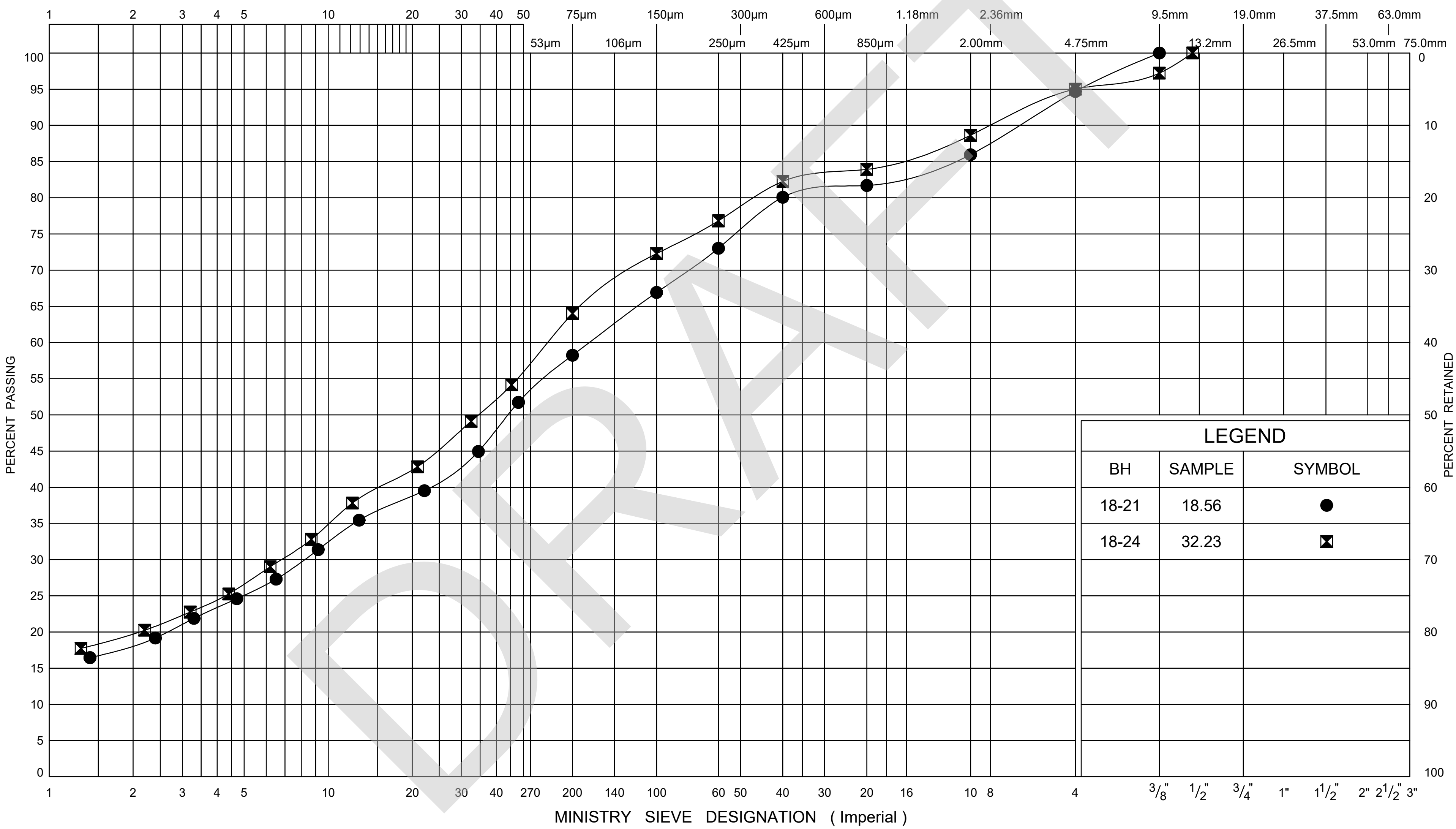
SILT and SAND TILL

FIG No B21

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18



LEGEND		
BH	SAMPLE	SYMBOL
18-21	18.56	●
18-24	32.23	⊠

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILT and SAND TILL

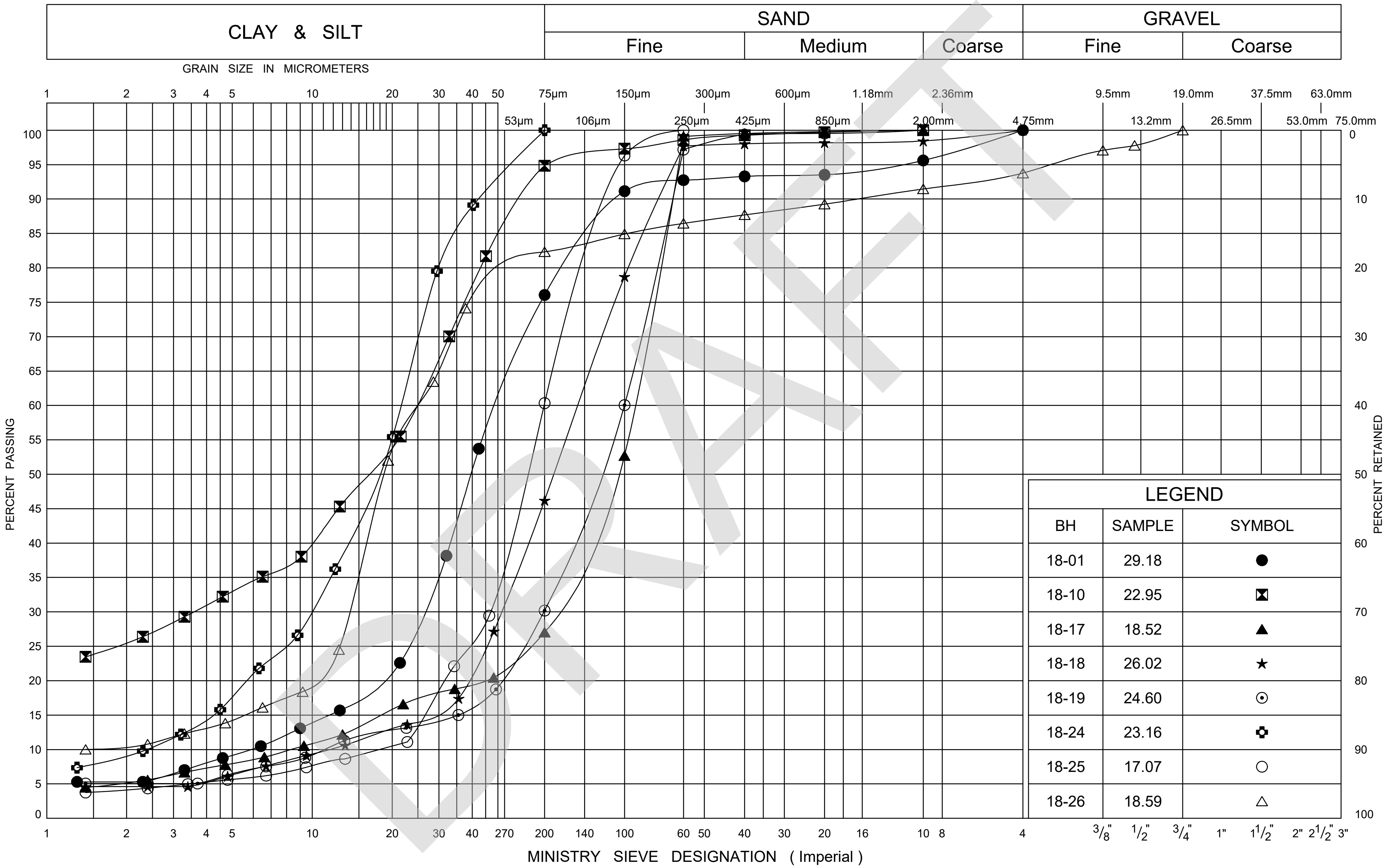
FIG No B22

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILT to Silty SAND

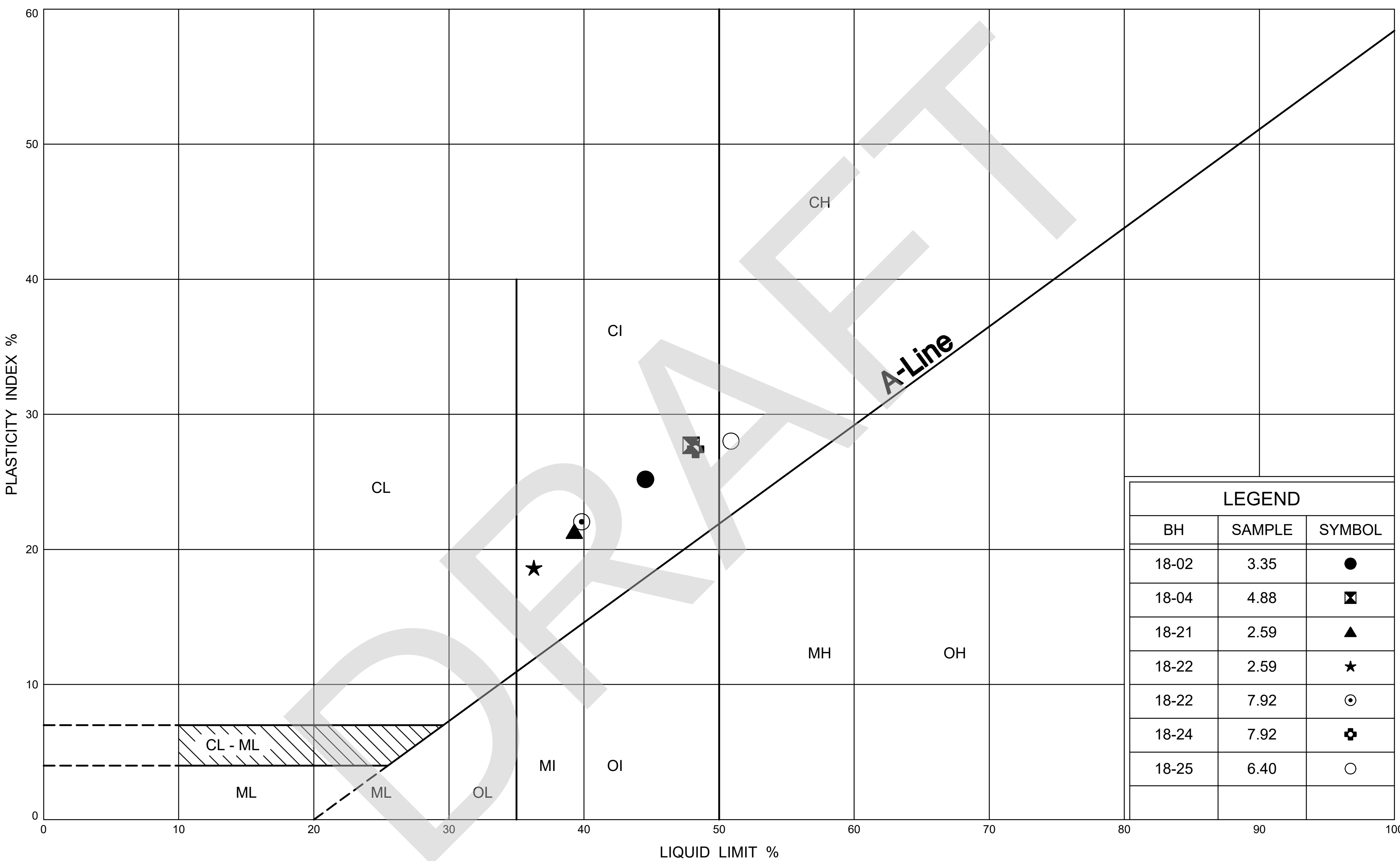
FIG No B23

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT GRAIN SIZE 2 MTO-18426.GPJ ONTARIO MOT.GDT 8/21/18

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
18-02	3.35	●
18-04	4.88	⧗
18-21	2.59	▲
18-22	2.59	★
18-22	7.92	⊙
18-24	7.92	✜
18-25	6.40	○

Ministry of Transportation
Ontario

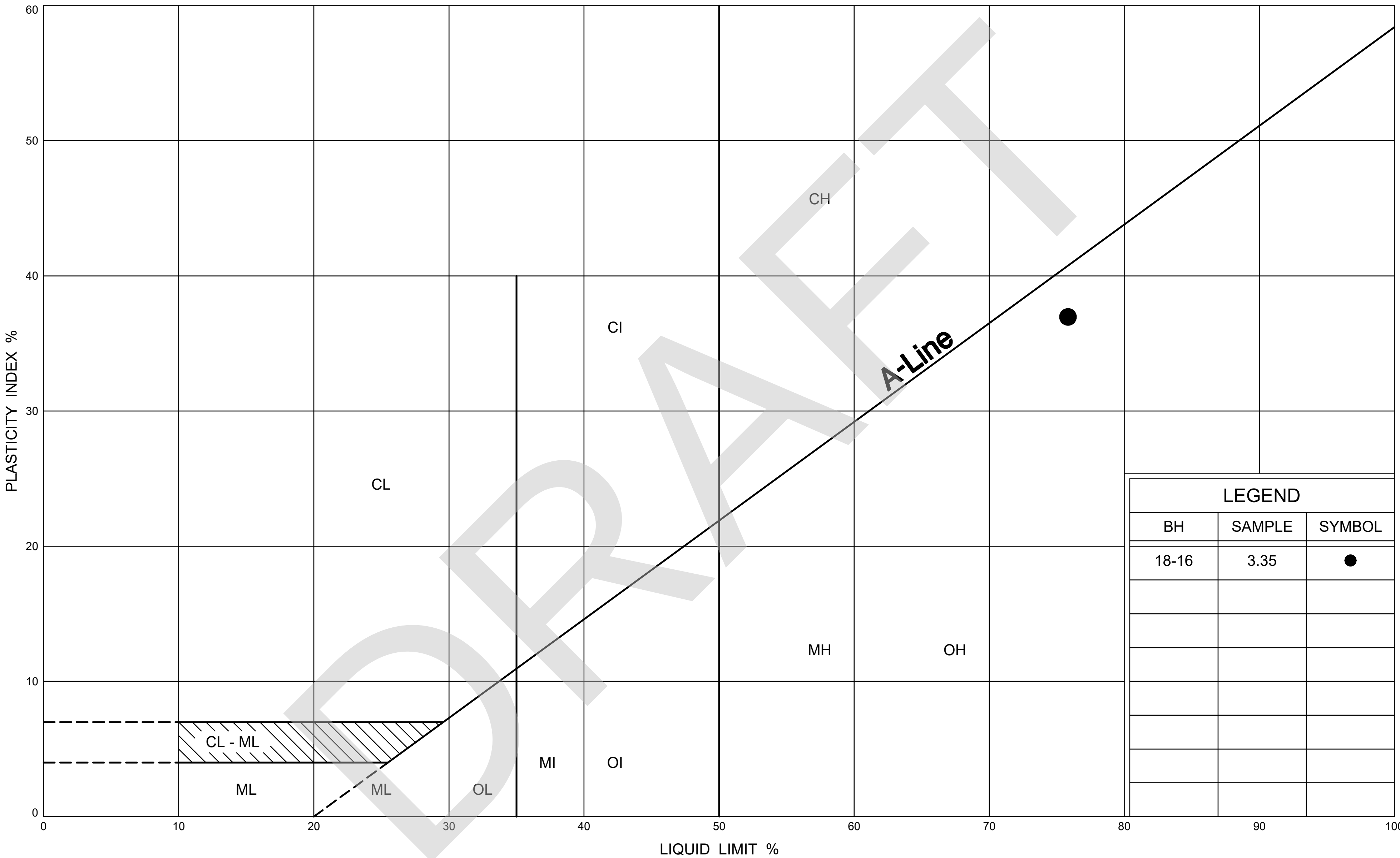
PLASTICITY CHART
Cohesive FILL

FIG No B24

GWP 2430-15-00

Welland River Bridge Replacement

Oct 75, FF - S - 21



LEGEND

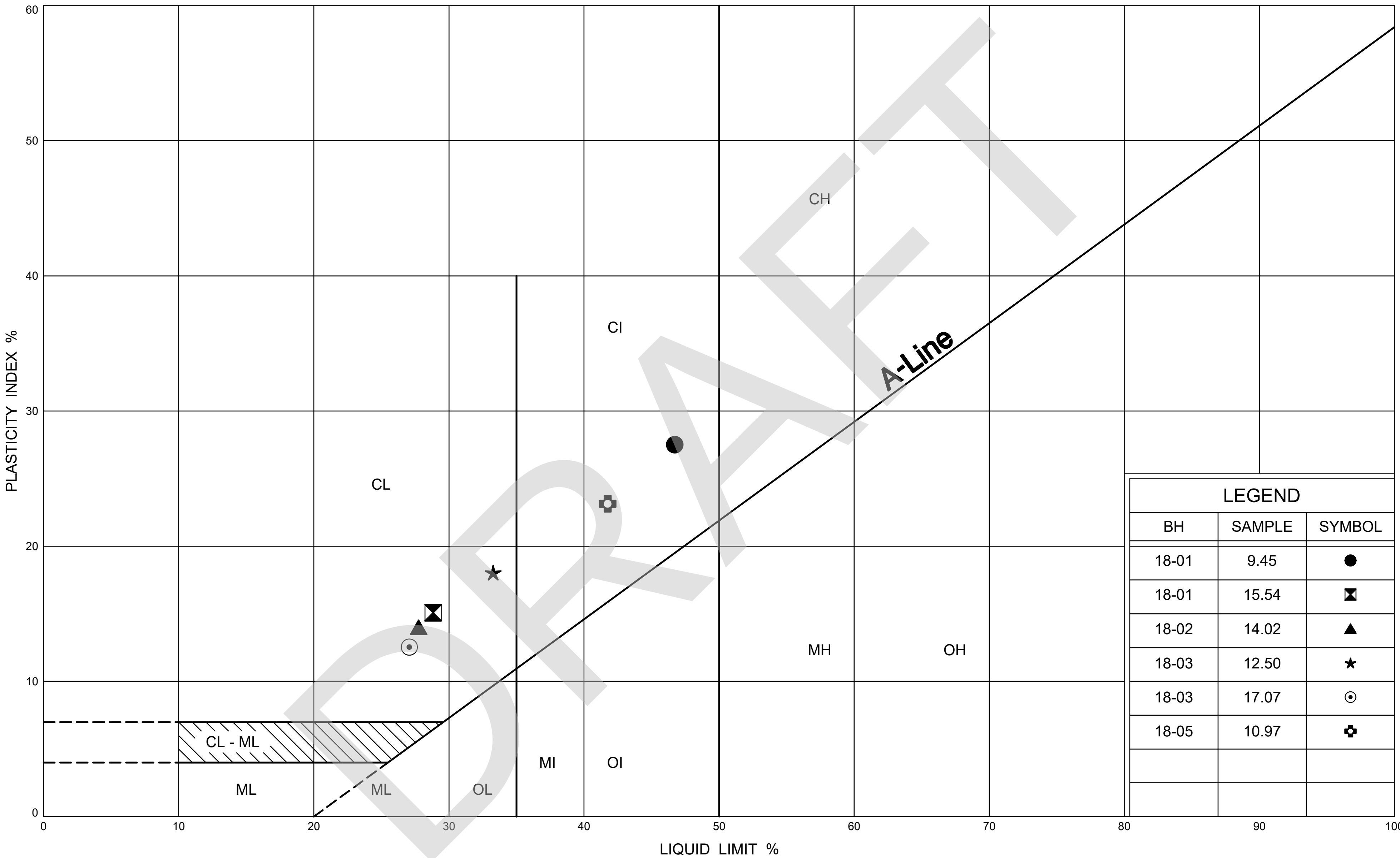
BH	SAMPLE	SYMBOL
18-16	3.35	●

Ministry of Transportation
Ontario

PLASTICITY CHART
Organic SILT

FIG No B25
GWP 2430-15-00
Welland River Bridge Replacement

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
18-01	9.45	●
18-01	15.54	⧗
18-02	14.02	▲
18-03	12.50	★
18-03	17.07	⊙
18-05	10.97	✪

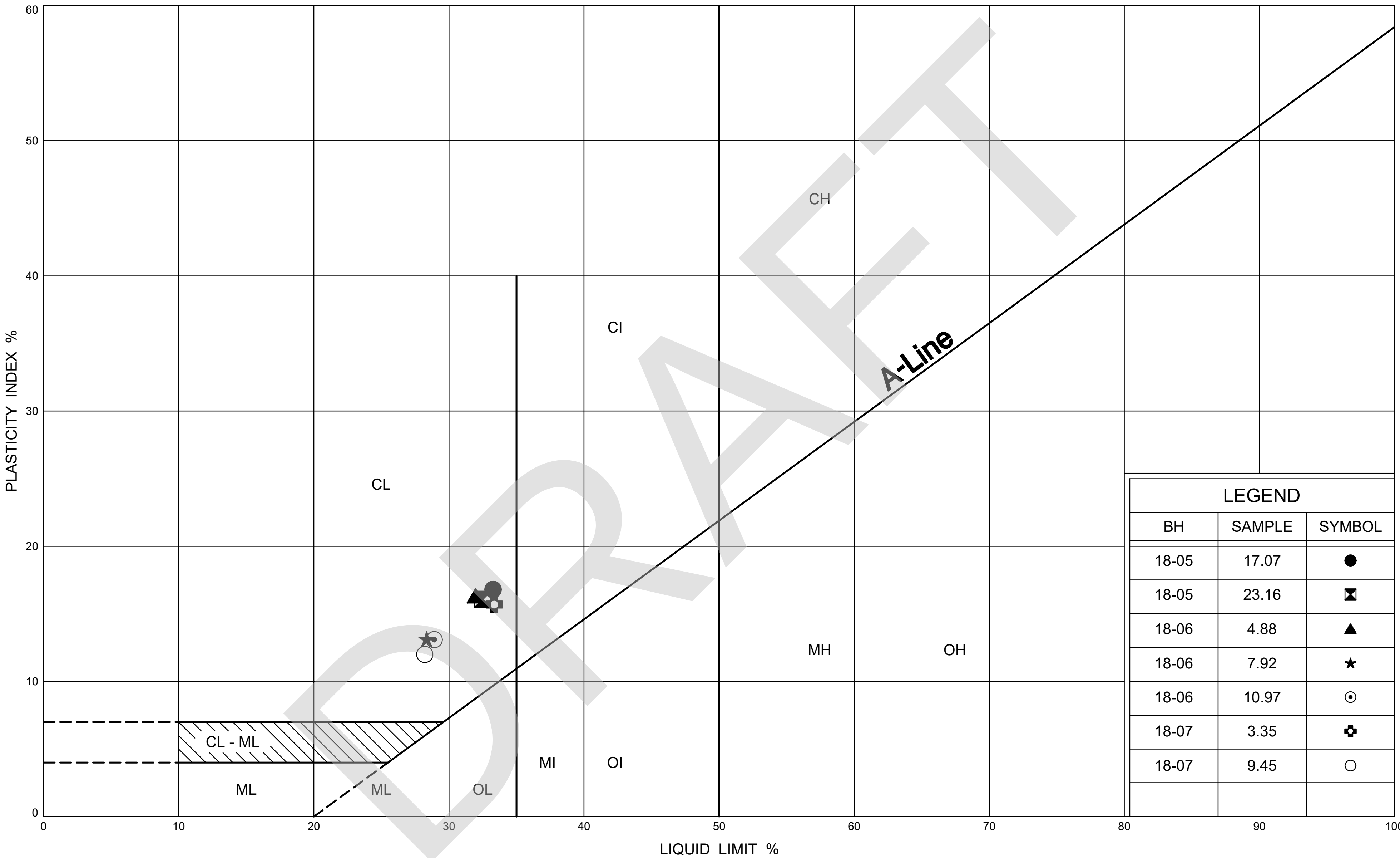
Ministry of Transportation
Ontario

PLASTICITY CHART
Silty CLAY

FIG No B26
GWP 2430-15-00
Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
18-05	17.07	●
18-05	23.16	⧗
18-06	4.88	▲
18-06	7.92	★
18-06	10.97	⊙
18-07	3.35	✿
18-07	9.45	○

Ministry of Transportation
Ontario

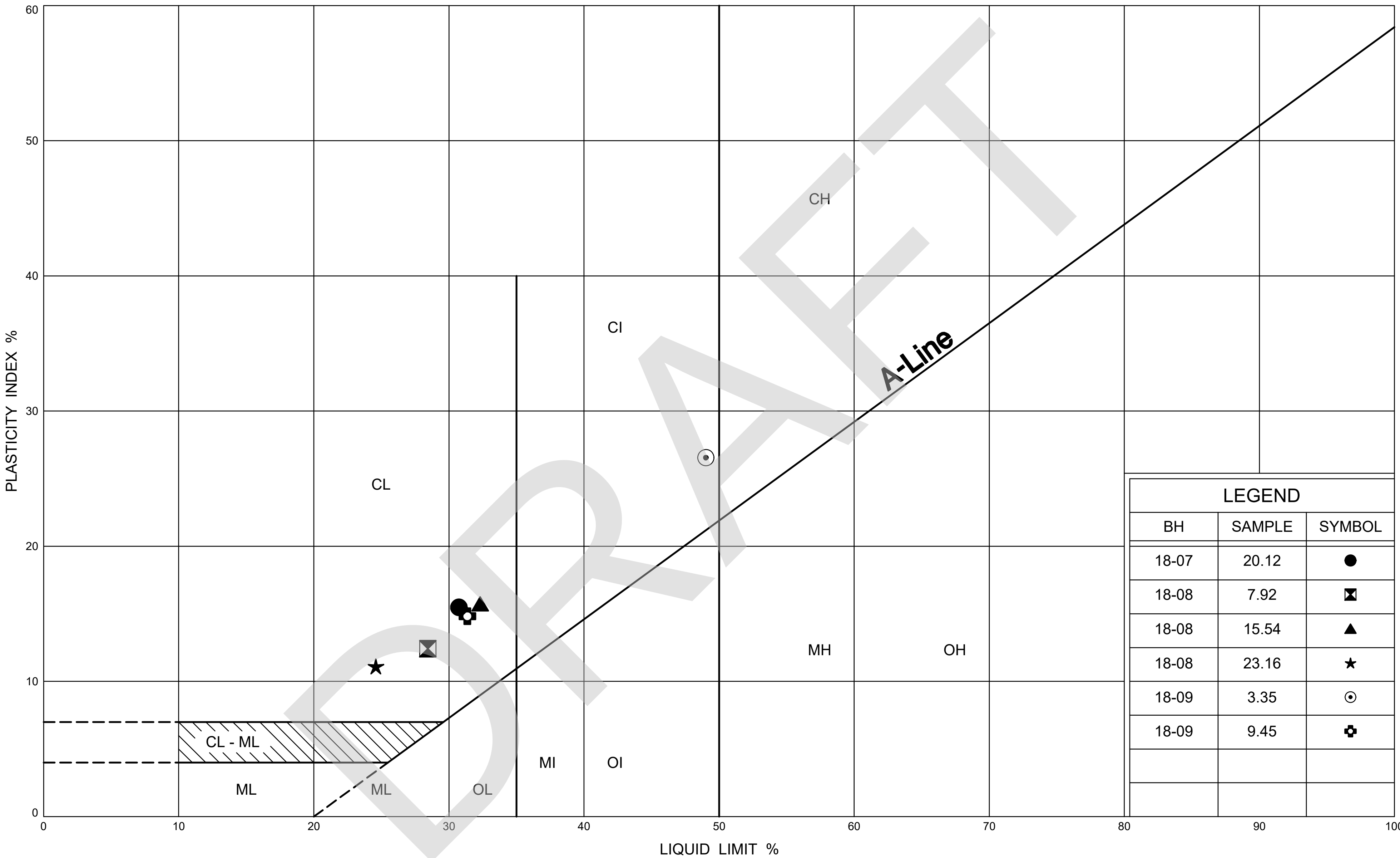
PLASTICITY CHART
Silty CLAY

FIG No B27

GWP 2430-15-00

Welland River Bridge Replacement

Oct 75, FF - S - 21



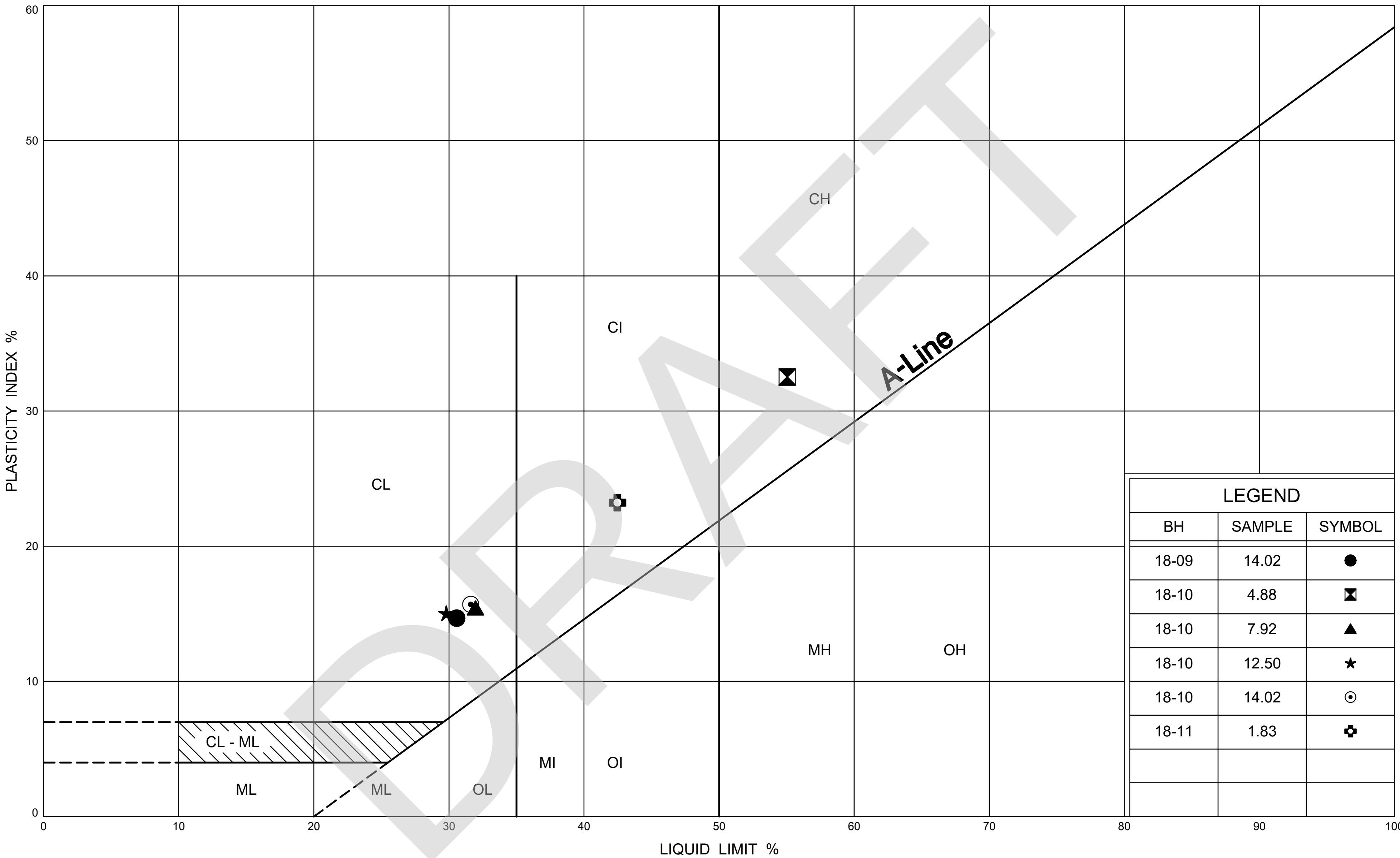
Ministry of Transportation
Ontario

PLASTICITY CHART
Silty CLAY

FIG No B28
GWP 2430-15-00
Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
18-09	14.02	●
18-10	4.88	⧗
18-10	7.92	▲
18-10	12.50	★
18-10	14.02	⊙
18-11	1.83	✿

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty CLAY

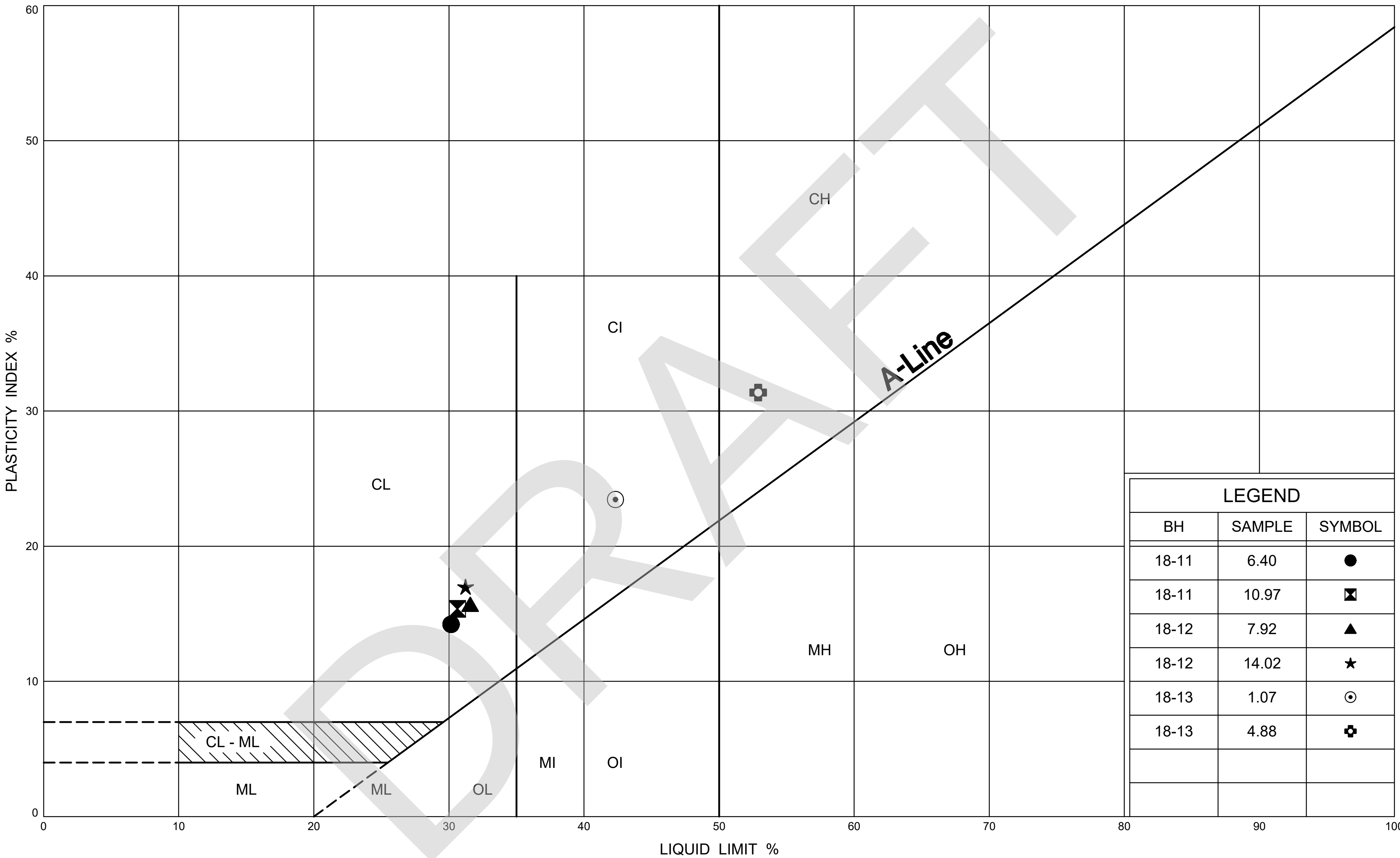
FIG No B29

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
18-11	6.40	●
18-11	10.97	⧗
18-12	7.92	▲
18-12	14.02	★
18-13	1.07	⊙
18-13	4.88	✿

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty CLAY

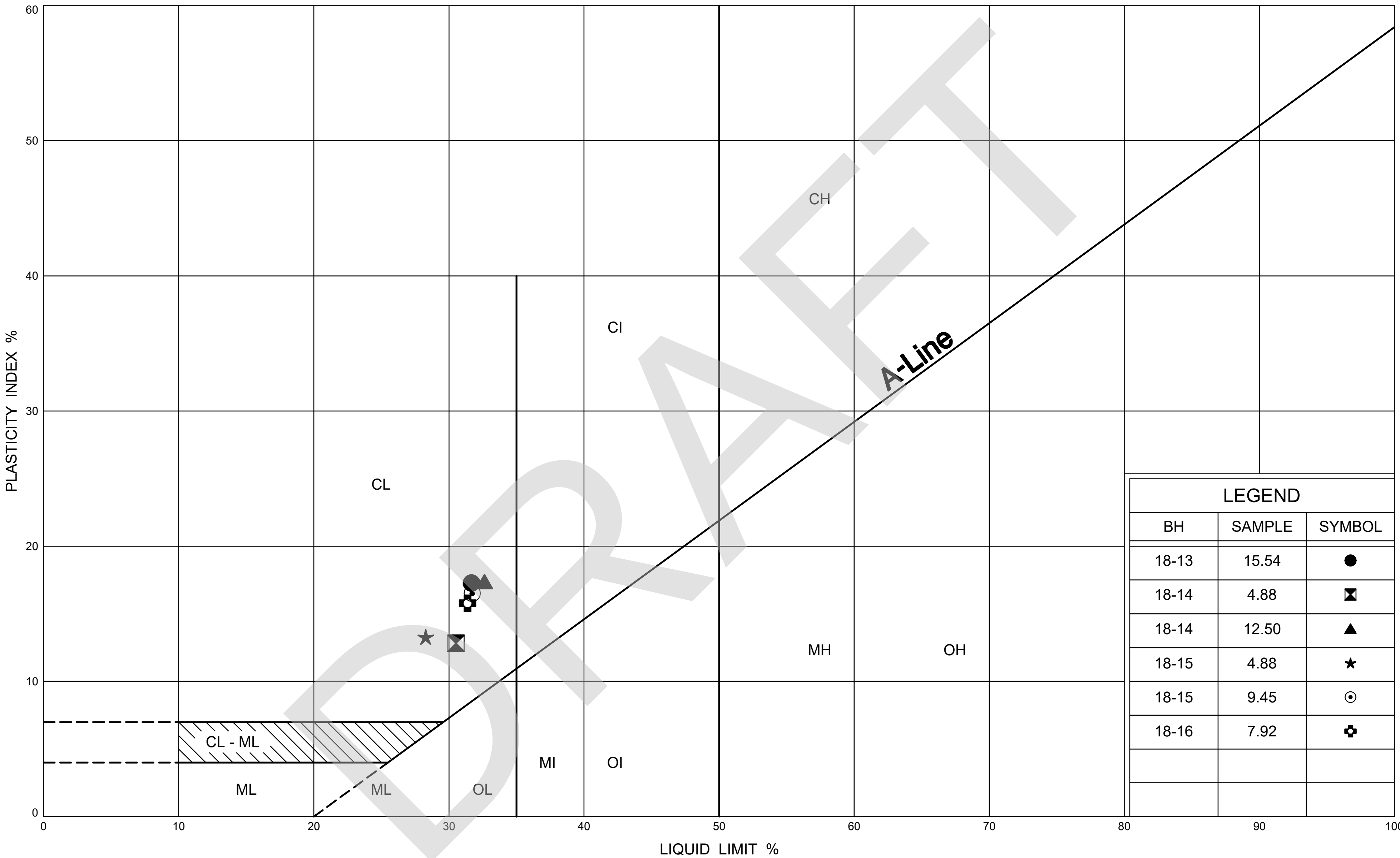
FIG No B30

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
18-13	15.54	●
18-14	4.88	⧗
18-14	12.50	▲
18-15	4.88	★
18-15	9.45	⊙
18-16	7.92	✿

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty CLAY

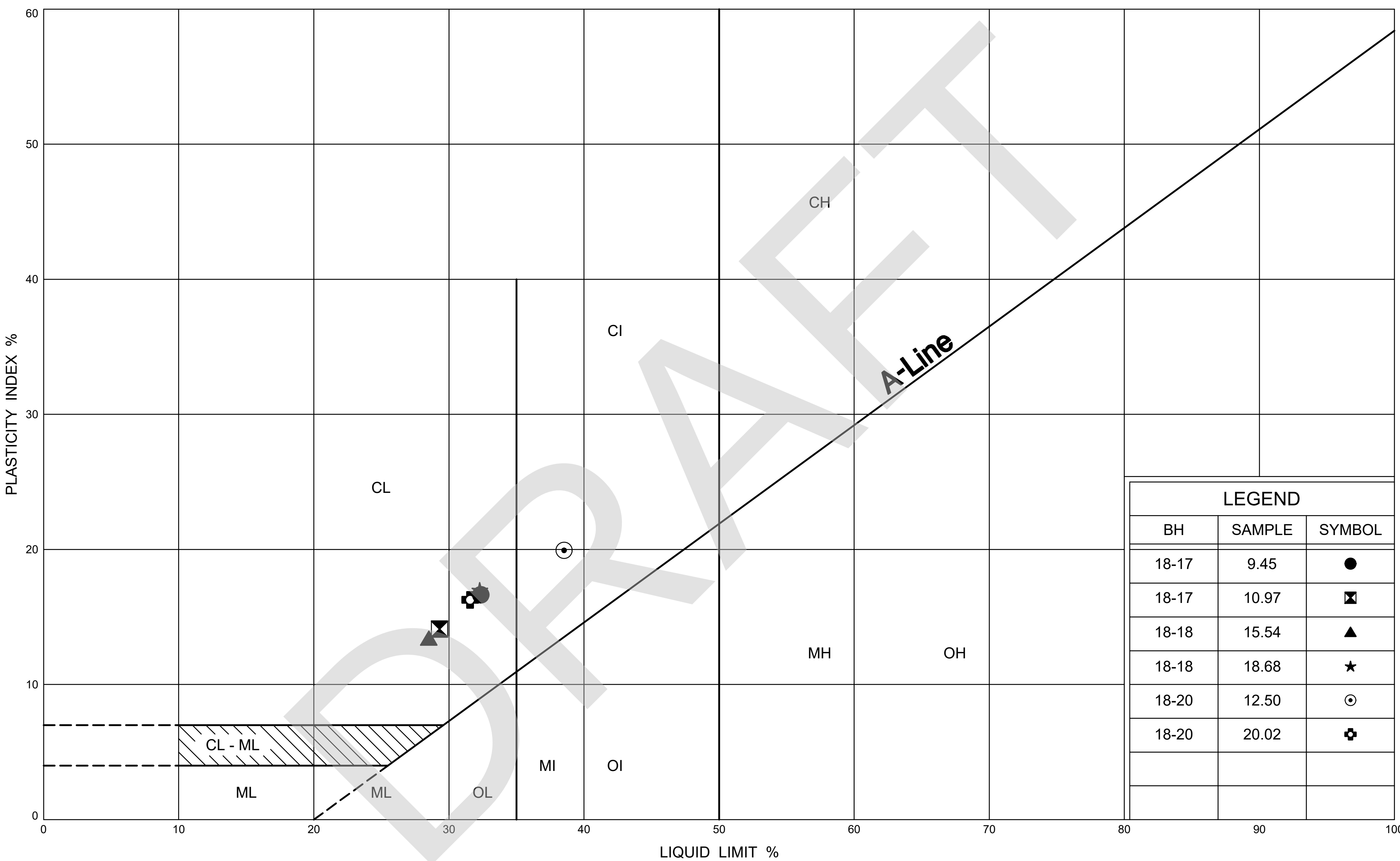
FIG No B31

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



Ministry of Transportation

Ontario

PLASTICITY CHART

Silty CLAY

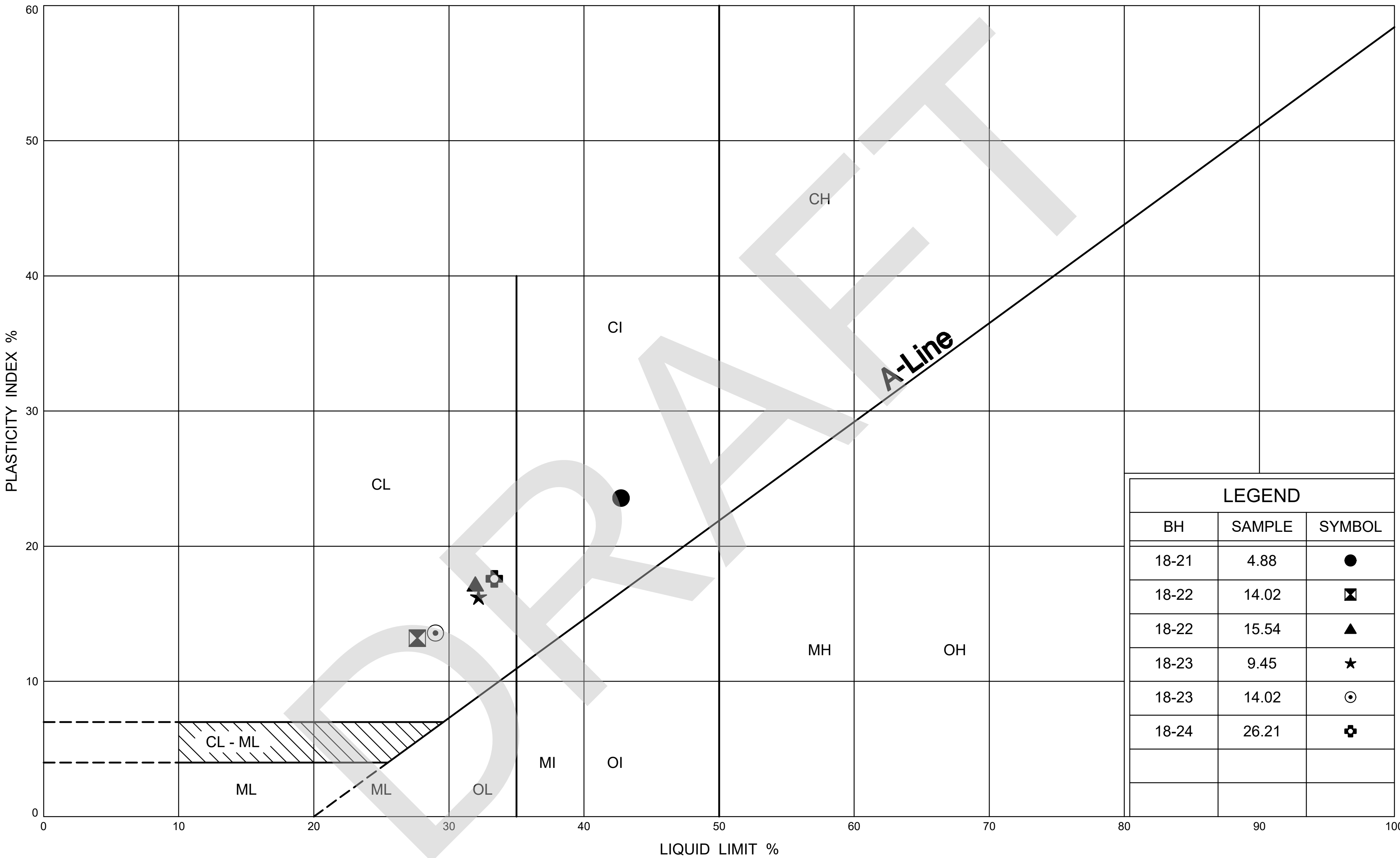
FIG No B32

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
18-21	4.88	●
18-22	14.02	⧗
18-22	15.54	▲
18-23	9.45	★
18-23	14.02	⊙
18-24	26.21	✿

Ministry of Transportation
Ontario

PLASTICITY CHART

Silty CLAY

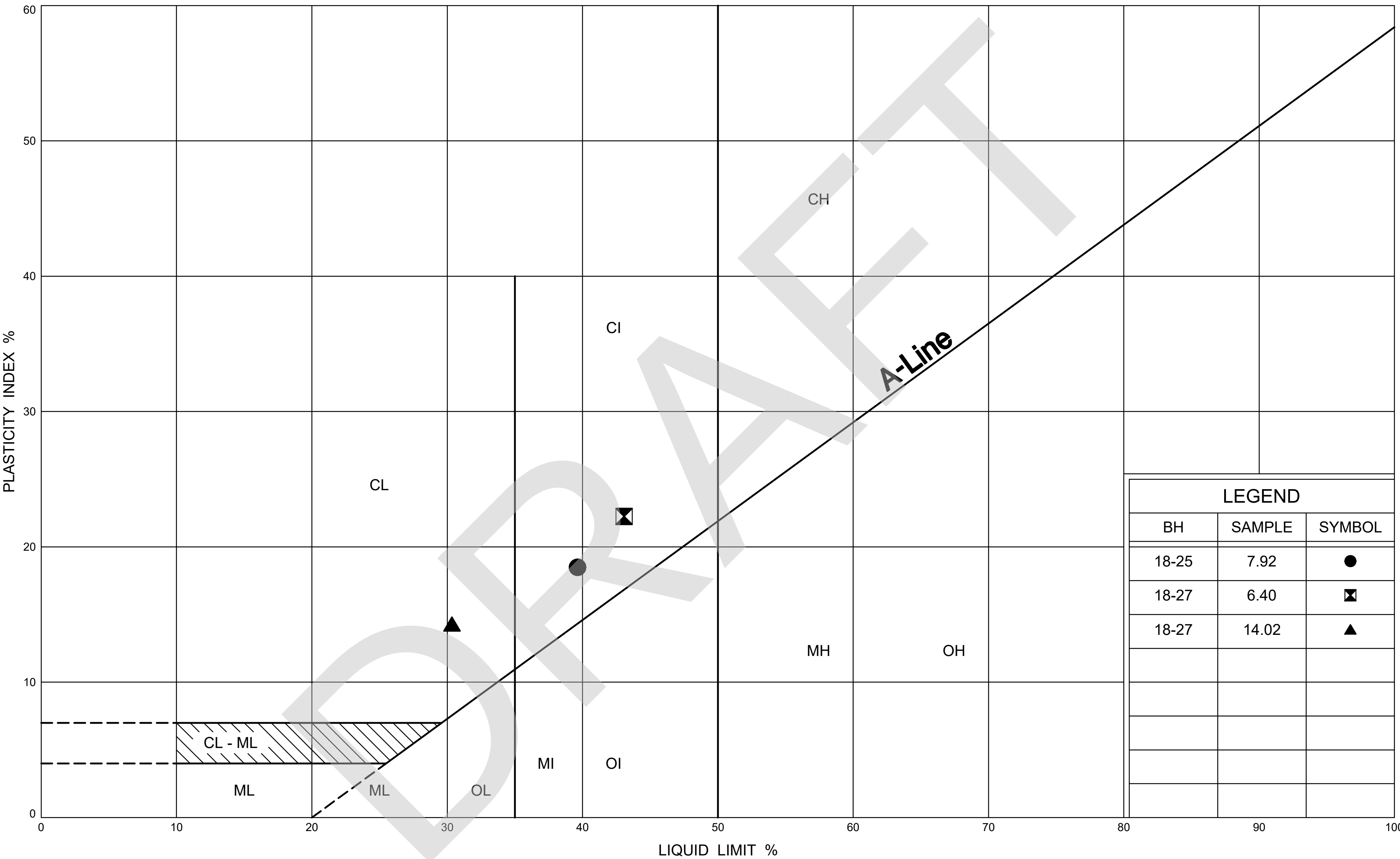
FIG No B33

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
18-25	7.92	●
18-27	6.40	⧗
18-27	14.02	▲

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Ministry of Transportation
Ontario

PLASTICITY CHART

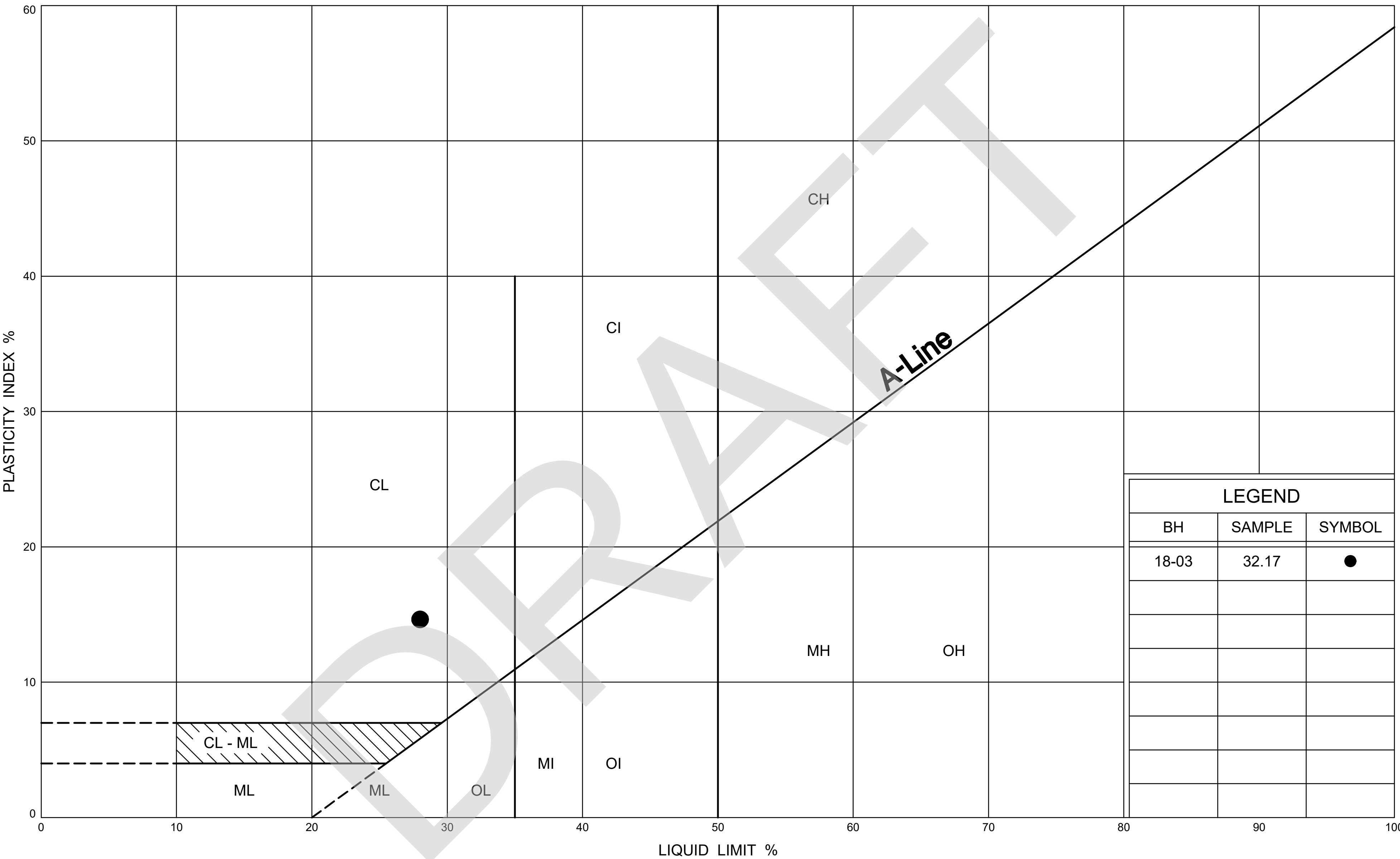
Silty CLAY

FIG No B34

GWP 2430-15-00

Welland River Bridge Replacement

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
18-03	32.17	●

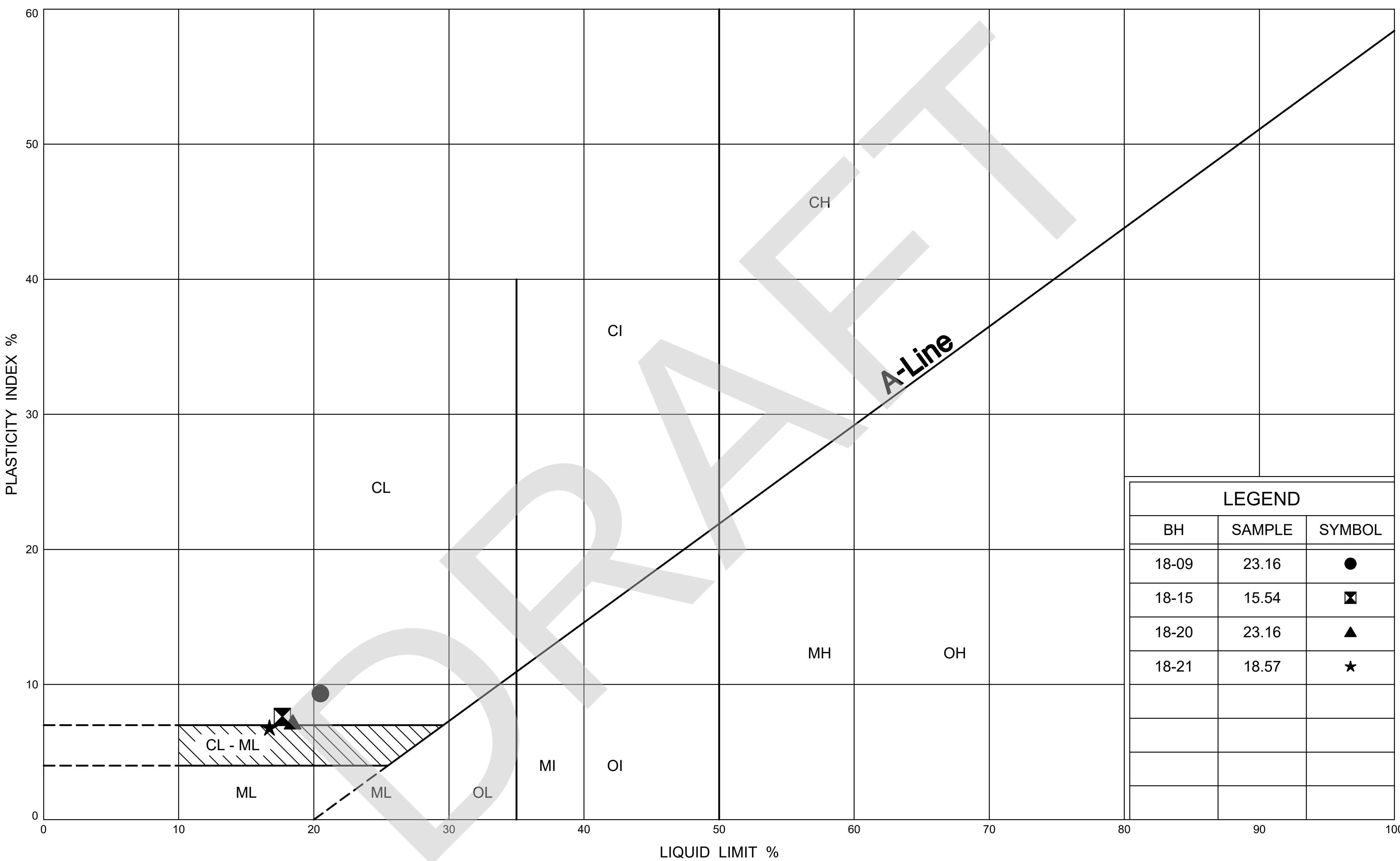
ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

Ministry of Transportation
Ontario

PLASTICITY CHART
Silty CLAY TILL

FIG No B35
GWP 2430-15-00
Welland River Bridge Replacement

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
18-09	23.16	●
18-15	15.54	⧗
18-20	23.16	▲
18-21	18.57	★

Ministry of Transportation
Ontario

PLASTICITY CHART

SILT and SAND TILL

FIG No B36

GWP 2430-15-00

Welland River Bridge Replacement

ONTARIO MOT PLASTICITY CHART MTO-18426.GPJ ONTARIO MOT.GDT 8/16/18

THURBER ENGINEERING LTD.

Consolidation Test Report

CLIENT: **WSP** FILE NUMBER: 18426

PROJECT: Welland River Bridges REPORT DATE: 24-Apr-2018

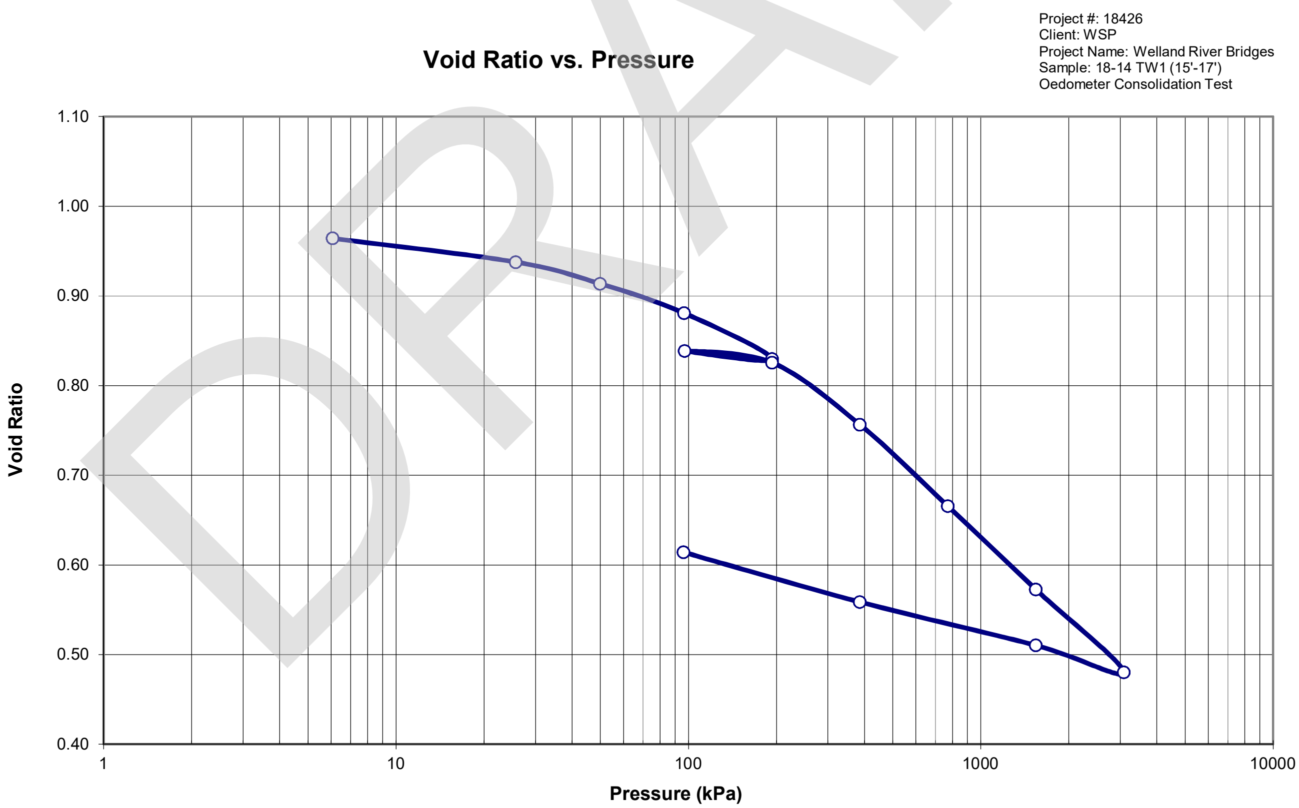
TEST DATES: April 09, 2018 - April 22, 2018

SAMPLE: 18-14 TW1 (15'-17')
Silty Clay
Silt = 59%, Clay = 41%, LL=30.5%, PL=17.7%.

PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method A

	Start of Test	End of Test
Wet Dens. (kg/m^3)	2000.8	2154.2
Dry Dens. (kg/m^3)	1411.0	1721.3
Moisture Cont. (%)	41.8	25.1
Void Ratio	0.969	0.614

Void Ratio vs. Pressure



TEST DONE BY: EA/BT
REVIEWED BY: GRL

THURBER ENGINEERING LTD.

Consolidation Test Report

Welland River Bridges
18426 18-14 TW1 (15'-17')

TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

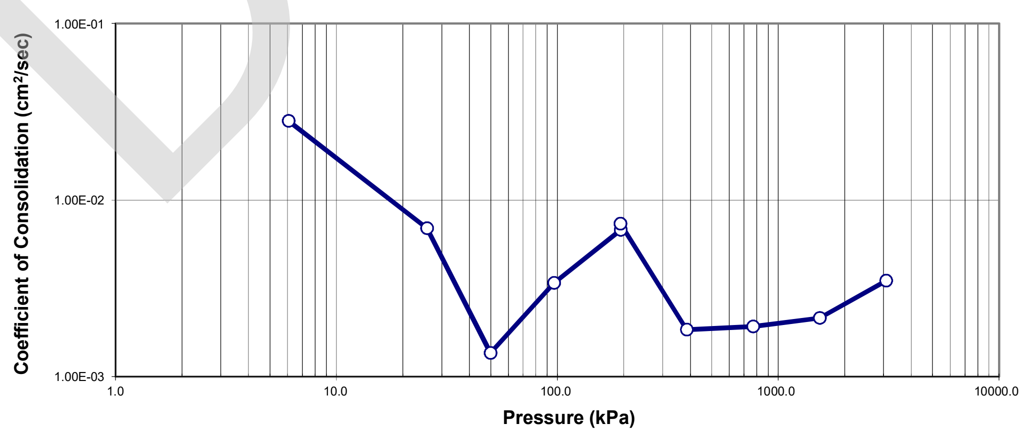
LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after a constant load increment duration of 24 hours.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm^2/s)	Void Ratio	m_v (m^2/kN)	k (cm/s)
0.0	25.400					0.969		
6.1	25.338	25.369	-0.048	0.81	2.81E-02	0.964	4.02E-04	1.11E-06
25.7	24.996	25.167	-0.173	3.24	6.91E-03	0.938	6.89E-04	4.66E-07
49.9	24.685	24.840	-0.161	16.00	1.36E-03	0.914	5.15E-04	6.88E-08
96.6	24.263	24.474	-0.190	6.25	3.39E-03	0.881	3.65E-04	1.21E-07
193.2	23.601	23.932	-0.244	2.99	6.76E-03	0.830	2.82E-04	1.87E-07
97.0	23.717	23.659				0.839		
193.0	23.551	23.634	-0.089	2.690	0.007	0.826	7.27E-05	5.23E-08
385.7	22.658	23.104	-0.502	10.240	0.002	0.756	1.97E-04	3.56E-08
770.0	21.487	22.072	-0.659	8.94	1.93E-03	0.666	1.34E-04	2.54E-08
1540.0	20.288	20.887	-0.700	7.18	2.15E-03	0.573	7.25E-05	1.53E-08
3080.0	19.096	19.692	-0.646	3.92	3.49E-03	0.480	3.81E-05	1.31E-08
1540.0	19.482	19.289				0.510		
385.0	20.106	19.794				0.559		
96.3	20.821	20.463				0.614		

Project #: 18426
Client: WSP
Project Name: Welland River Bridges
Sample: 18-14 TW1 (15'-17')
Oedometer Consolidation Test

Coefficient of Consoildation vs. Pressure



Notes: C_v and k calculated using t_{90} values

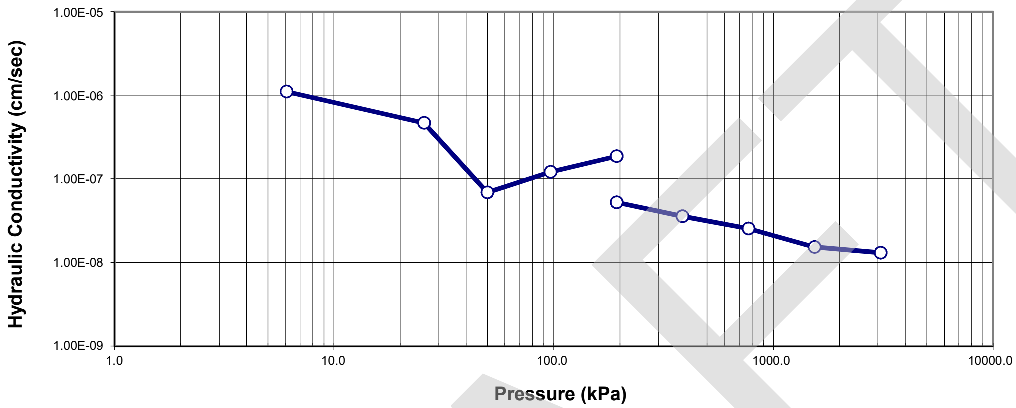
TEST DONE BY: EA/BT
REVIEWED BY: GRL

Consolidation Test Report

Welland River Bridges
18426 | 18-14 TW1 (15'-17')

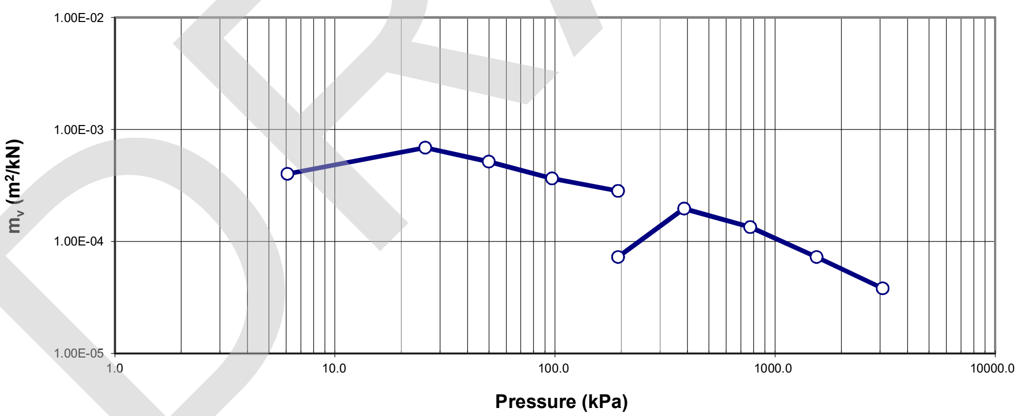
Project #: 18426
Client: WSP
Project Name: Welland River Bridges
Sample: 18-14 TW1 (15'-17')
Oedometer Consolidation Test

Hydraulic Conductivity vs. Pressure



Project #: 18426
Client: WSP
Project Name: Welland River Bridges
Sample: 18-14 TW1 (15'-17')
Oedometer Consolidation Test

m_v vs. Pressure



TEST DONE BY: EA/BT
REVIEWED BY: GRL

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW1
Borehole Number	18-11	Sample Depth, m	6.86-7.47

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	04/06/2018		
Date Completed	04/22/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	19.26
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.93
Area, cm^2	31.65	Specific Gravity, measured	2.75
Volume, cm^3	80.29	Solids Height, cm	1.405
Water Content, %	28.97	Volume of Solids, cm^3	44.46
Wet Mass, g	157.69	Volume of Voids, cm^3	35.83
Dry Mass, g	122.27	Degree of Saturation, %	98.8

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.537	0.806	2.537				
6.01	2.533	0.803	2.535	11	1.24E-01	2.36E-04	2.87E-06
10.72	2.527	0.799	2.530	125	1.09E-02	5.52E-04	5.88E-07
20.17	2.513	0.789	2.520	799	1.68E-03	5.71E-04	9.44E-08
40.12	2.490	0.772	2.501	960	1.38E-03	4.64E-04	6.29E-08
79.27	2.456	0.748	2.473	714	1.82E-03	3.38E-04	6.02E-08
156.18	2.409	0.715	2.432	614	2.04E-03	2.42E-04	4.85E-08
79.27	2.412	0.717	2.411				
156.18	2.405	0.712	2.409	126	9.76E-03	3.84E-05	3.68E-08
310.96	2.331	0.659	2.368	454	2.62E-03	1.89E-04	4.85E-08
621.38	2.230	0.588	2.280	540	2.04E-03	1.27E-04	2.55E-08
1249.94	2.133	0.519	2.182	359	2.81E-03	6.08E-05	1.67E-08
2501.26	2.040	0.452	2.086	277	3.33E-03	2.95E-05	9.65E-09
1251.05	2.046	0.456	2.043				
314.18	2.074	0.476	2.060				
79.27	2.115	0.506	2.095				
20.68	2.163	0.540	2.139				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen taken 2-6 cm from top of the tube.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

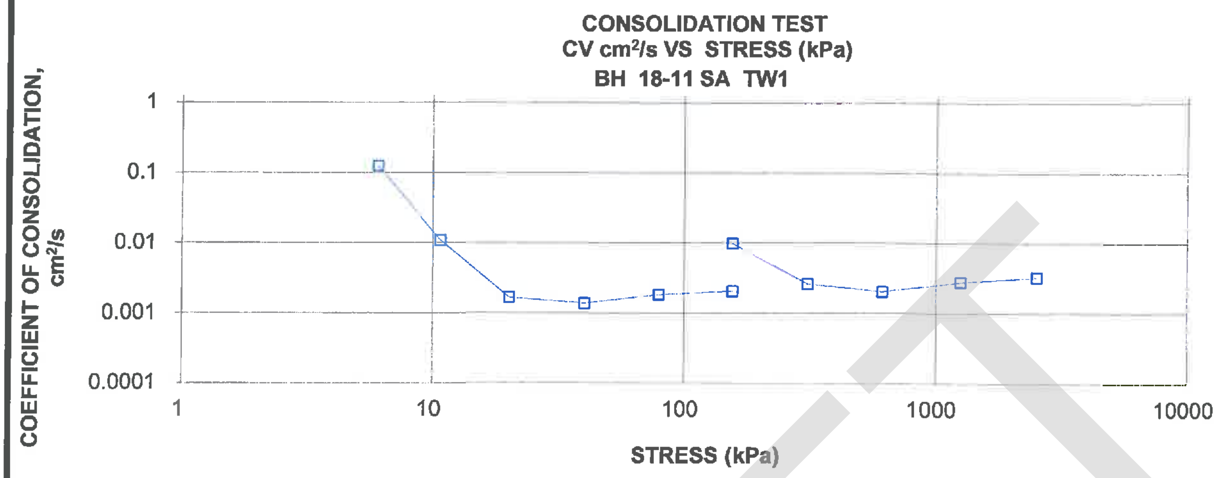
Sample Height, cm	2.16	Unit Weight, kN/m^3	21.12
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.52
Area, cm^2	31.65	Specific Gravity, measured	2.75
Volume, cm^3	68.45	Solids Height, cm	1.405
Water Content, %	20.60	Volume of Solids, cm^3	44.46
Wet Mass, g	147.46	Volume of Voids, cm^3	23.99
Dry Mass, g	122.27		

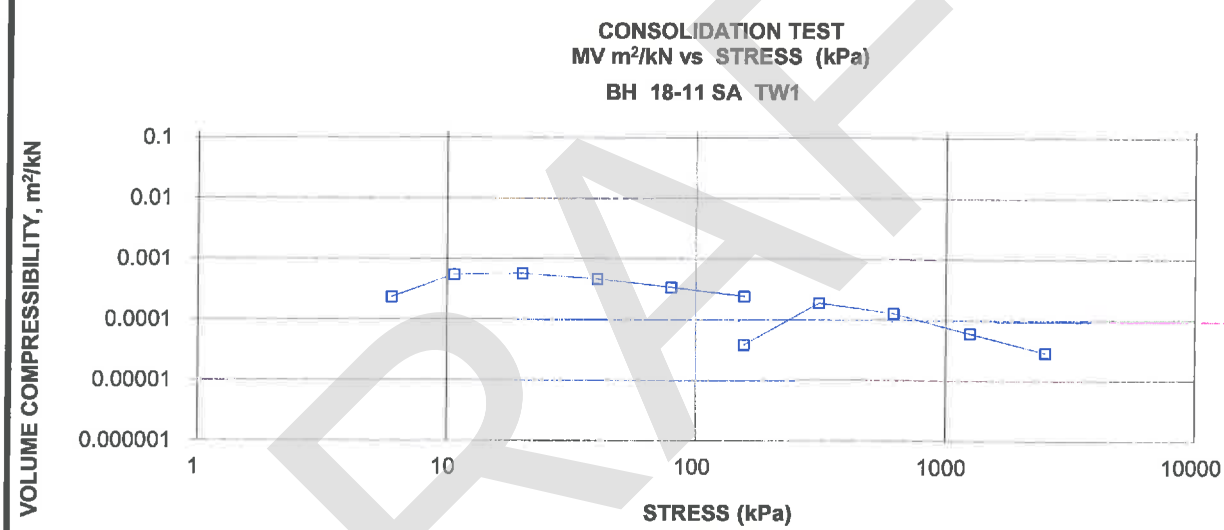
Prepared By: LH

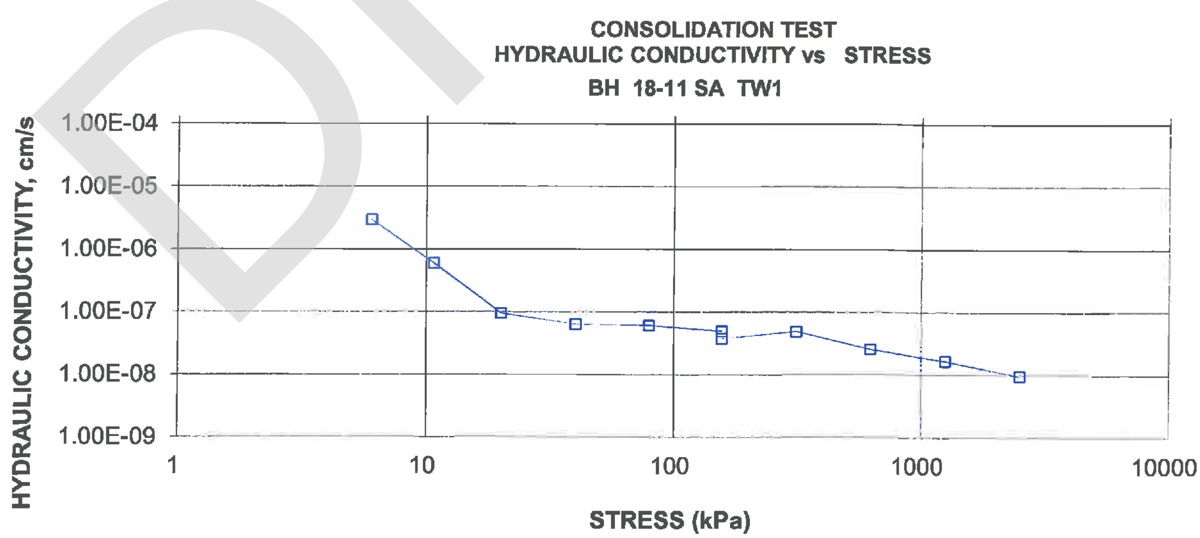
Golder Associates

Checked By: [signature]

CONSOLIDATION TEST SUMMARY	FIGURE







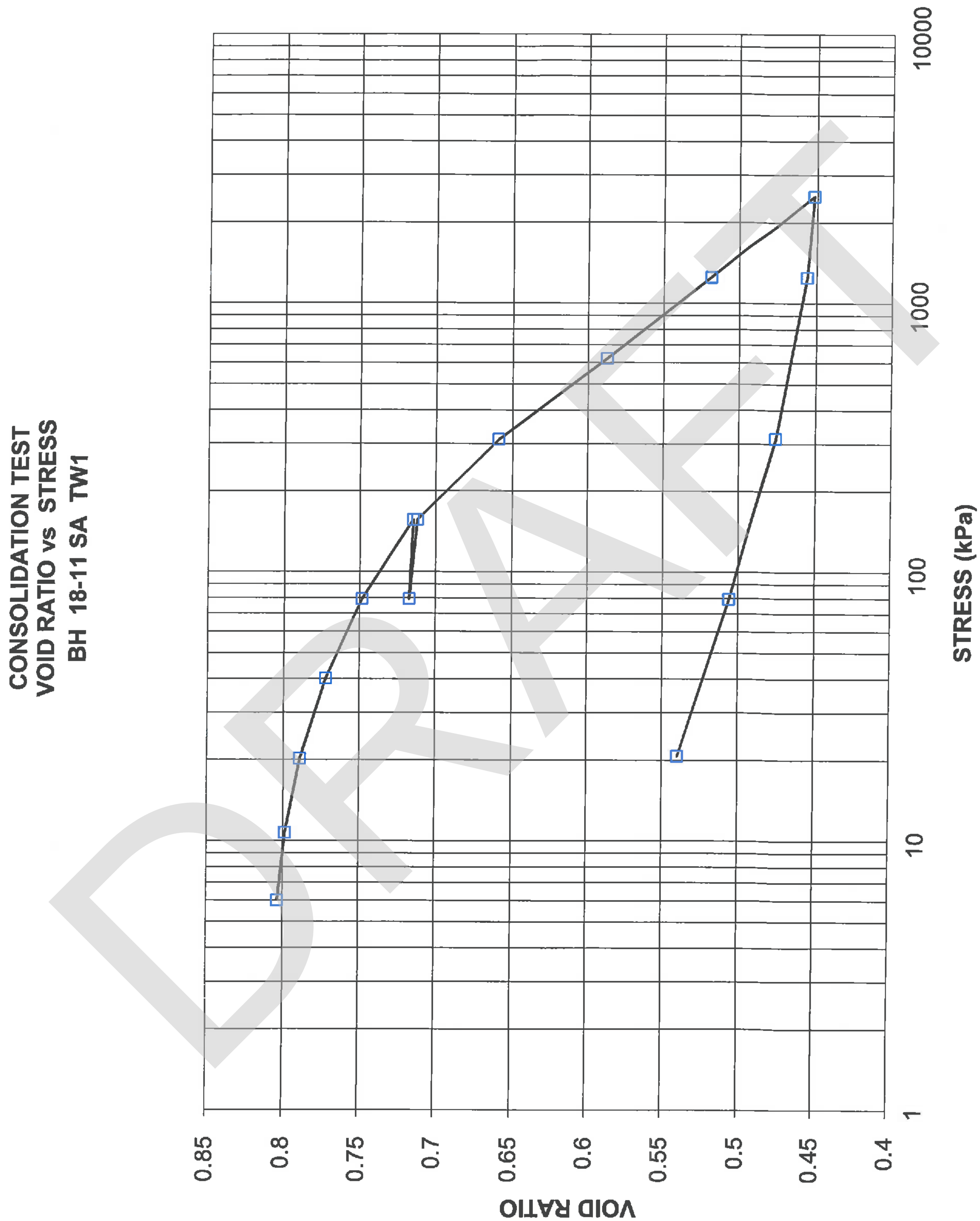
Project No. 1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE



Project No. 1897138(2000)

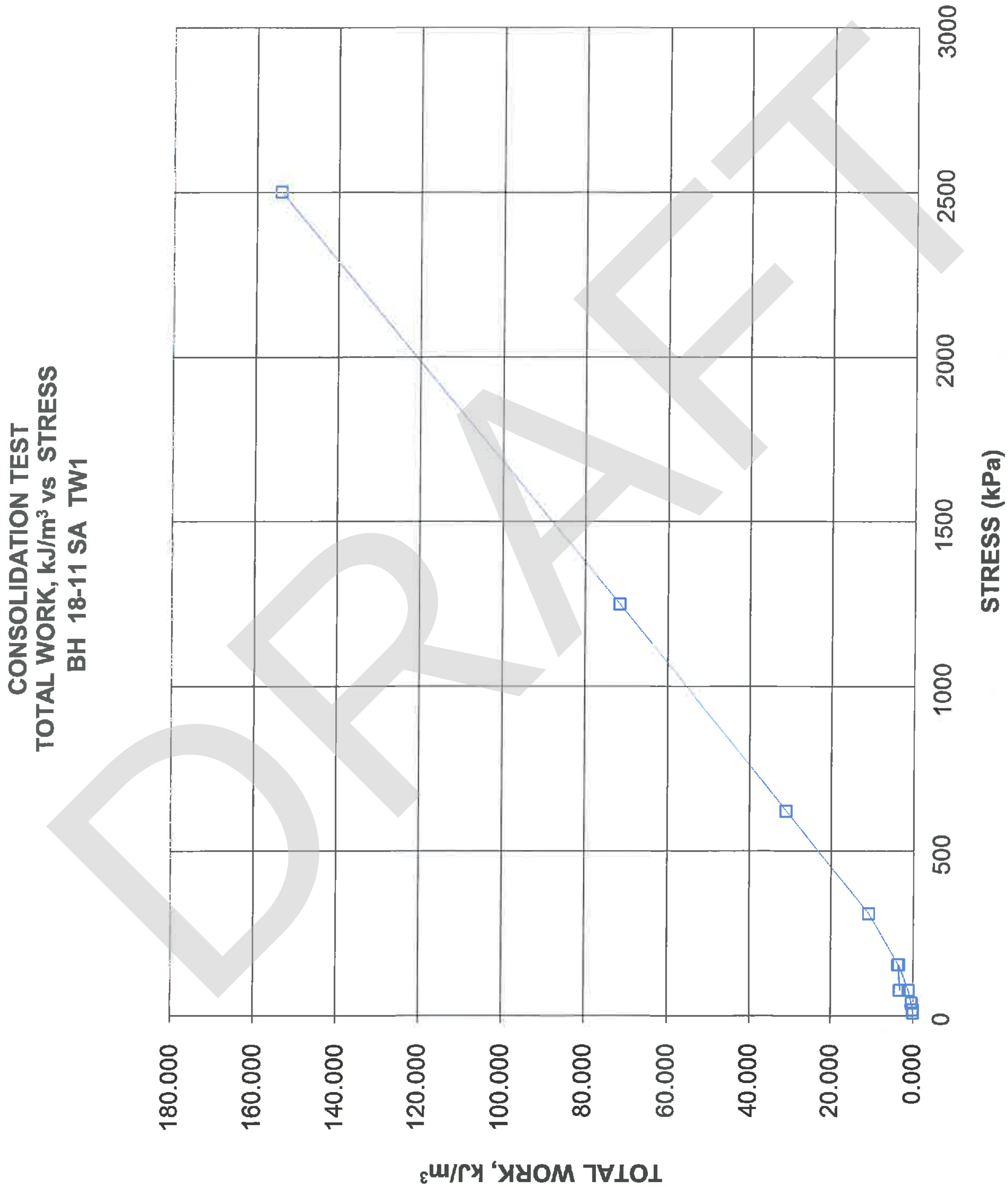
Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE



Project No. 1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

SUMMARY OF ATTERBERG LIMITS DETERMINATION

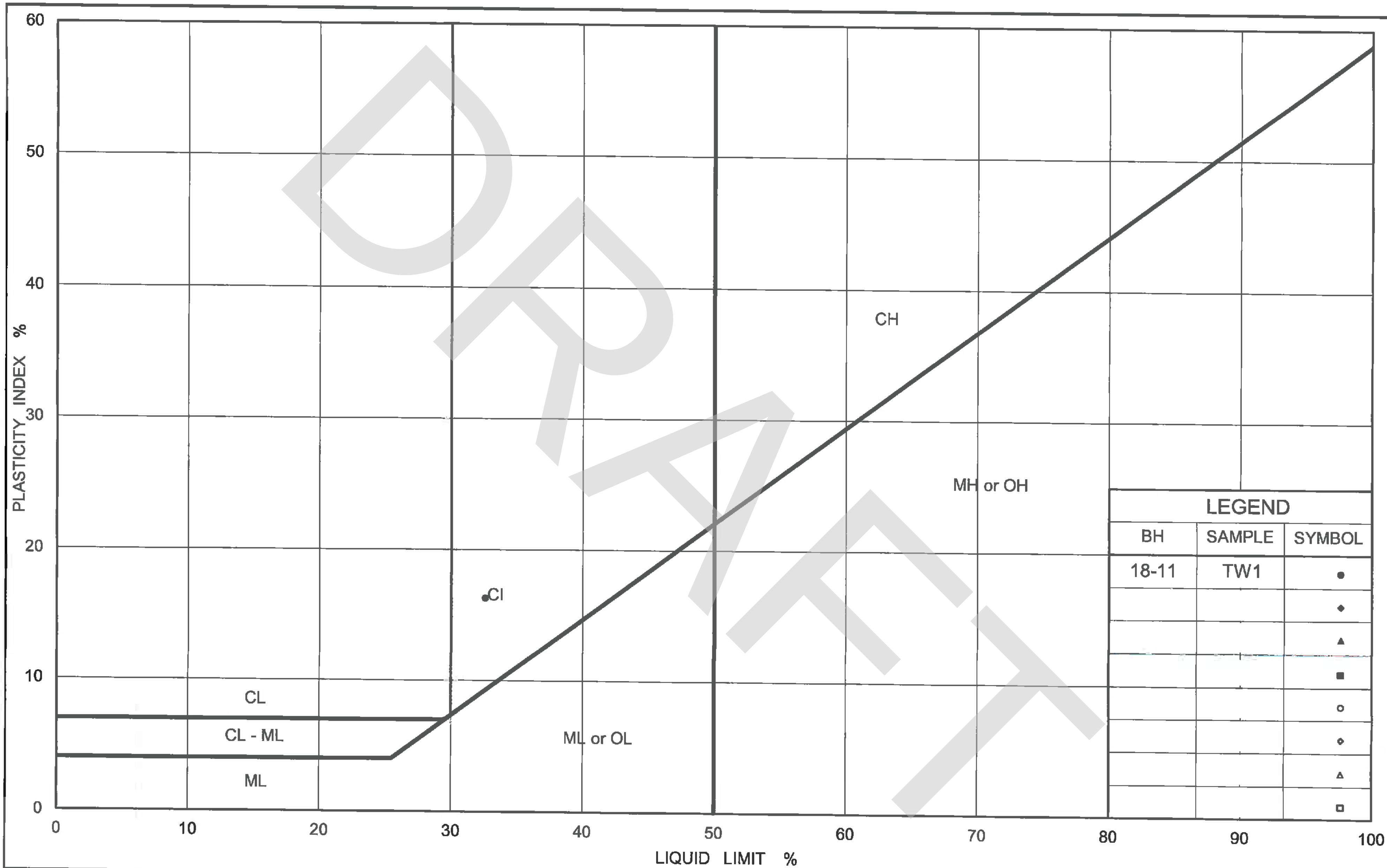
ASTM D 4318

PROJECT NUMBER	1897138 (2000)
PROJECT NAME	ThurberEng/Lab Testing/Miss
DATE TESTED	April 18, 2018

Borehole No.	Sample No.	Depth (ft)	Depth (m)	Atterberg Limits LL=, PL=, PI=
18-11	TW1	22.5-24.5	6.86-7.47	LL=32.6, PL=16.3, PI=16.3

Checked By: LM

Golder Associates



GOLDER

PLASTICITY CHART

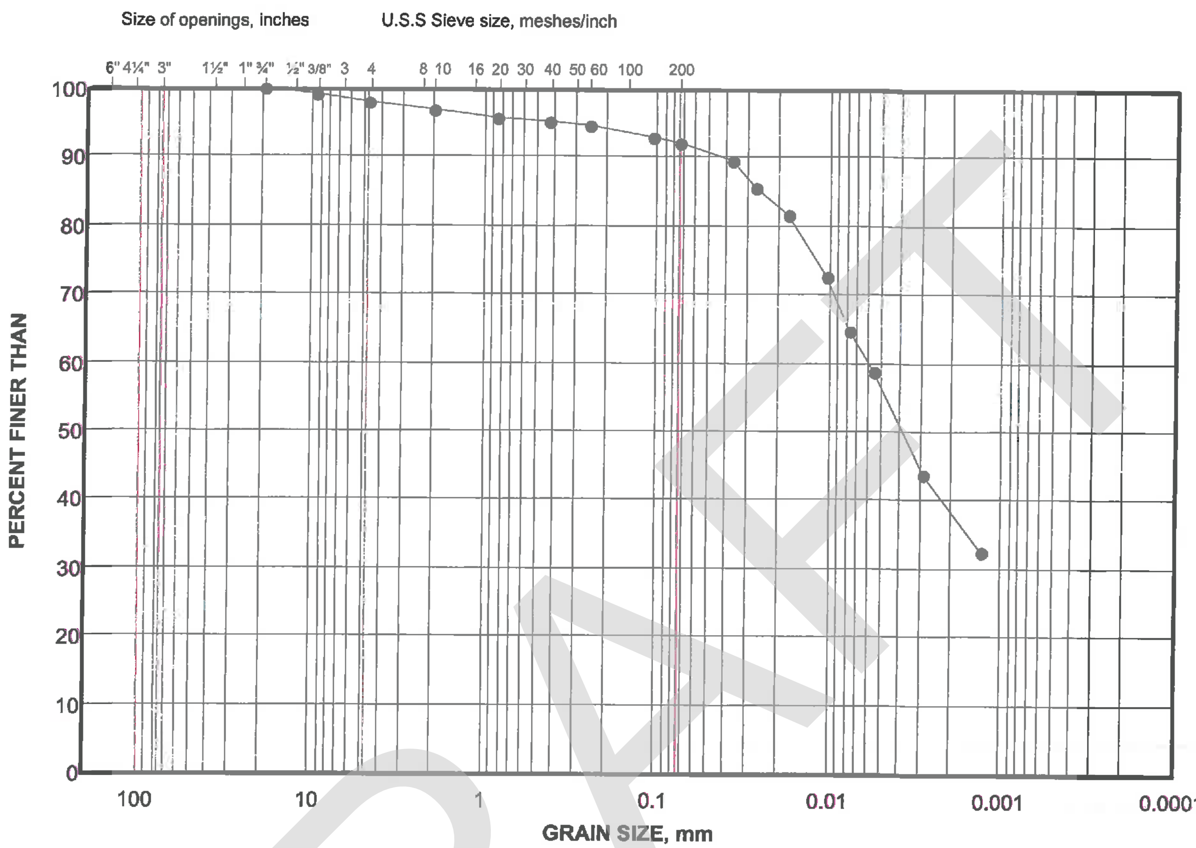
Figure No.

Project No. 1897138 (2000)

Checked By:

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-11	TW1	6.86 - 7.47

Project Number: 1897138

Checked By: [signature]

Golder Associates

Date: 17-Apr-18

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854 TEST METHOD B

PROJECT NUMBER	1897138 (2000)
PROJECT NAME	ThurberEng/Lab Testing/Miss
DATE TESTED	April 11, 2018

Borehole No.	Sample No.	Specific Gravity
18-11	TW1	2.75

Note: Test carried out on soil particles <4.75mm using distilled water.

DRAFT

Checked By: *[signature]*

Golder Associates

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW2
Borehole Number	18-04	Sample Depth, ft	15.24-15.85

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	05/01/2018		
Date Completed	05/16/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.56	Unit Weight, kN/m^3	18.72
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.12
Area, cm^2	31.67	Specific Gravity, measured	2.73
Volume, cm^3	80.91	Solids Height, cm	1.348
Water Content, %	32.57	Volume of Solids, cm^3	42.69
Wet Mass, g	154.50	Volume of Voids, cm^3	38.23
Dry Mass, g	116.54	Degree of Saturation, %	99.3

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.555	0.895	2.555				
5.87	2.559	0.898	2.557				
10.71	2.561	0.900	2.560				
20.45	2.557	0.897	2.559	228	6.09E-03	1.49E-04	8.87E-08
39.71	2.548	0.890	2.552	240	5.75E-03	1.97E-04	1.11E-07
78.56	2.531	0.878	2.539	192	7.12E-03	1.64E-04	1.15E-07
155.86	2.509	0.861	2.520	205	6.57E-03	1.13E-04	7.30E-08
78.56	2.514	0.865	2.511				
155.86	2.506	0.859	2.510	43	3.11E-02	3.90E-05	1.19E-07
310.90	2.467	0.830	2.487	154	8.51E-03	9.85E-05	8.21E-08
620.03	2.349	0.742	2.408	305	4.03E-03	1.50E-04	5.92E-08
1237.01	2.224	0.650	2.286	184	6.02E-03	7.89E-05	4.65E-08
2472.52	2.115	0.569	2.170	154	6.48E-03	3.45E-05	2.19E-08
309.29	2.161	0.603	2.138				
79.21	2.210	0.639	2.185				
20.45	2.264	0.679	2.237				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 10.71kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

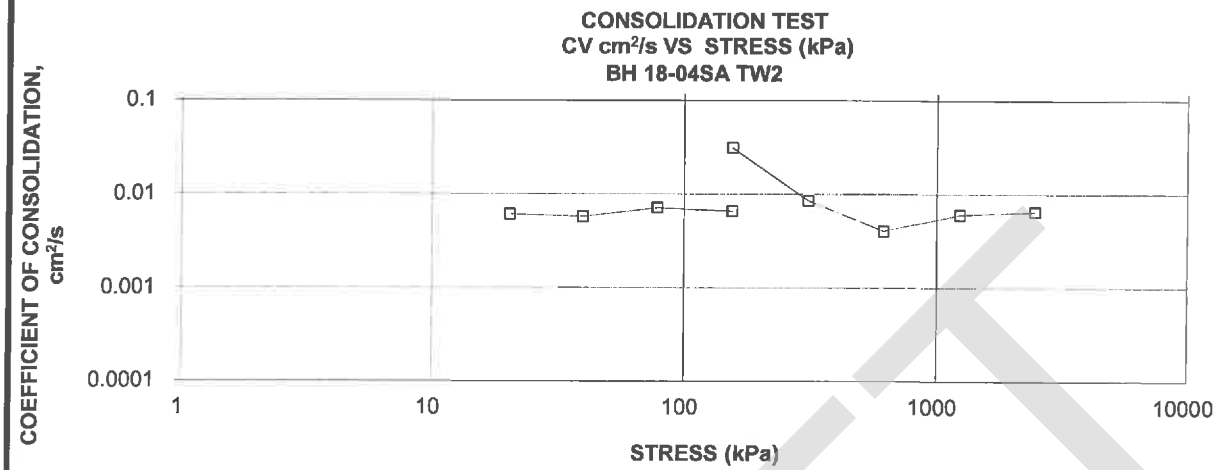
Sample Height, cm	2.26	Unit Weight, kN/m^3	20.04
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.94
Area, cm^2	31.67	Specific Gravity, measured	2.73
Volume, cm^3	71.69	Solids Height, cm	1.348
Water Content, %	25.70	Volume of Solids, cm ^{3}	42.69
Wet Mass, g	146.49	Volume of Voids, cm ^{3}	29.00
Dry Mass, g	116.54		

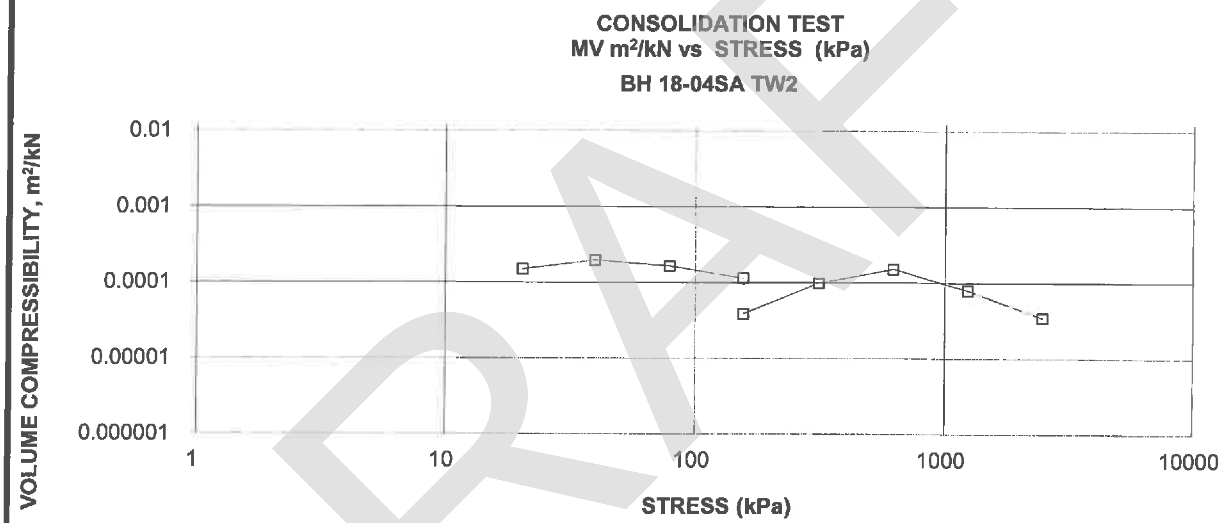
Prepared By: LH

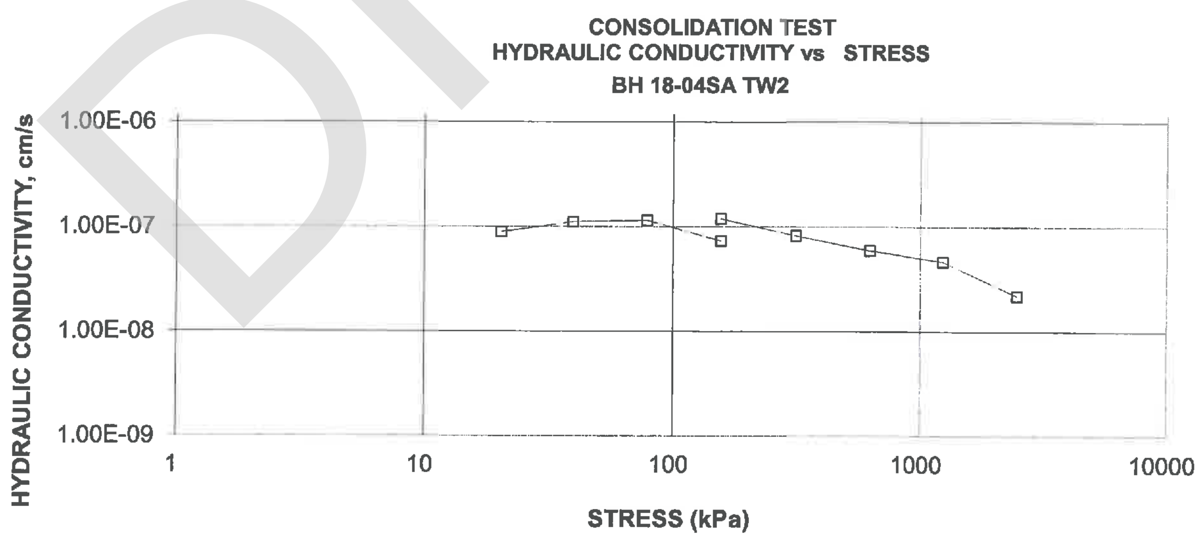
Golder Associates

Checked By:

CONSOLIDATION TEST SUMMARY	FIGURE







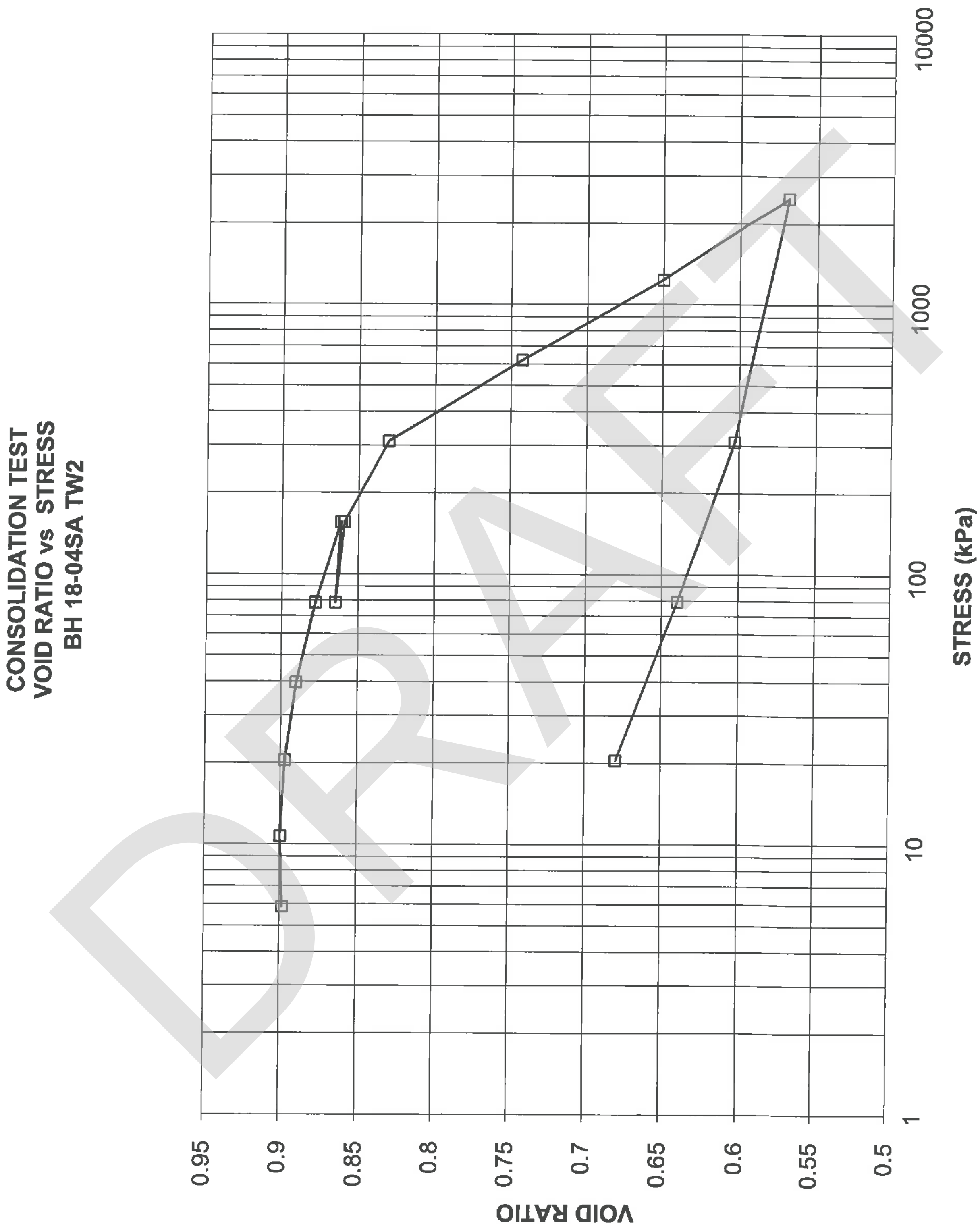
Project No. 1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE



Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST TOTAL WORK VS STRESS	FIGURE

Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW1
Borehole Number	18-24	Sample Depth, ft	12.20-12.80

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	05/01/2018		
Date Completed	05/17/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	19.26
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	14.96
Area, cm^2	31.65	Specific Gravity, measured	2.72
Volume, cm^3	80.29	Solids Height, cm	1.423
Water Content, %	28.74	Volume of Solids, cm^3	45.04
Wet Mass, g	157.71	Volume of Voids, cm^3	35.26
Dry Mass, g	122.5	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.537	0.783	2.537				
5.88	2.527	0.776	2.532	135	1.01E-02	6.44E-04	6.35E-07
10.72	2.519	0.770	2.523	240	5.62E-03	6.84E-04	3.77E-07
20.68	2.504	0.760	2.511	454	2.95E-03	6.02E-04	1.74E-07
40.08	2.484	0.745	2.494	386	3.42E-03	4.10E-04	1.37E-07
78.73	2.458	0.728	2.471	217	5.97E-03	2.57E-04	1.50E-07
156.03	2.428	0.706	2.443	73	1.73E-02	1.57E-04	2.66E-07
310.63	2.387	0.677	2.407	86	1.43E-02	1.04E-04	1.46E-07
156.03	2.390	0.680	2.389				
310.55	2.381	0.673	2.386	54	2.23E-02	2.37E-05	5.20E-08
619.95	2.337	0.642	2.359	147	8.02E-03	5.68E-05	4.47E-08
1238.70	2.275	0.598	2.306	130	8.67E-03	3.94E-05	3.35E-08
2477.61	2.204	0.549	2.239	101	1.05E-02	2.25E-05	2.32E-08
310.55	2.236	0.571	2.220				
78.62	2.267	0.593	2.252				
20.68	2.302	0.617	2.284				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

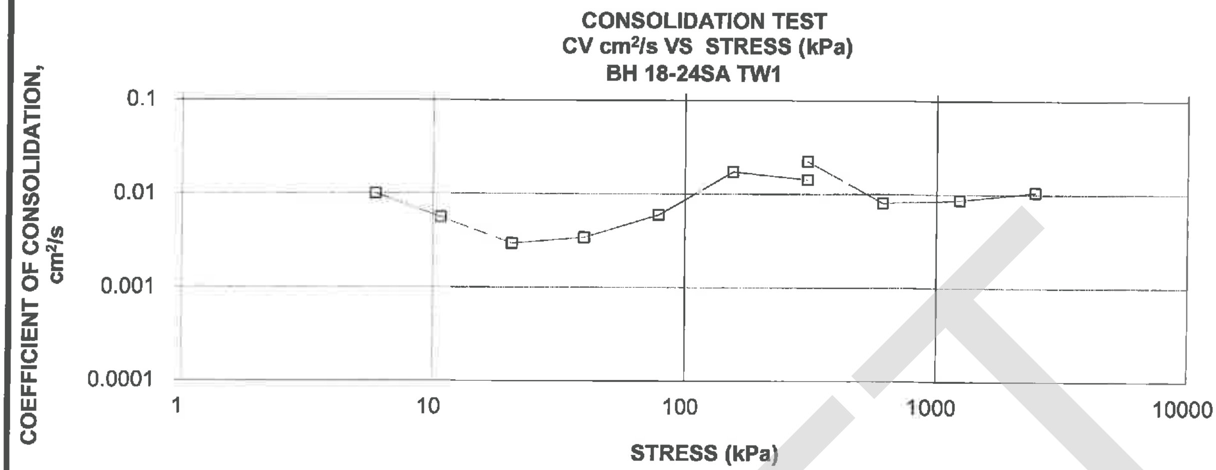
Sample Height, cm	2.30	Unit Weight, kN/m^3	20.39
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.49
Area, cm^2	31.65	Specific Gravity, measured	2.72
Volume, cm^3	72.84	Solids Height, cm	1.423
Water Content, %	23.63	Volume of Solids, cm^3	45.04
Wet Mass, g	151.45	Volume of Voids, cm^3	27.80
Dry Mass, g	122.5		

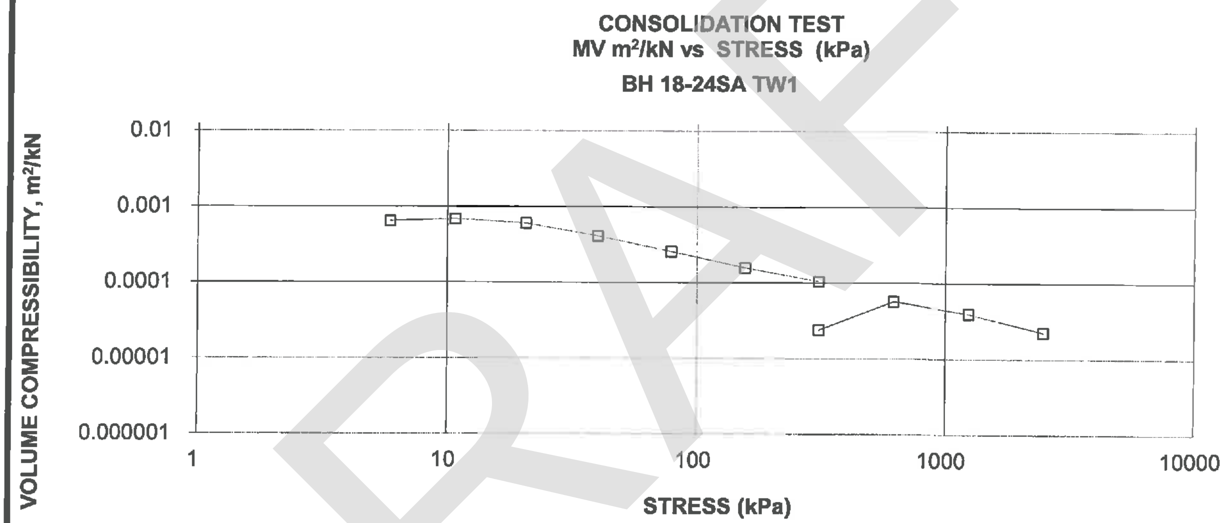
Prepared By: LH

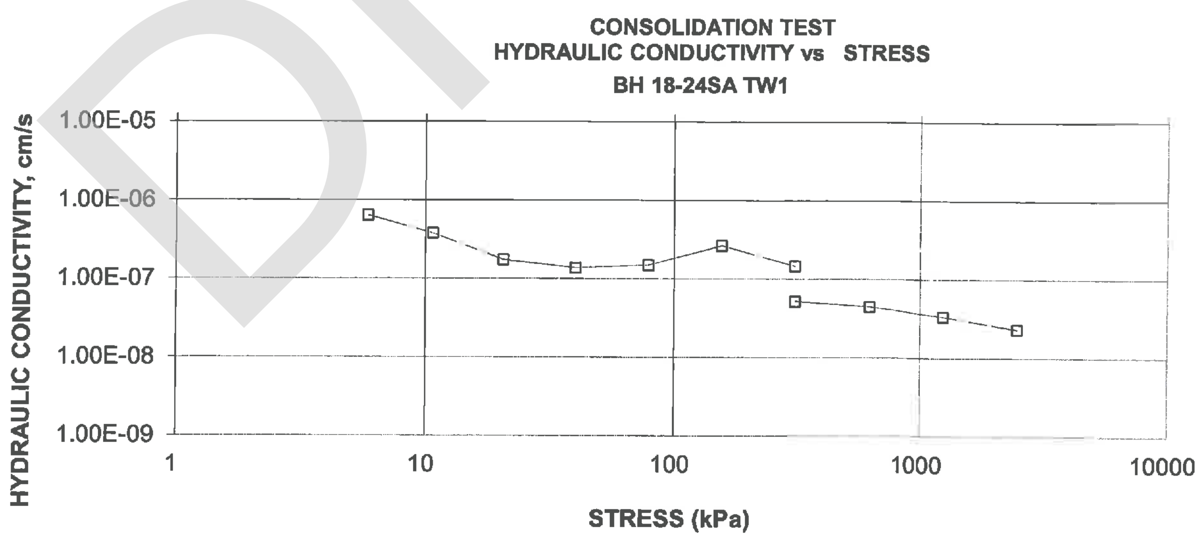
Golder Associates

Checked By: lvb

CONSOLIDATION TEST SUMMARY	FIGURE







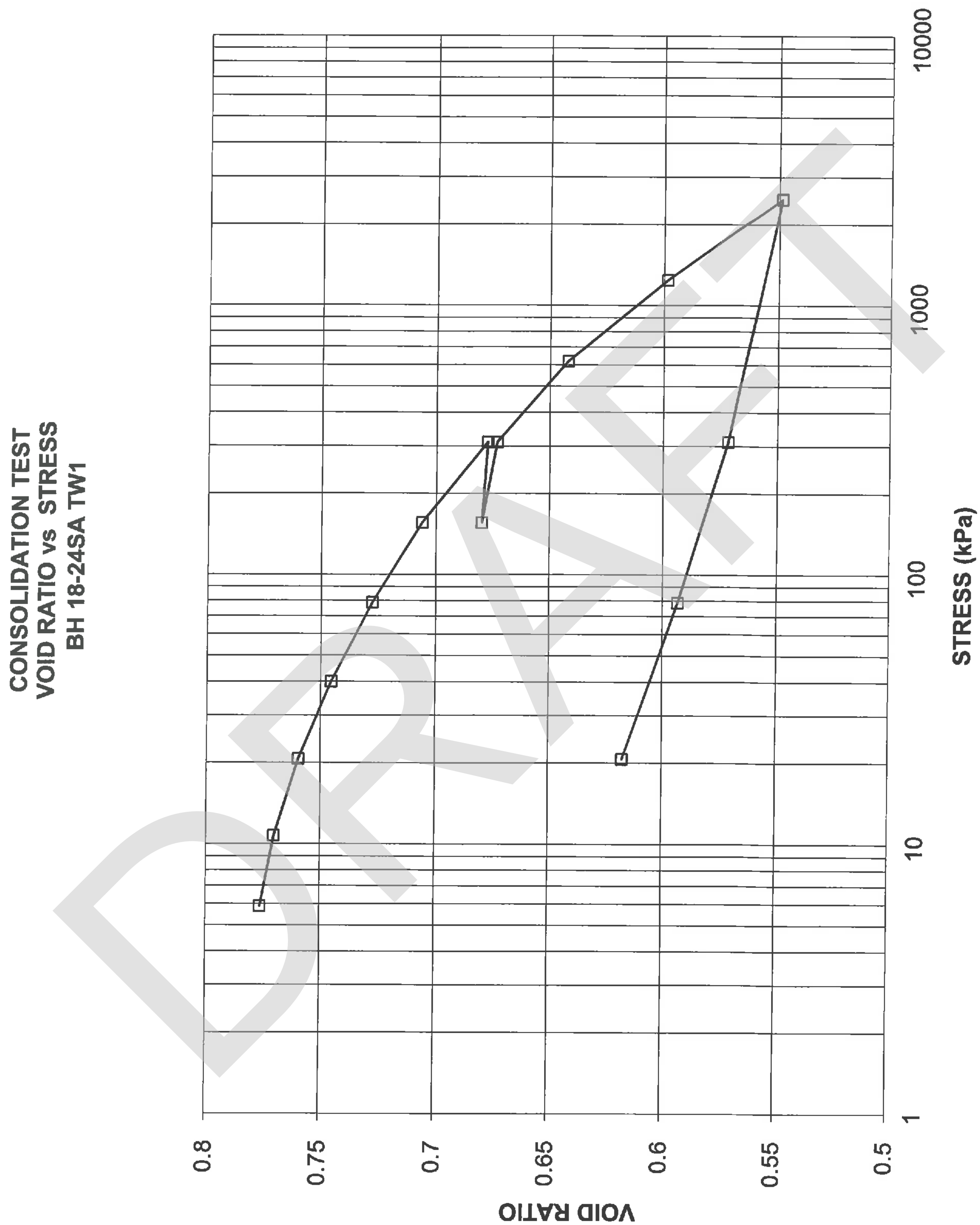
Project No. 1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE



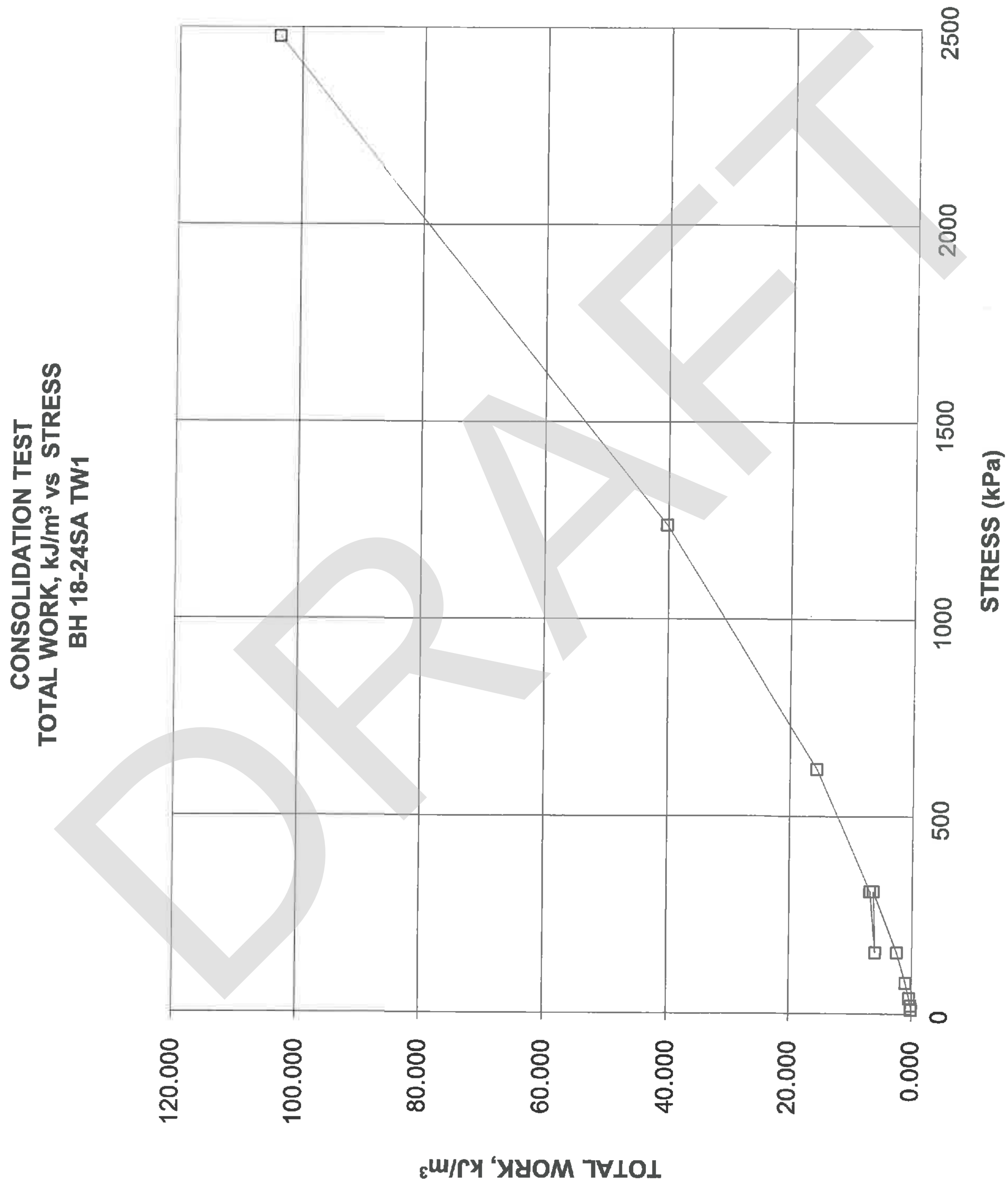
Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST TOTAL WORK VS STRESS	FIGURE



Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

GOLDER

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854 TEST METHOD B

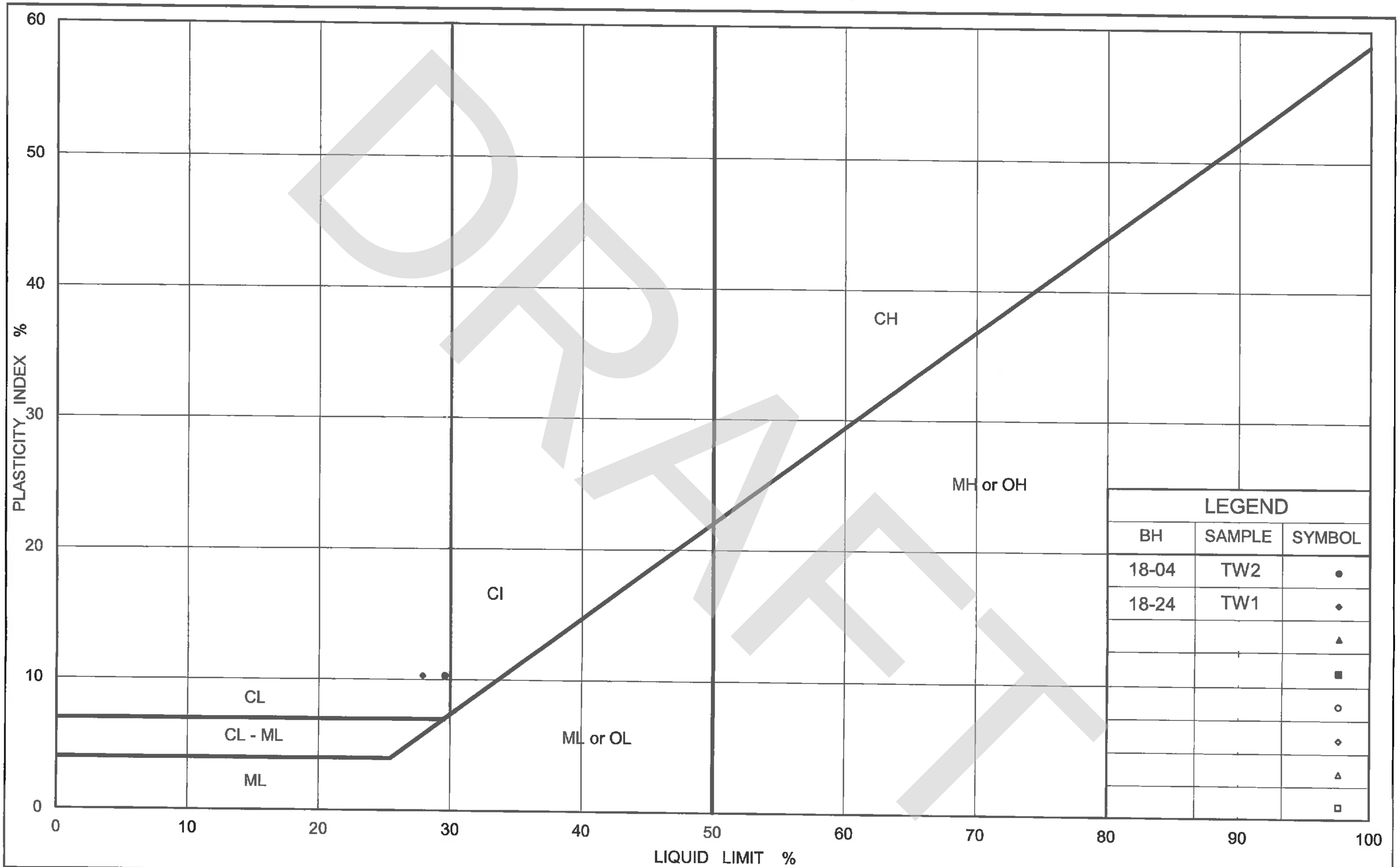
PROJECT NUMBER	1897138 (2000)
PROJECT NAME	ThurberEng/Lab Testing/Miss
DATE TESTED	May 10, 2018

Borehole No.	Sample No.	Specific Gravity
18-04	TW2	2.73
18-24	TW1	2.72

Note: Test carried out on soil particles <4.75mm using distilled water.

Checked By: *[signature]*

Golder Associates



GOLDER

PLASTICITY CHART

Figure No.

Project No. 1897138 (2000)

Checked By:

SUMMARY OF ATTERBERG LIMITS DETERMINATION

ASTM D 4318

PROJECT NUMBER	1897138 (2000)
PROJECT NAME	ThurberEng/Lab Testing/Miss
DATE TESTED	May 29, 2018

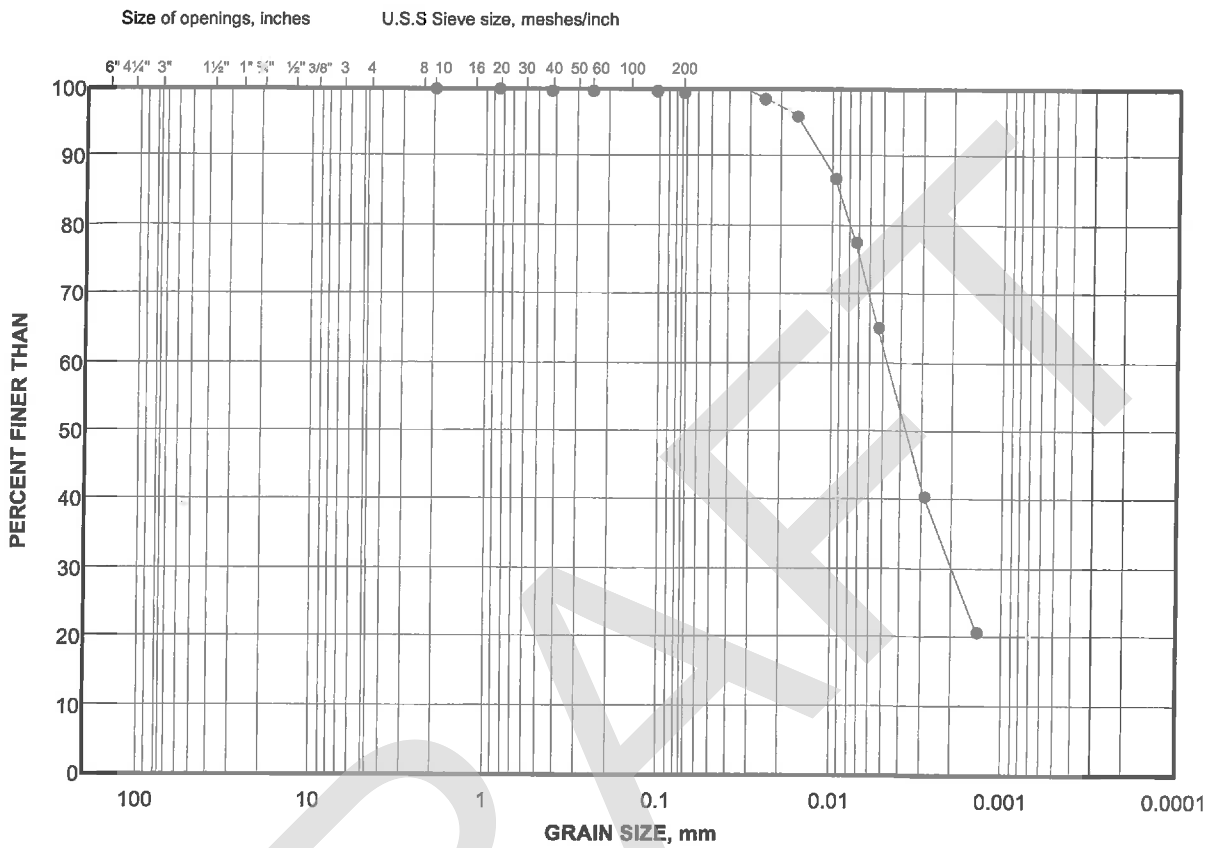
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Atterberg Limits LL=, PL=, PI=
18-04	TW2	50-52	15.24-15.85	LL=29.6, PL=19.3, PI=10.3
18-24	TW1	40-42	12.19-12.80	LL=27.9, PL=17.6, PI=10.3

Checked By: LM

Golder Associates

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-04	TW2	15.24 - 15.85

Project Number: 1897138

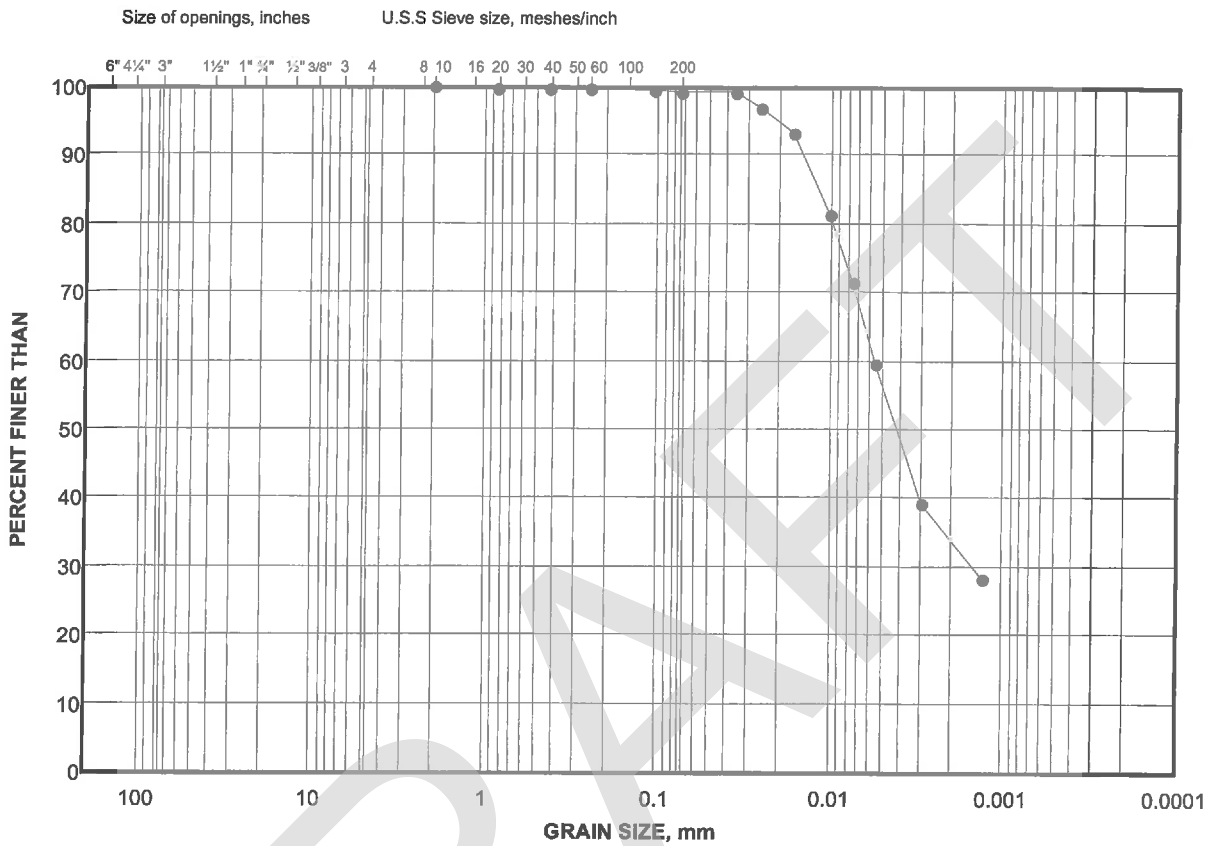
Checked By: LM

Golder Associates

Date: 29-May-18

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-24	TW1	12.20 - 12.80

Project Number: 1897138

Checked By: [signature]

Golder Associates

Date: 29-May-18

GOLDER

SPECIFIC GRAVITY TEST RESULTS

ASTM D 854 TEST METHOD B

PROJECT NUMBER	1897138 (2000)
PROJECT NAME	ThurberEng/Lab Testing/Miss
DATE TESTED	August 3, 2018

Borehole No.	Sample No.	Specific Gravity
18-01	TW1	2.79
18-21	TW2	2.77
18-25	TW1	2.76
18-19	TW1	2.76

Note: Test carried out on soil particles <4.75mm using distilled water.

Checked By: [signature]

Golder Associates

SUMMARY OF ATTERBERG LIMITS DETERMINATION

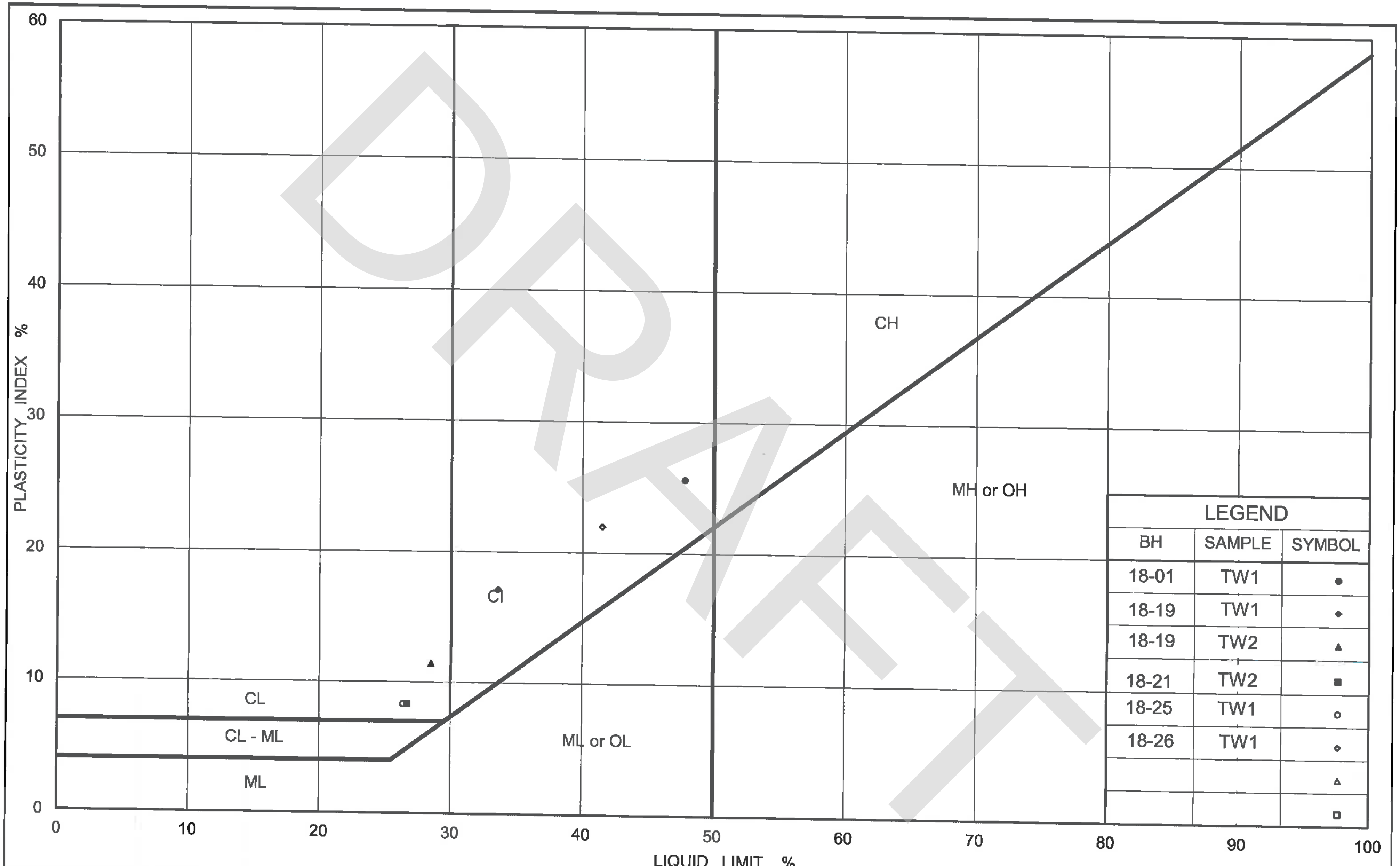
ASTM D 4318

PROJECT NUMBER	1897138 (2000)
PROJECT NAME	ThurberEng/Lab Testing/Miss
DATE TESTED	Aug 2, 2018

Borehole No.	Sample No.	Depth (ft)	Depth (m)	Atterberg Limits LL=, PL=, PI=
18-01	TW1	40-42	12.19-12.80	LL=47.8, PL=22.2, PI=25.6
18-19	TW1	30-32	9.14-9.75	LL=33.6, PL=16.5, PI=17.1
18-19	TW2	45-47	13.72-14.33	LL=28.5, PL=17.1, PI=11.4
18-21	TW2	45-47	13.72-14.33	LL=26.7, PL=18.4, PI=8.3
18-25	TW1	40-42	12.19-12.80	LL=26.4, PL=18.1, PI=8.3
18-26	TW1	25-27	7.62-8.23	LL=41.5, PL=19.5, PI=22.0

Checked By: [signature]

Golder Associates



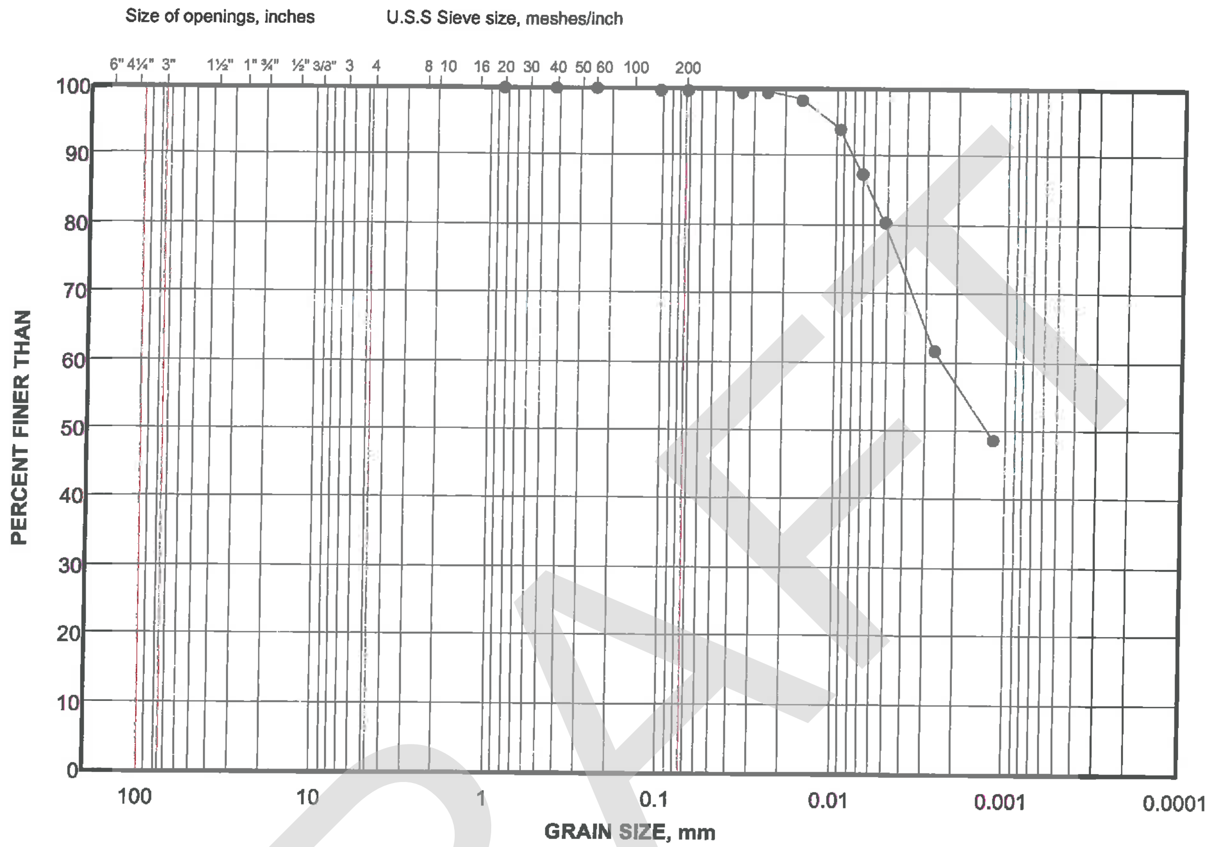
GOLDER

PLASTICITY CHART

Figure No.	
Project No. 1897138 (2000)	
Checked By:	[signature]

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-01	TW1	12.19 - 12.80

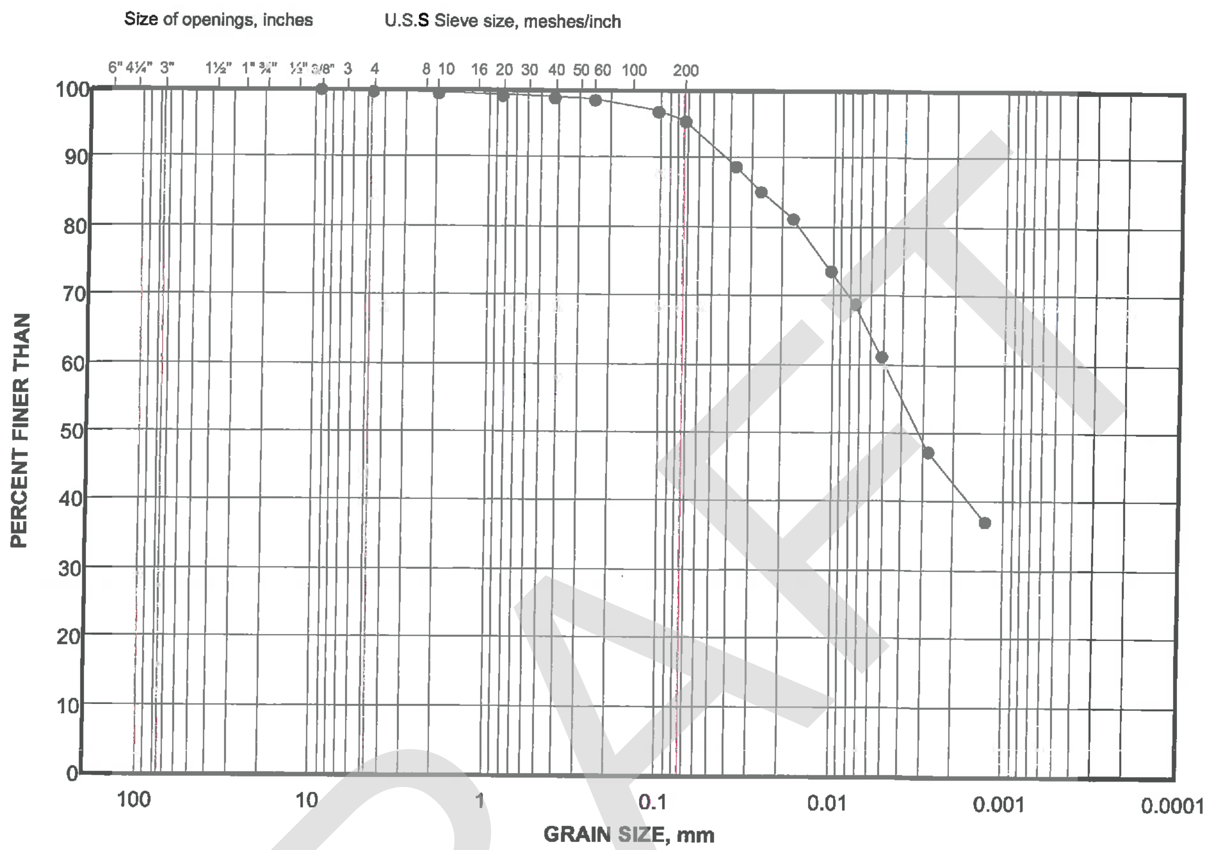
Project Number: 1897138

Checked By: ________

Golder Associates

Date: 08-Aug-18

GRAIN SIZE DISTRIBUTION	FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-19	TW1	9.14 - 9.75

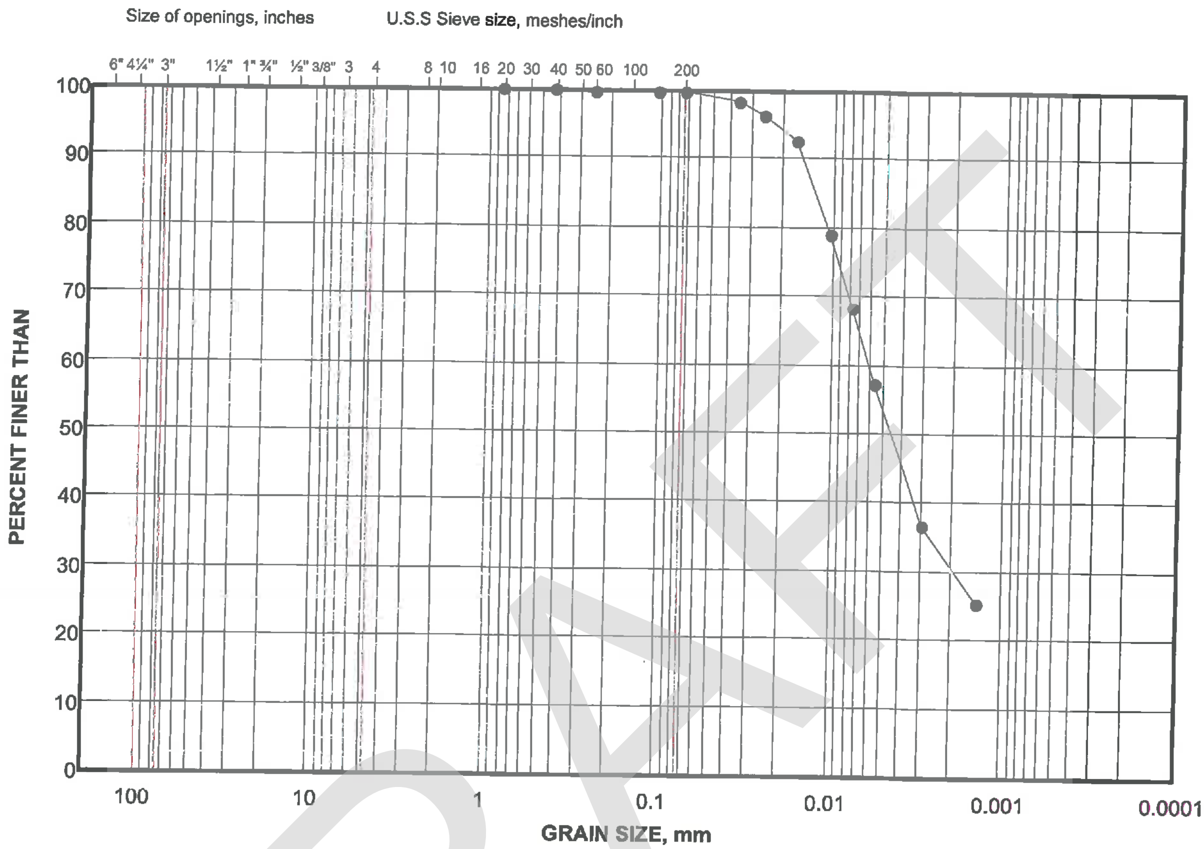
Project Number: 1897138

Checked By: ______

Golder Associates

Date: 08-Aug-18

GRAIN SIZE DISTRIBUTION	FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-19	TW2	13.72 - 14.33

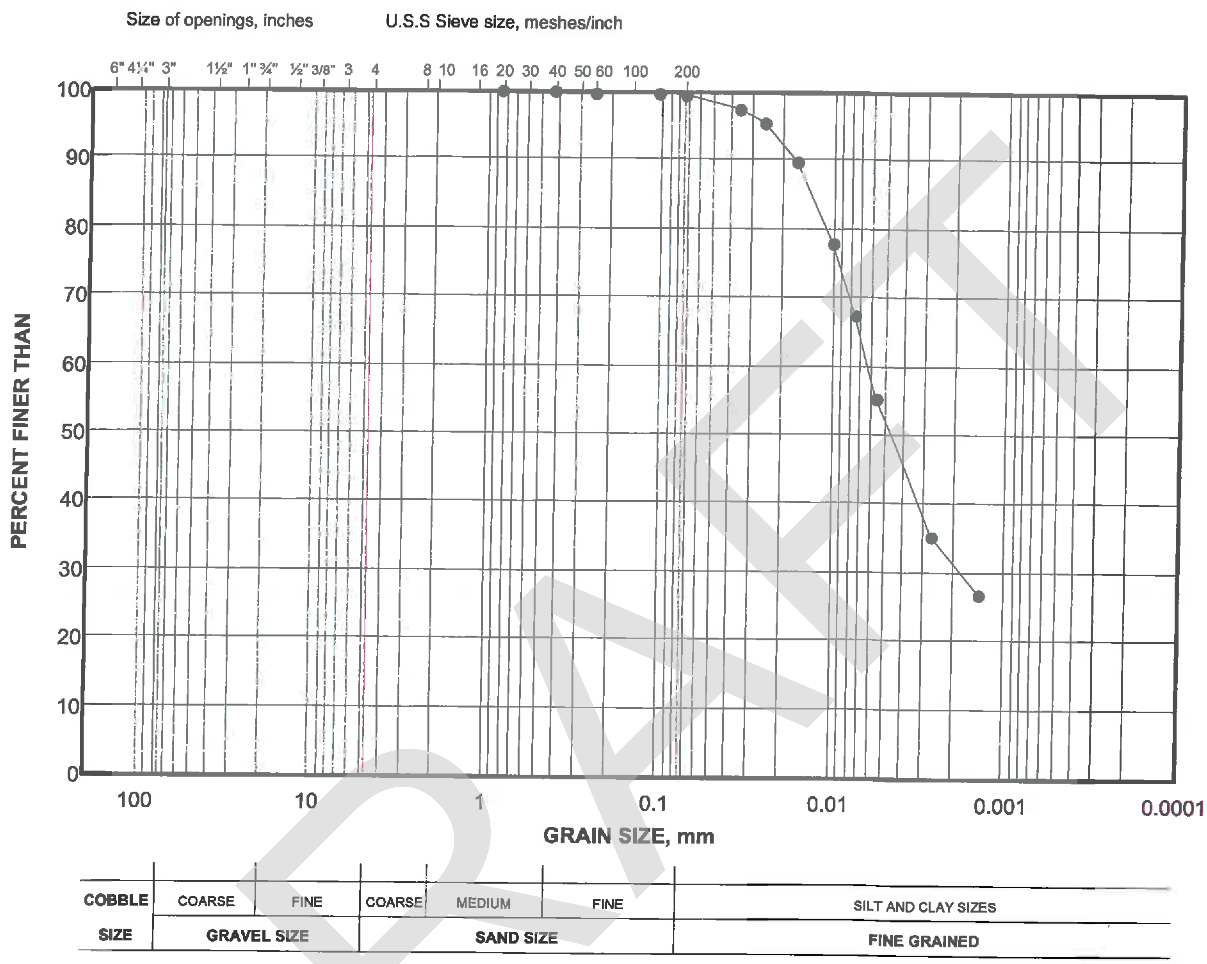
Project Number: 1897138

Checked By: [signature]

Golder Associates

Date: 08-Aug-18

GRAIN SIZE DISTRIBUTION	FIGURE



LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-21	TW2	13.72 - 14.33

Project Number: 1897138

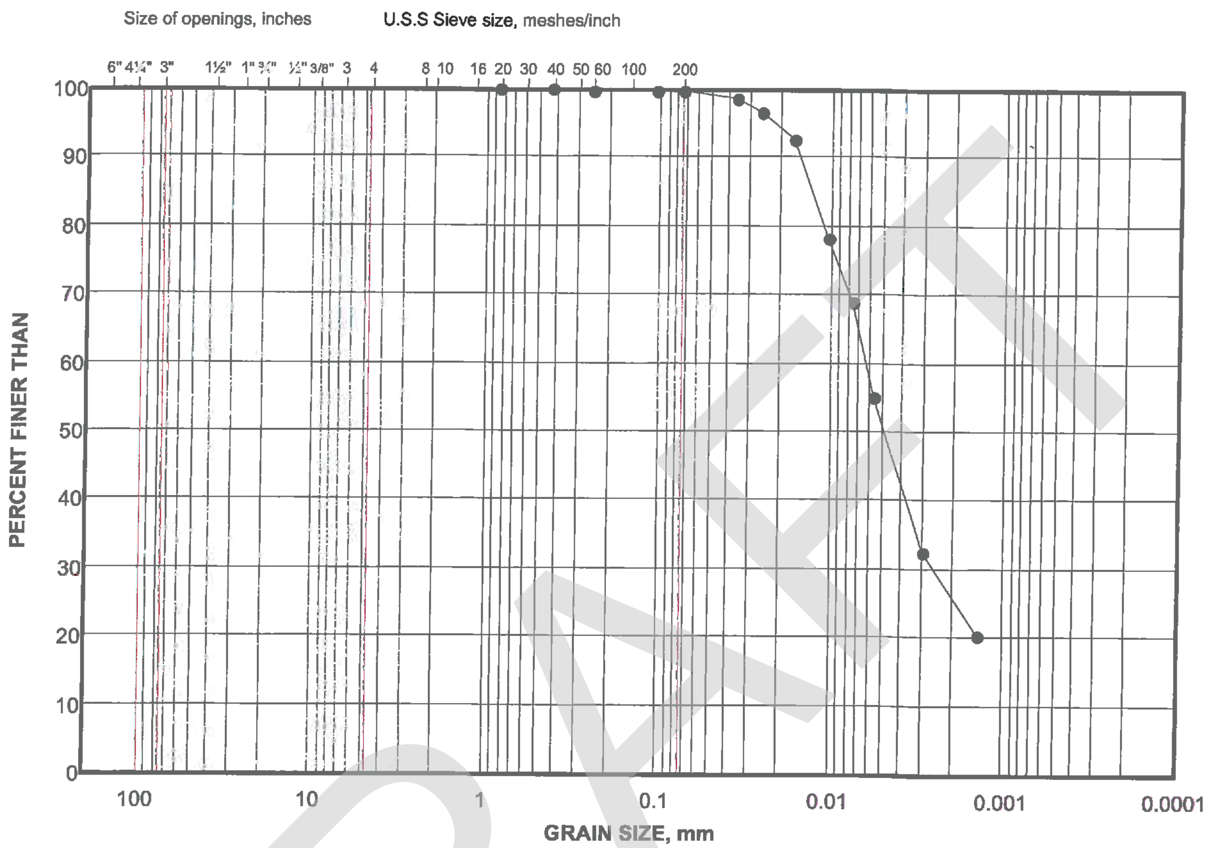
Checked By: ____________

Golder Associates

Date: 08-Aug-18

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-25	TW1	12.19 - 12.80

Project Number: 1897138

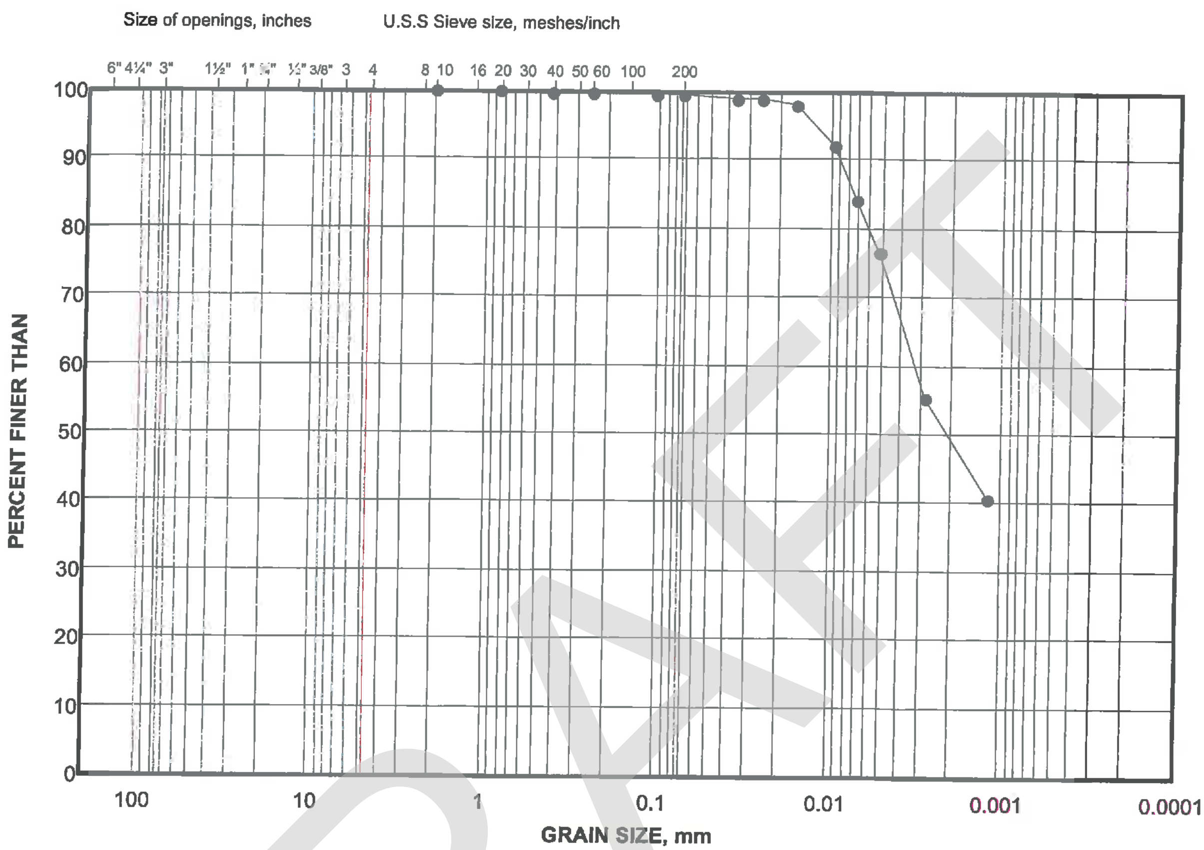
Checked By: ____________

Golder Associates

Date: 08-Aug-18

GRAIN SIZE DISTRIBUTION

FIGURE



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	Borehole	SAMPLE	DEPTH(m)
●	18-26	TW1	7.62 - 8.23

Project Number: 1897138

Checked By: _______________

Golder Associates

Date: 08-Aug-18

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW2
Borehole Number	18-21	Sample Depth, ft	13.72-14.33

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	07/26/2018		
Date Completed	08/10/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	20.25
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	16.36
Area, cm^2	31.50	Specific Gravity, measured	2.77
Volume, cm^3	59.94	Solids Height, cm	1.146
Water Content, %	23.81	Volume of Solids, cm^3	36.09
Wet Mass, g	123.79	Volume of Voids, cm^3	23.85
Dry Mass, g	99.98	Degree of Saturation, %	99.8

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.903	0.661	1.903				
6.49	1.903	0.661	1.903				
11.14	1.901	0.659	1.902	240	3.20E-03	2.03E-04	6.37E-08
21.23	1.895	0.654	1.898	154	4.96E-03	3.12E-04	1.52E-07
40.72	1.887	0.647	1.891	135	5.62E-03	2.26E-04	1.25E-07
79.55	1.874	0.635	1.880	60	1.25E-02	1.73E-04	2.12E-07
157.22	1.859	0.622	1.866	54	1.37E-02	1.03E-04	1.38E-07
79.55	1.859	0.622	1.859				
157.15	1.857	0.621	1.858	54	1.36E-02	1.35E-05	1.80E-08
311.77	1.833	0.600	1.845	49	1.47E-02	8.09E-05	1.17E-07
622.61	1.791	0.563	1.812	73	9.53E-03	7.22E-05	6.74E-08
1245.42	1.738	0.517	1.764	86	7.67E-03	4.40E-05	3.31E-08
2501.44	1.688	0.473	1.713	101	6.16E-03	2.13E-05	1.29E-08
1245.42	1.691	0.476	1.689				
314.13	1.701	0.484	1.696				
79.55	1.711	0.493	1.706				
21.00	1.725	0.506	1.718				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 6.49kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

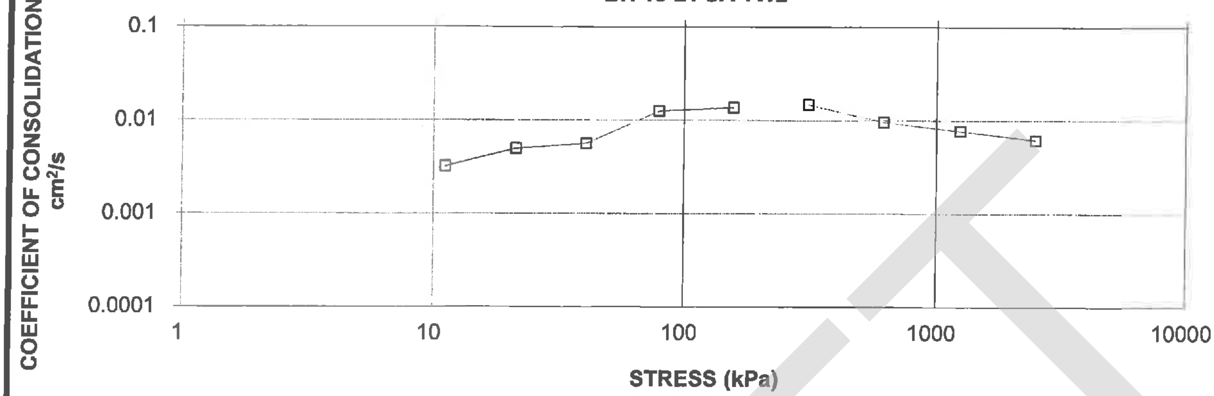
Sample Height, cm	1.73	Unit Weight, kN/m^3	21.55
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	18.04
Area, cm^2	31.50	Specific Gravity, measured	2.77
Volume, cm^3	54.34	Solids Height, cm	1.146
Water Content, %	19.42	Volume of Solids, cm^3	36.09
Wet Mass, g	119.40	Volume of Voids, cm^3	18.25
Dry Mass, g	99.98		

Prepared By: LH Golder Associates Checked By: bb

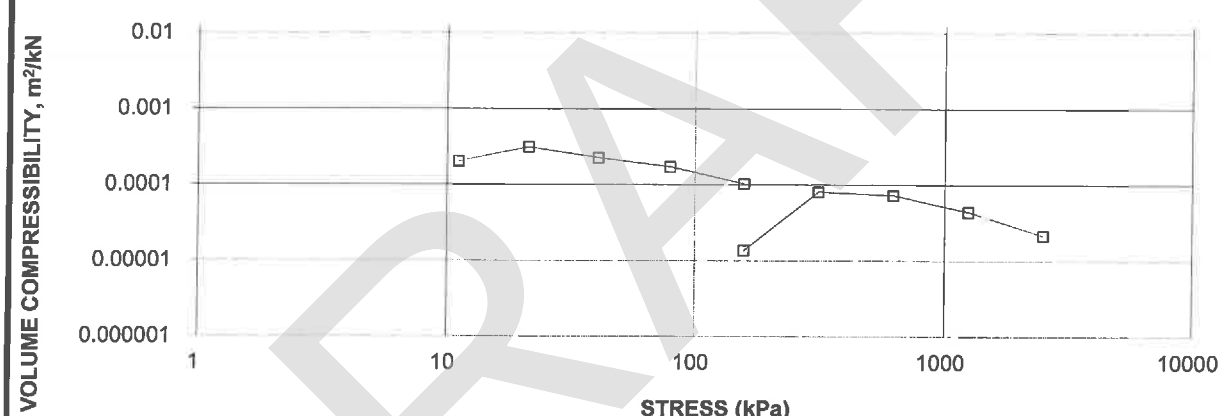
CONSOLIDATION TEST SUMMARY

FIGURE

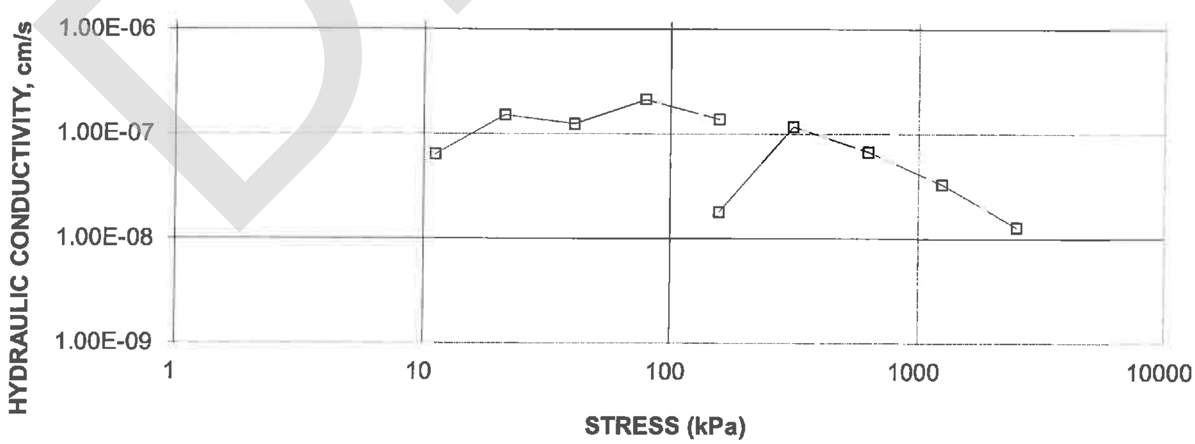
CONSOLIDATION TEST
CV cm^2/s VS STRESS (kPa)
BH 18-21 SA TW2



CONSOLIDATION TEST
MV m^2/kN vs STRESS (kPa)
BH 18-21 SA TW2



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs STRESS
BH 18-21 SA TW2



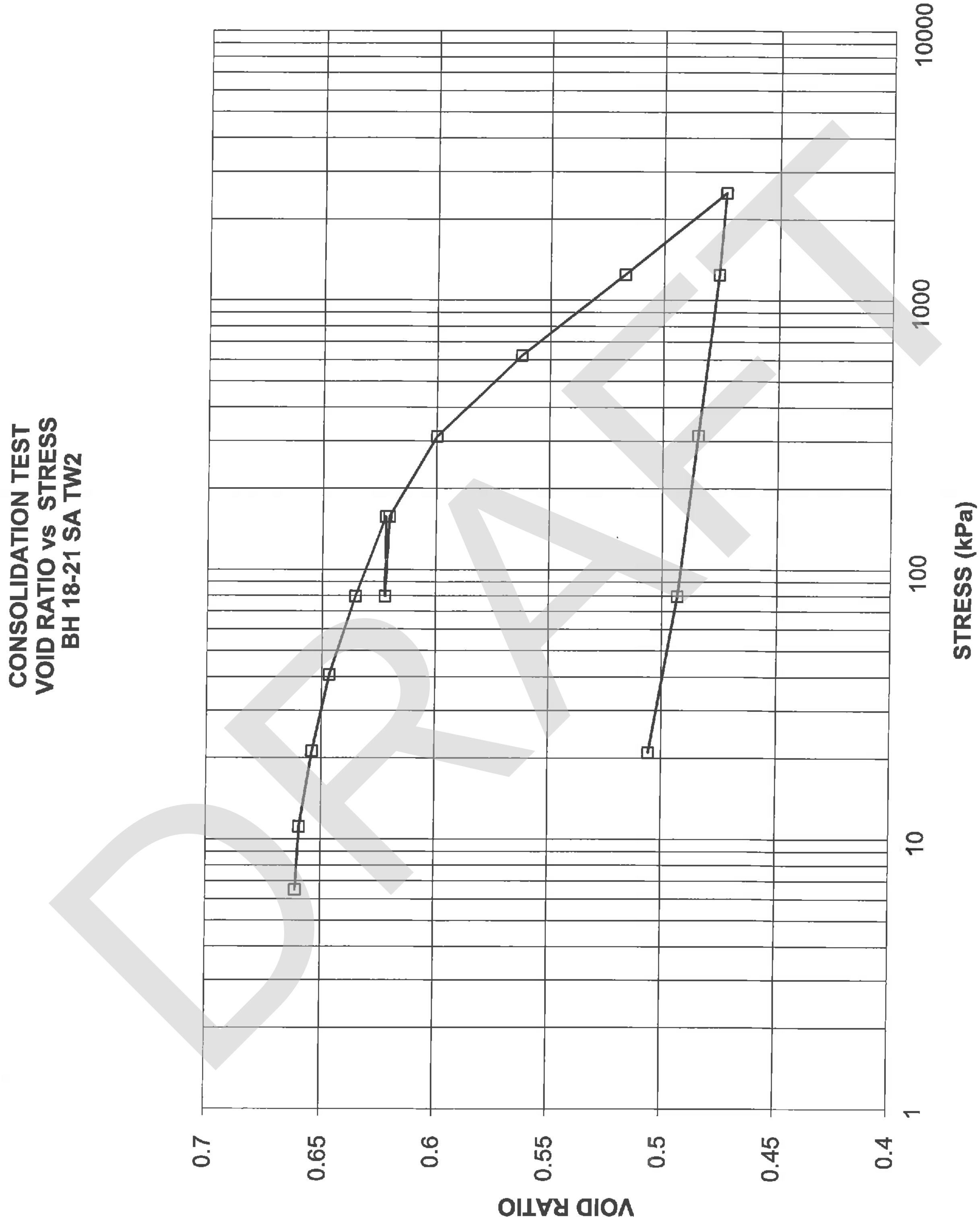
Project No. 1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE



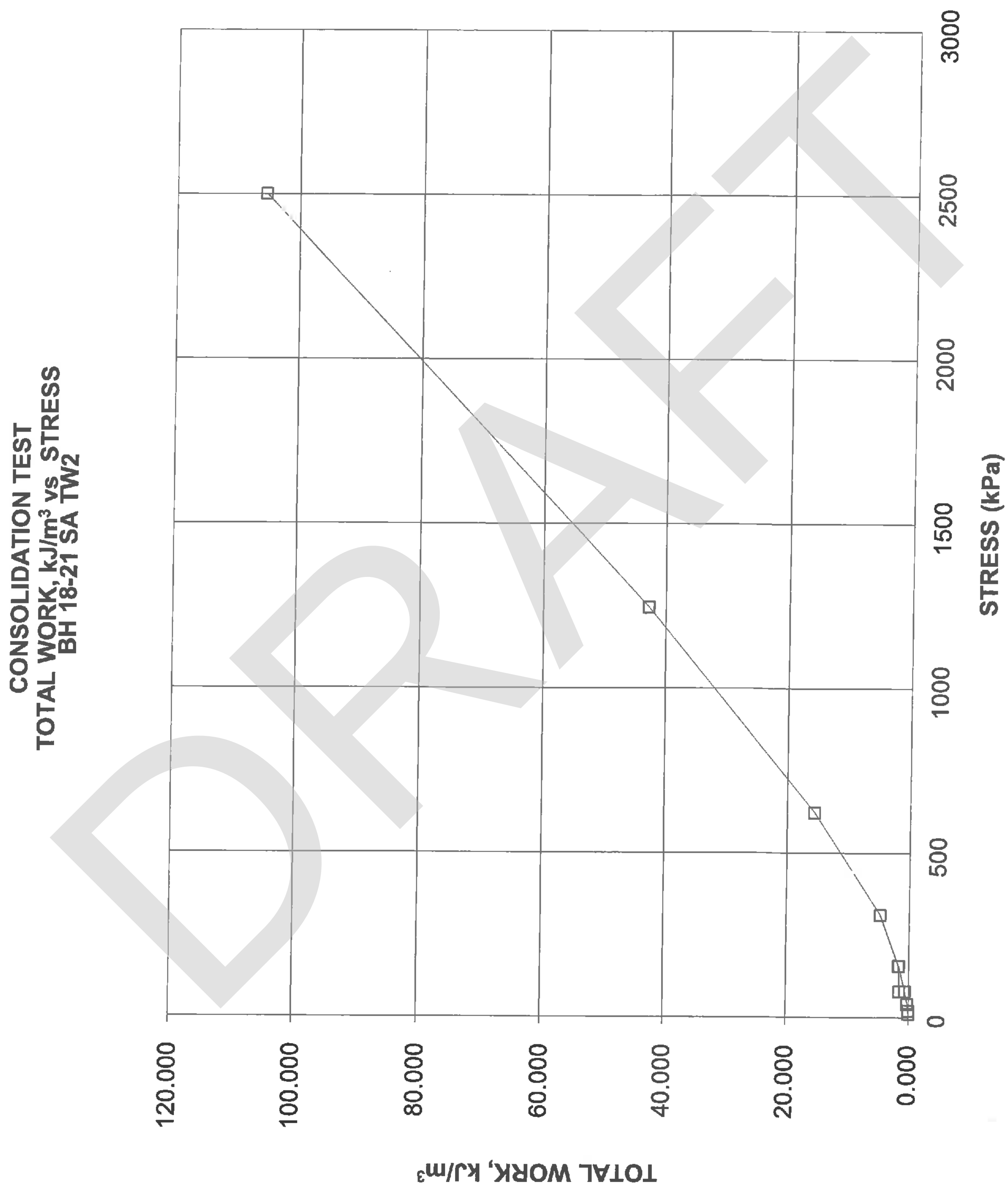
Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST TOTAL WORK VS STRESS	FIGURE



Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST SUMMARY

FIGURE

ASTM D2435/D2435M

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW1
Borehole Number	18-19	Sample Depth, ft	9.15-9.76

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	07/26/2018		
Date Completed	08/09/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	19.66
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	15.63
Area, cm^2	31.60	Specific Gravity, measured	2.76
Volume, cm^3	59.72	Solids Height, cm	1.091
Water Content, %	25.79	Volume of Solids, cm^3	34.49
Wet Mass, g	119.74	Volume of Voids, cm^3	25.23
Dry Mass, g	95.19	Degree of Saturation, %	97.3

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.890	0.732	1.890				
5.88	1.899	0.740	1.895				
10.72	1.902	0.743	1.901				
20.45	1.903	0.744	1.903				
39.92	1.902	0.743	1.903	505	1.52E-03	2.16E-05	3.22E-09
78.10	1.897	0.738	1.900	1270	6.02E-04	6.94E-05	4.10E-09
151.26	1.885	0.727	1.891	987	7.68E-04	8.78E-05	6.61E-09
78.70	1.885	0.727	1.885				
152.21	1.883	0.725	1.884	154	4.89E-03	1.44E-05	6.89E-09
311.21	1.871	0.714	1.877	554	1.35E-03	3.94E-05	5.20E-09
621.14	1.845	0.690	1.858	390	1.88E-03	4.45E-05	8.19E-09
1240.72	1.807	0.656	1.826	290	2.44E-03	3.26E-05	7.78E-09
2501.07	1.749	0.603	1.778	346	1.94E-03	2.42E-05	4.59E-09
1250.96	1.751	0.604	1.750				
310.87	1.774	0.625	1.762				
78.70	1.806	0.655	1.790				
20.45	1.844	0.689	1.825				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 20.45kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

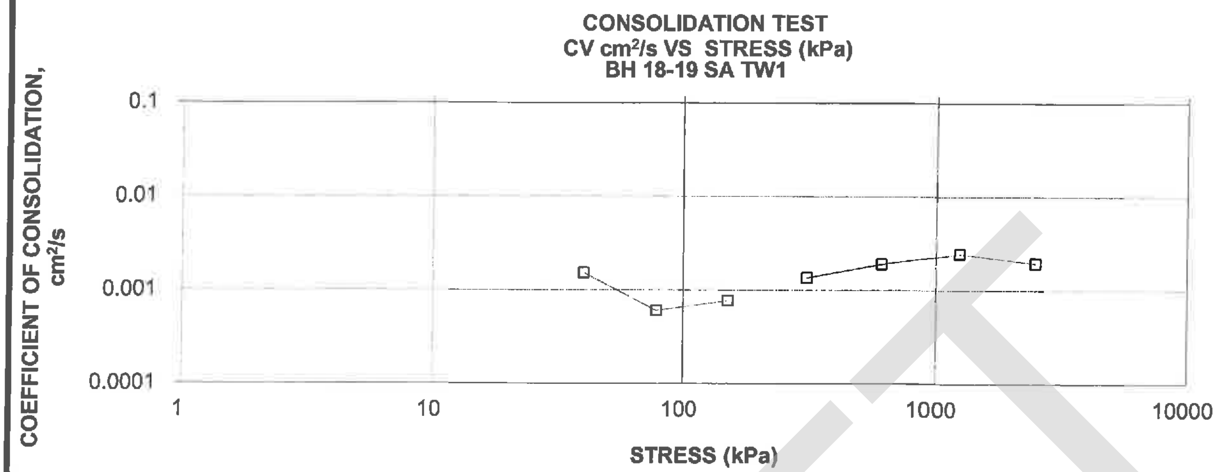
Sample Height, cm	1.84	Unit Weight, kN/m^3	20.32
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.02
Area, cm^2	31.60	Specific Gravity, measured	2.76
Volume, cm^3	58.27	Solids Height, cm	1.091
Water Content, %	26.86	Volume of Solids, cm^3	34.49
Wet Mass, g	120.76	Volume of Voids, cm^3	23.78
Dry Mass, g	95.19		

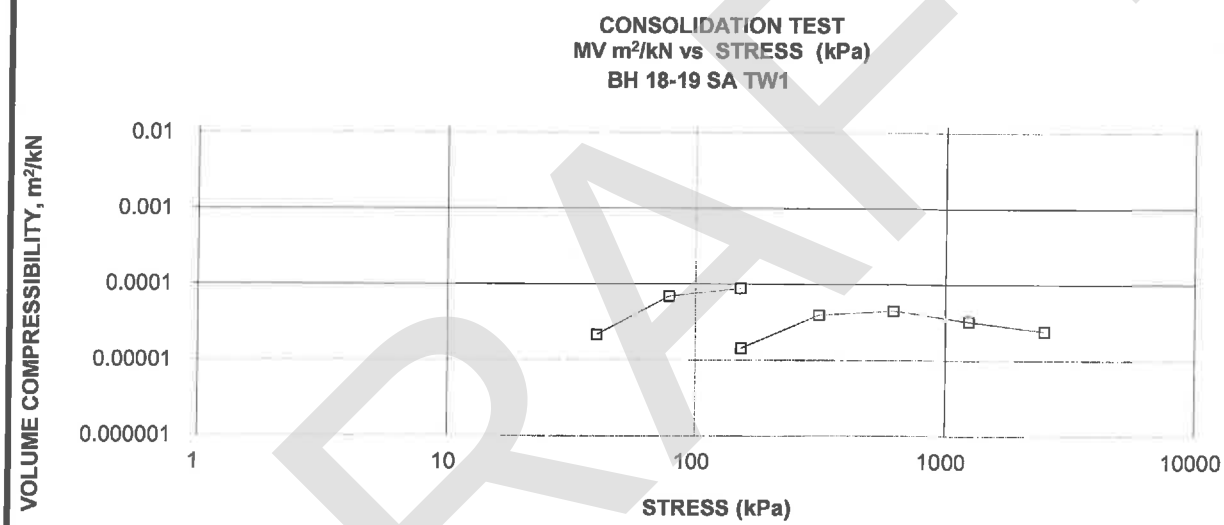
Prepared By: LH

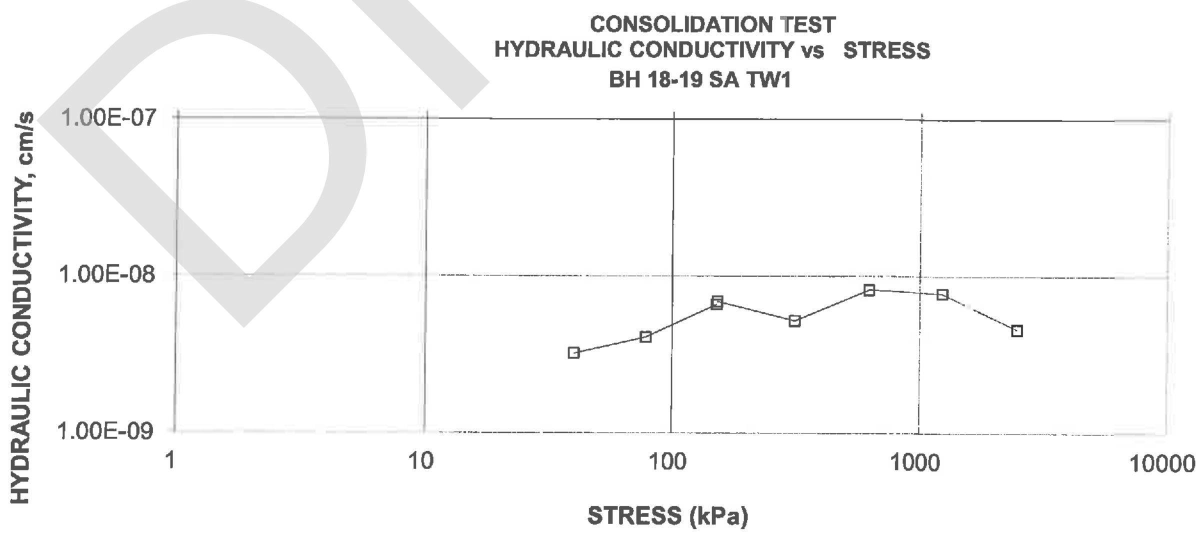
Golder Associates

Checked By: [signature]

CONSOLIDATION TEST SUMMARY	FIGURE







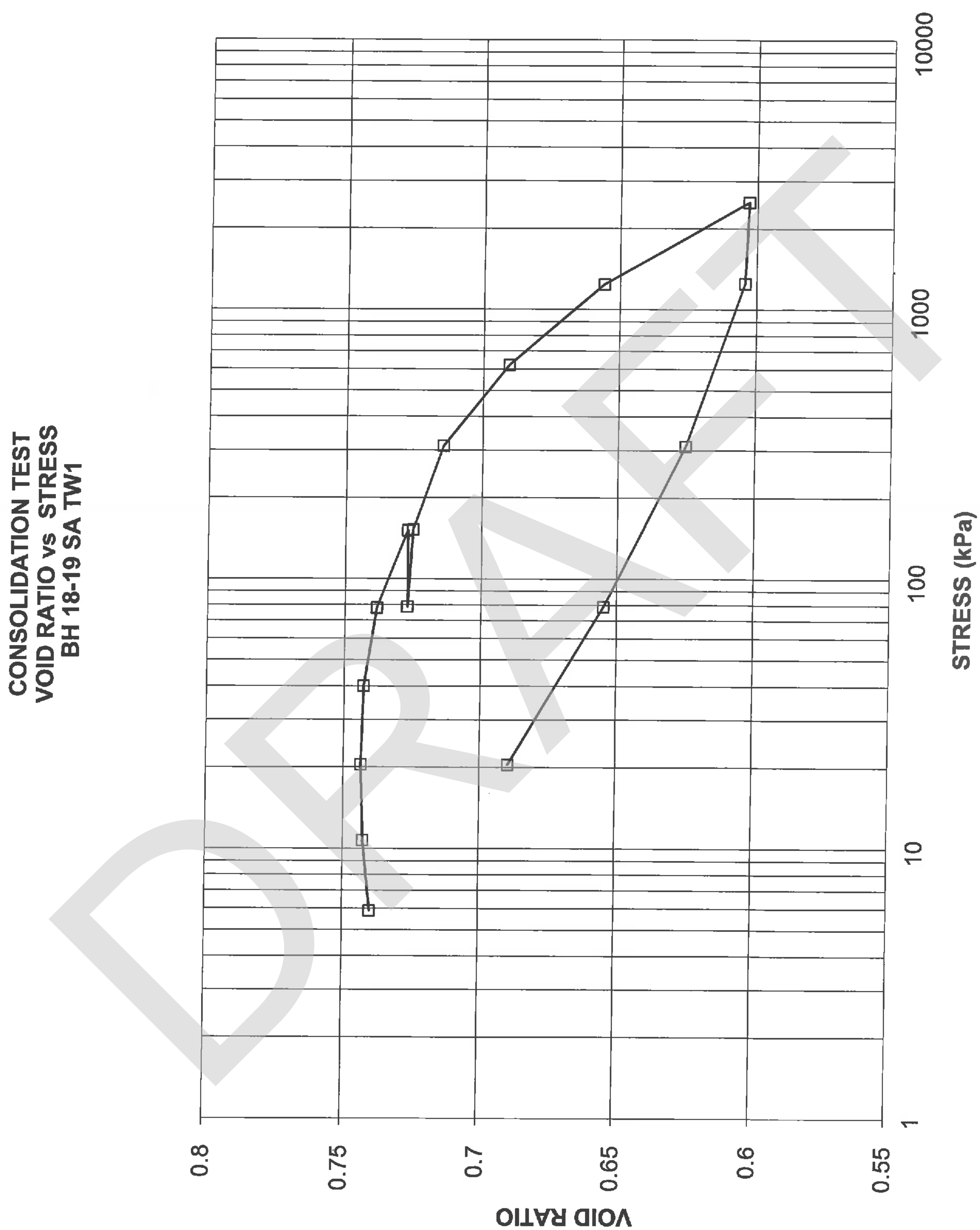
Project No. 1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE



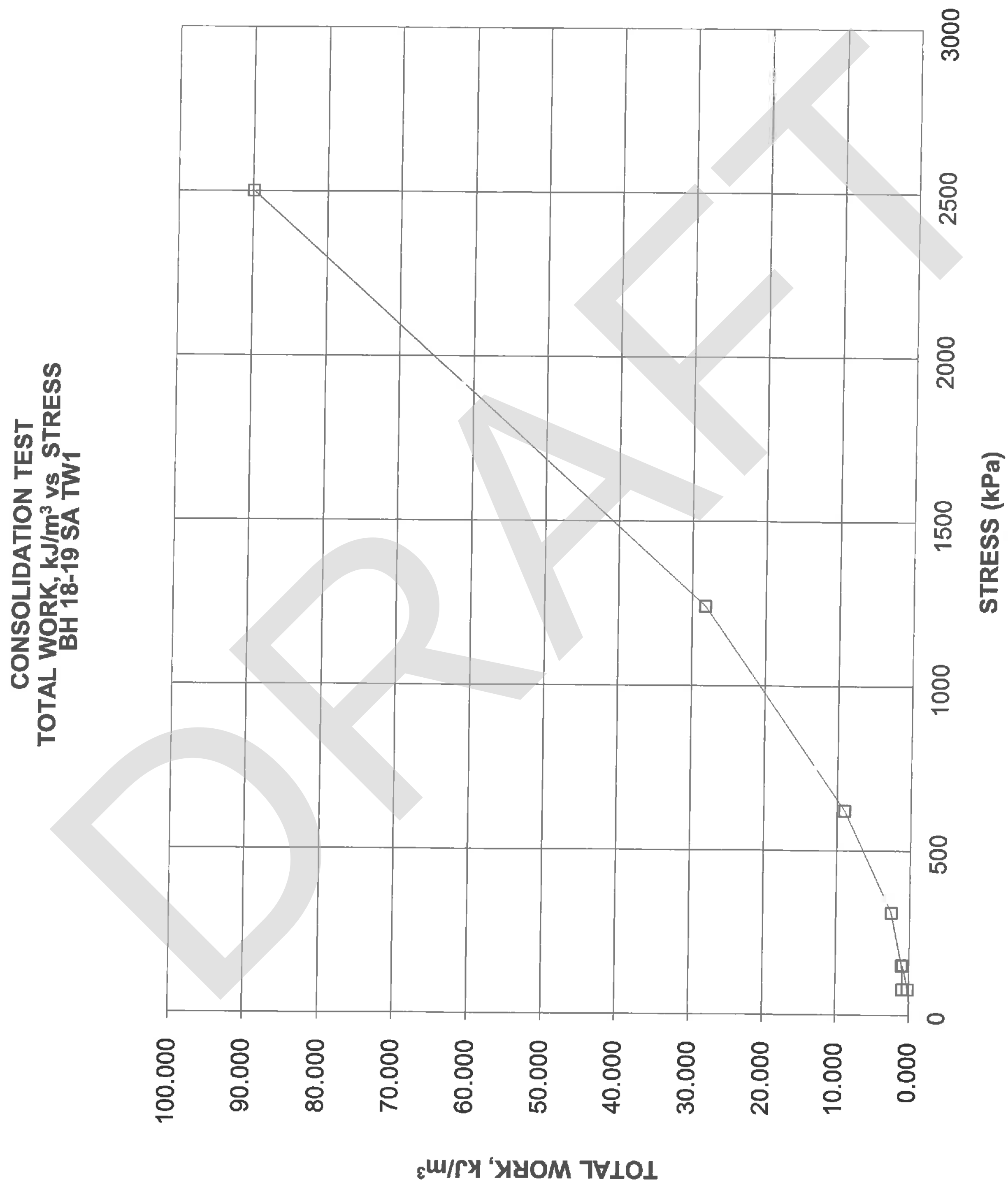
Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST TOTAL WORK VS STRESS	FIGURE



Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: *[signature]*

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW1
Borehole Number	18-25	Sample Depth, ft	12.20-12.80

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	07/26/2018		
Date Completed	08/10/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	19.34
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	15.09
Area, cm^2	31.50	Specific Gravity, measured	2.76
Volume, cm^3	59.53	Solids Height, cm	1.053
Water Content, %	28.22	Volume of Solids, cm^3	33.18
Wet Mass, g	117.44	Volume of Voids, cm^3	26.35
Dry Mass, g	91.59	Degree of Saturation, %	98.1

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.890	0.794	1.890				
5.89	1.892	0.796	1.891				
10.76	1.891	0.795	1.892	46	1.65E-02	9.78E-05	1.58E-07
20.56	1.890	0.794	1.891	48	1.58E-02	6.03E-05	9.33E-08
40.07	1.887	0.791	1.888	37	2.04E-02	9.18E-05	1.84E-07
78.98	1.874	0.778	1.880	57	1.31E-02	1.78E-04	2.30E-07
156.63	1.854	0.760	1.864	54	1.36E-02	1.34E-04	1.79E-07
78.97	1.855	0.761	1.854				
156.63	1.853	0.759	1.854	73	9.98E-03	1.43E-05	1.40E-08
311.39	1.820	0.728	1.837	101	7.08E-03	1.11E-04	7.73E-08
622.23	1.757	0.668	1.789	154	4.40E-03	1.08E-04	4.65E-08
1246.55	1.689	0.603	1.723	101	6.23E-03	5.76E-05	3.52E-08
2501.13	1.625	0.542	1.657	94	6.19E-03	2.70E-05	1.64E-08
1246.55	1.632	0.550	1.629				
313.87	1.648	0.564	1.640				
78.98	1.667	0.582	1.658				
20.68	1.689	0.603	1.678				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 5.89kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.69	Unit Weight, kN/m^3	20.61
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	16.89
Area, cm^2	31.50	Specific Gravity, measured	2.76
Volume, cm^3	53.19	Solids Height, cm	1.053
Water Content, %	22.04	Volume of Solids, cm^3	33.18
Wet Mass, g	111.78	Volume of Voids, cm^3	20.00
Dry Mass, g	91.59		

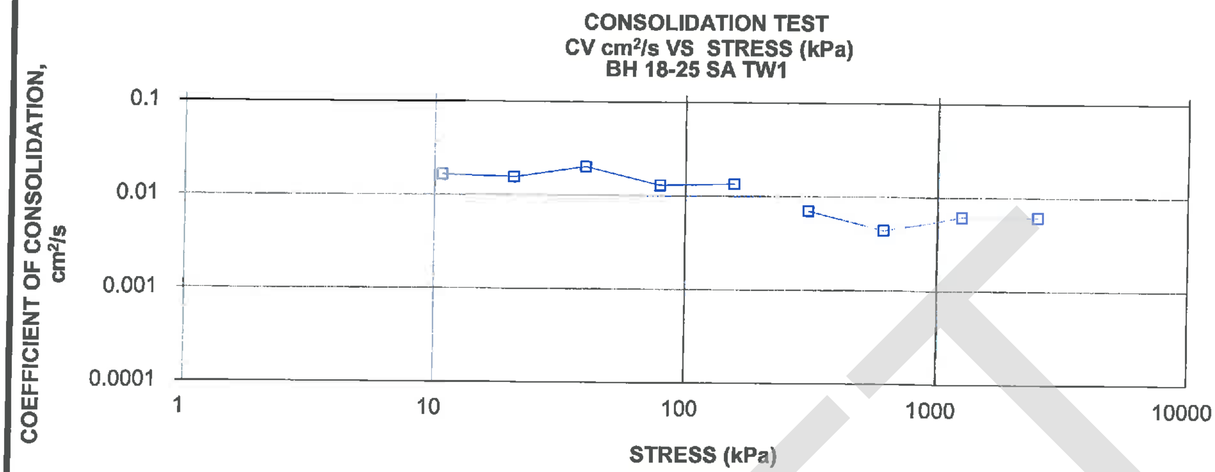
Prepared By: LH

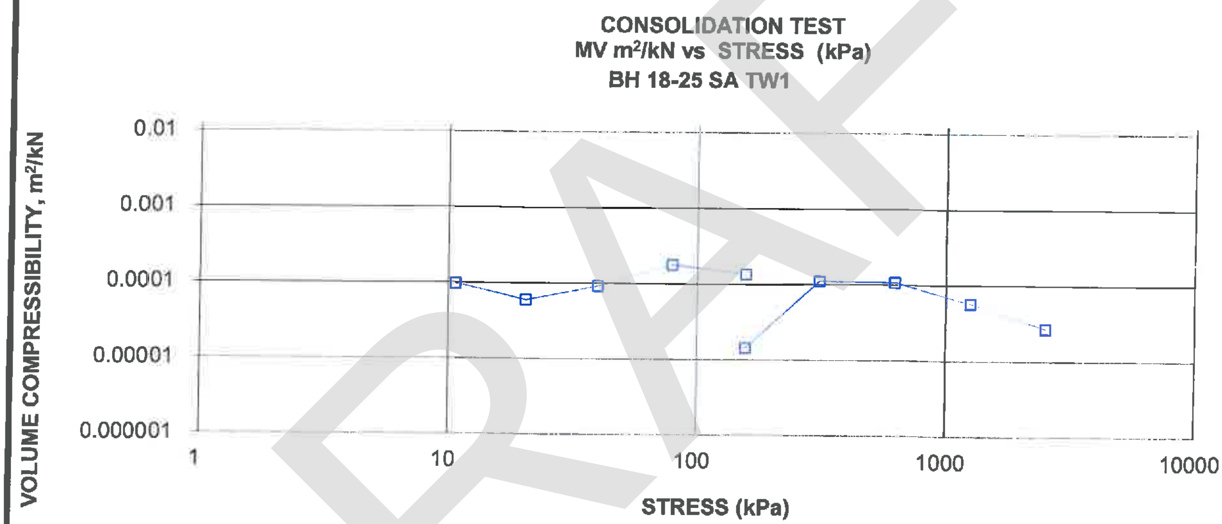
Golder Associates

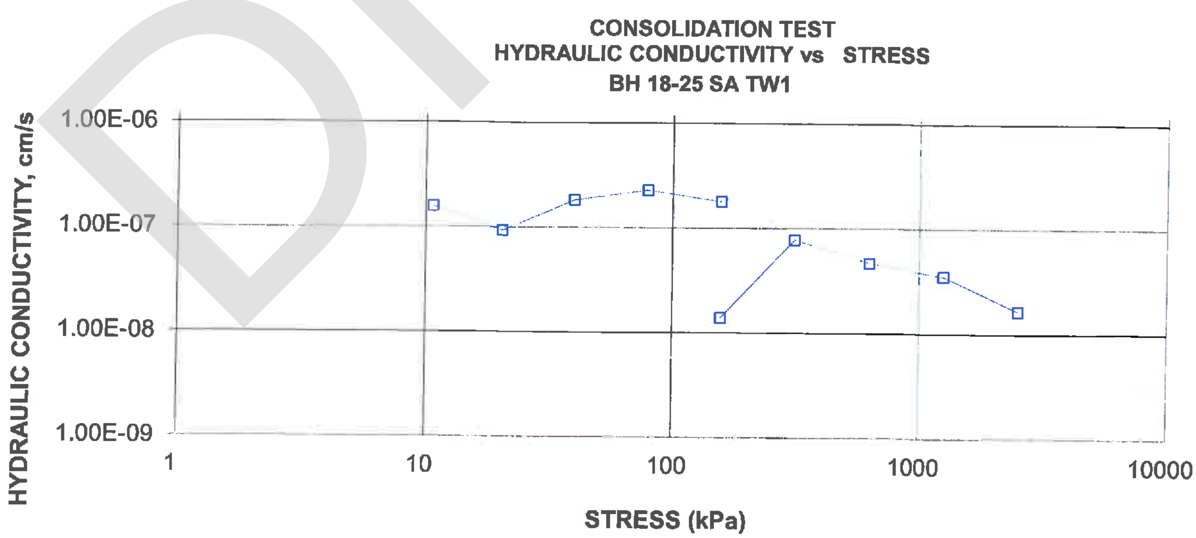
Checked By: LL

CONSOLIDATION TEST SUMMARY

FIGURE







Project No. 1897138(2000)

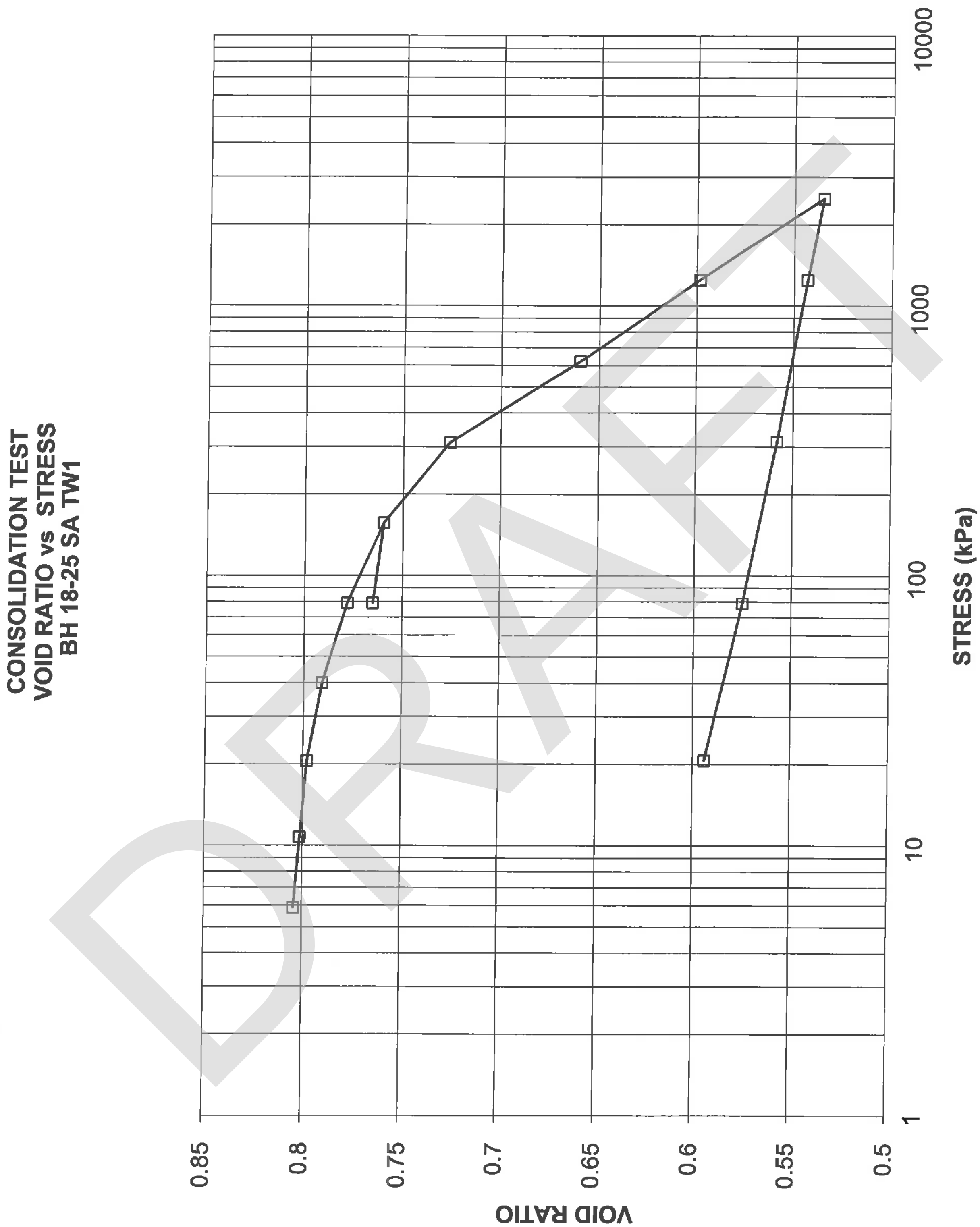
Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE



Project No.1897138(2000)

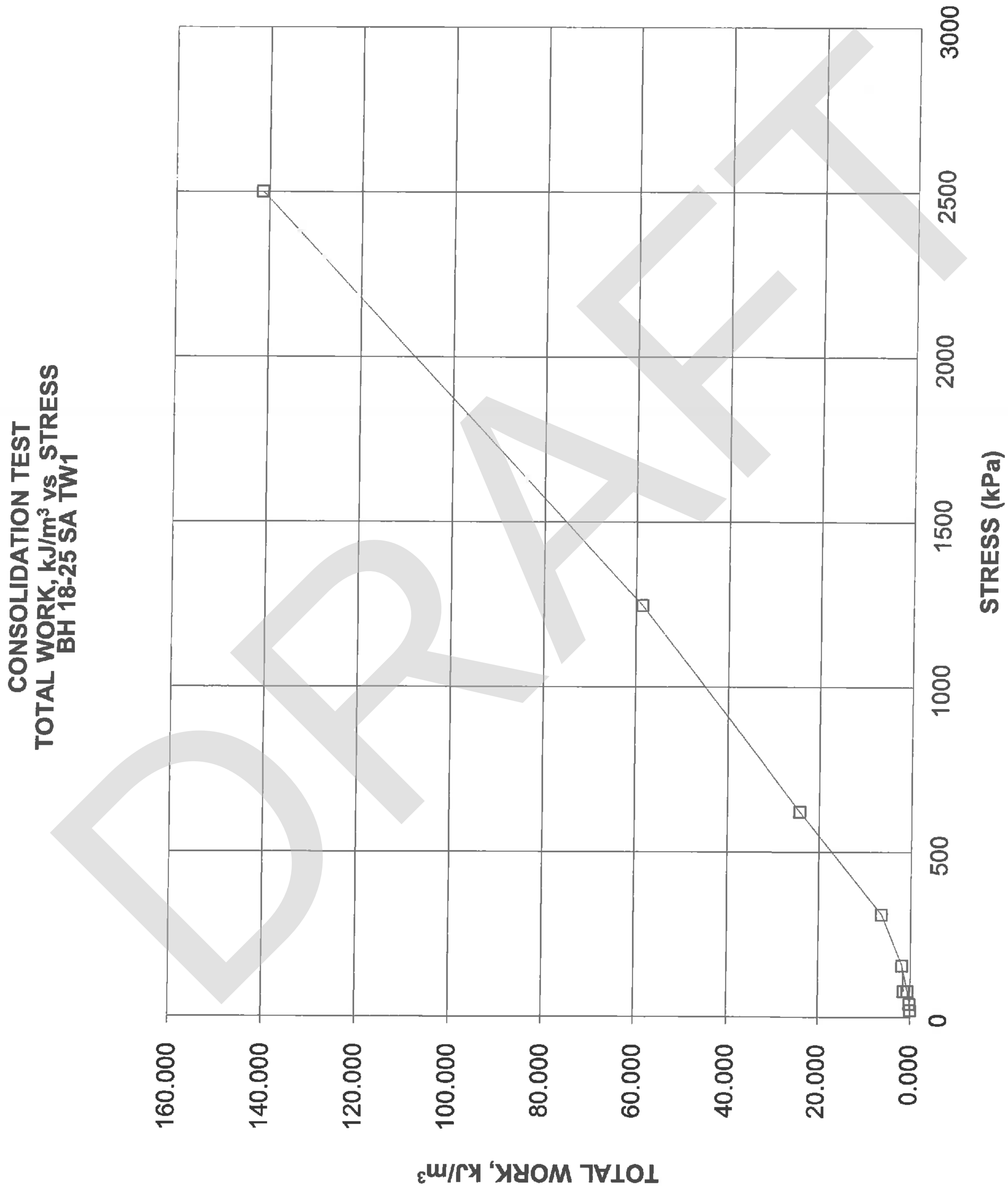
Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE



Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By:

CONSOLIDATION TEST SUMMARY

FIGURE

ASTM D2435/D2435M

SAMPLE IDENTIFICATION

Project Number	1897138(2000)	Sample Number	TW1
Borehole Number	18-01	Sample Depth, ft	12.20-12.80

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	07/26/2018		
Date Completed	08/09/2018		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	19.44
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	15.02
Area, cm^2	31.46	Specific Gravity, measured	2.79
Volume, cm^3	59.84	Solids Height, cm	1.044
Water Content, %	29.45	Volume of Solids, cm^3	32.84
Wet Mass, g	118.60	Volume of Voids, cm^3	27.00
Dry Mass, g	91.62	Degree of Saturation, %	99.9

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.902	0.822	1.902				
6.26	1.938	0.856	1.920				
10.94	1.934	0.853	1.936				
20.91	1.929	0.848	1.932	866	9.14E-04	2.80E-04	2.51E-08
40.26	1.918	0.837	1.923	799	9.82E-04	3.10E-04	2.99E-08
79.31	1.904	0.824	1.911	375	2.06E-03	1.79E-04	3.62E-08
152.15	1.894	0.814	1.899	346	2.21E-03	7.70E-05	1.67E-08
226.51	1.881	0.802	1.888	290	2.60E-03	8.78E-05	2.24E-08
152.39	1.885	0.805	1.883				
311.54	1.874	0.796	1.879	135	5.55E-03	3.42E-05	1.86E-08
621.16	1.849	0.772	1.862	173	4.25E-03	4.23E-05	1.76E-08
1242.95	1.815	0.739	1.832	126	5.65E-03	2.92E-05	1.61E-08
2503.01	1.655	0.586	1.735	240	2.66E-03	6.66E-05	1.74E-08
1251.42	1.668	0.598	1.661				
312.43	1.707	0.636	1.688				
79.31	1.757	0.683	1.732				
20.91	1.806	0.730	1.782				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 10.94kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

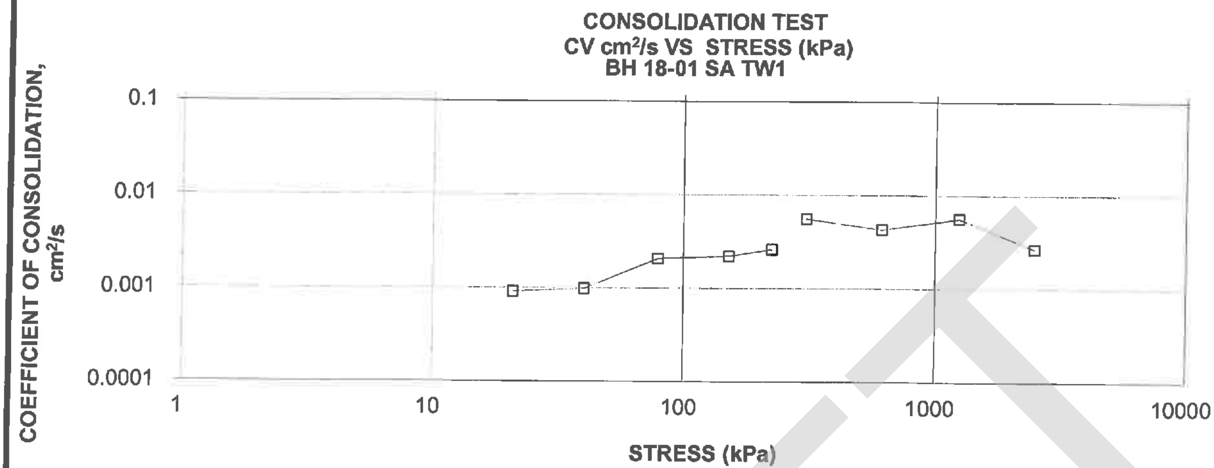
Sample Height, cm	1.81	Unit Weight, kN/m^3	20.53
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	15.81
Area, cm^2	31.46	Specific Gravity, measured	2.79
Volume, cm^3	56.83	Solids Height, cm	1.044
Water Content, %	29.84	Volume of Solids, cm ^{3}	32.84
Wet Mass, g	118.96	Volume of Voids, cm ^{3}	23.99
Dry Mass, g	91.62		

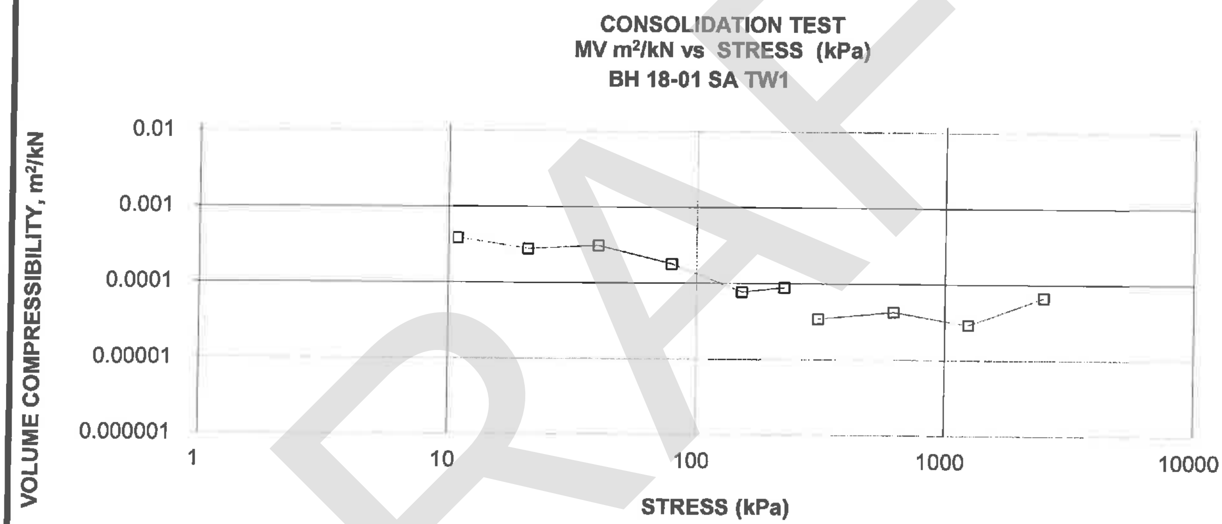
Prepared By: LH

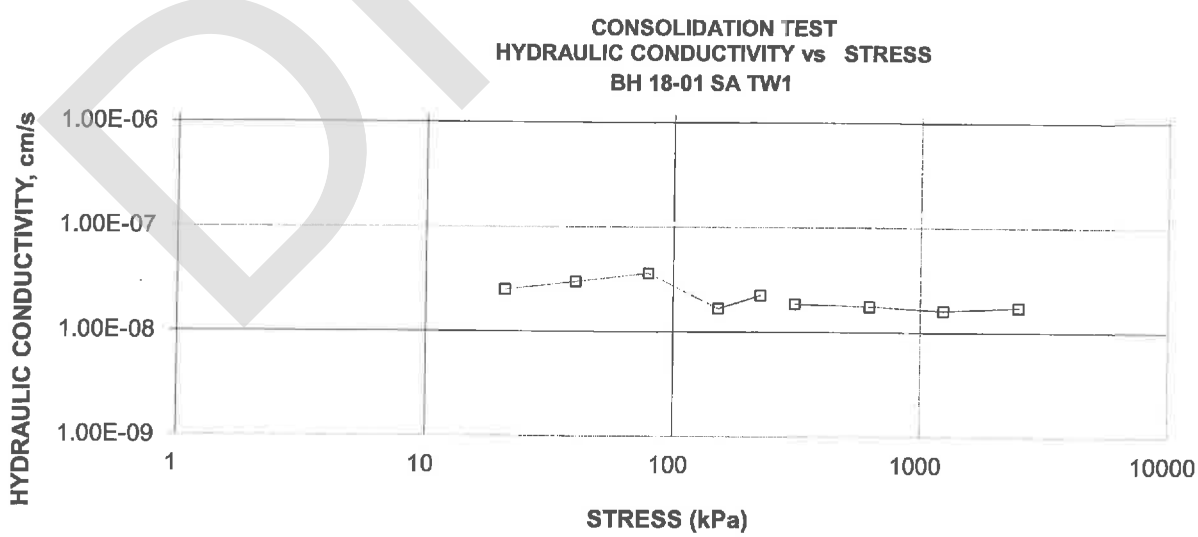
Golder Associates

Checked By: lil

CONSOLIDATION TEST SUMMARY	FIGURE







Project No. 1897138(2000)

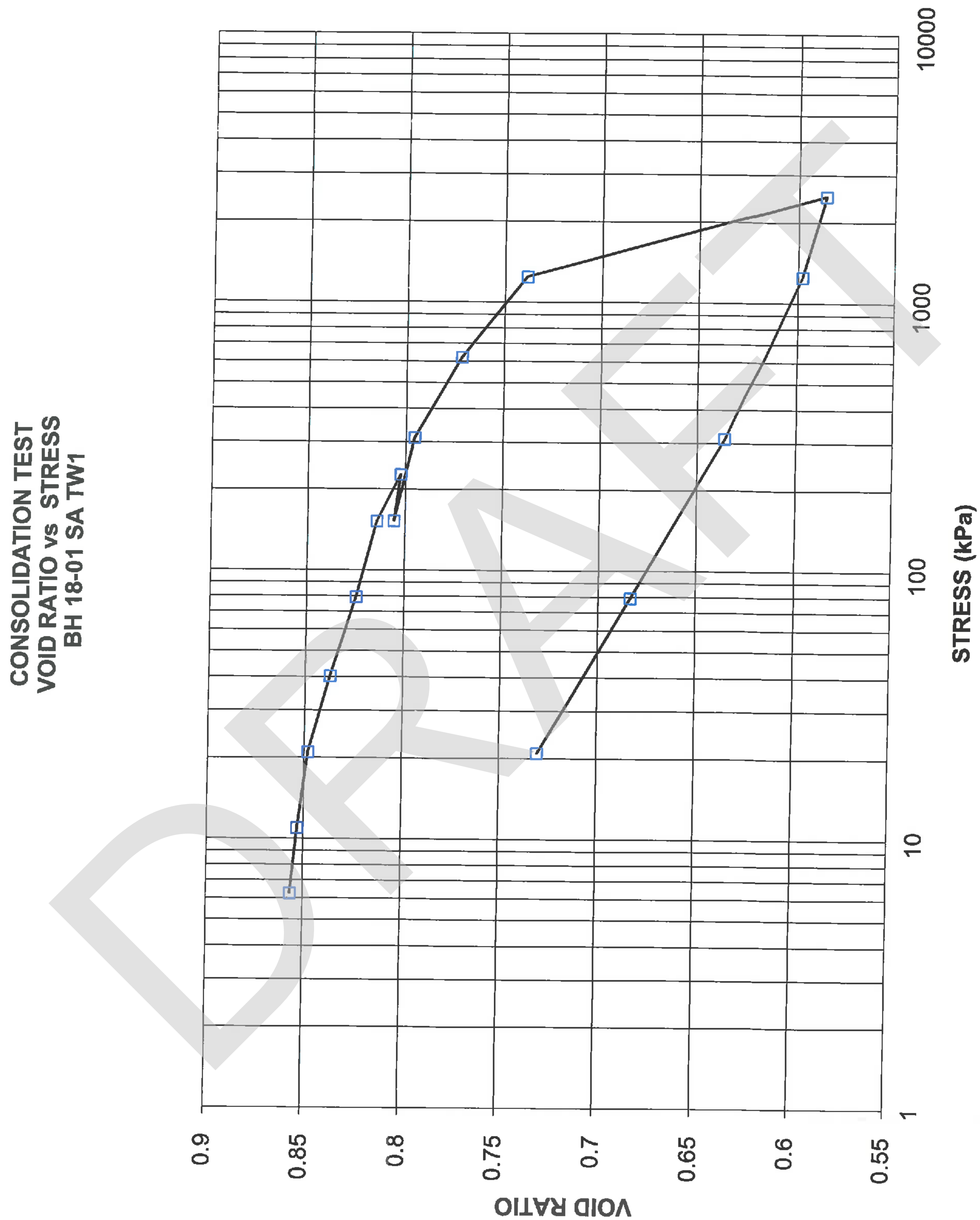
Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE



Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: lol

CONSOLIDATION TEST TOTAL WORK VS STRESS	FIGURE

Project No.1897138(2000)

Prepared By: LH

Golder Associates

Checked By: [signature]

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS
ASTM D4767
SHEET 1 OF 4

FIGURE

TEST STAGE	A	B	C
BOREHOLE NUMBER	18-19	18-21	
SAMPLE	T2	T2	
DEPTH, m	13.72-14.33	13.72-14.33	
SPECIMEN DIAMETER, cm	4.99	4.99	4.99
SPECIMEN HEIGHT, cm	10.17	10.10	10.11
NATURAL WATER CONTENT, %	31.5	29.4	28.0
DRY DENSITY, Mg/m^3	1.49	1.53	1.56
WATER CONTENT AFTER SATURATION, %	31.8	30.4	28.5
CELL PRESSURE, σ_3, kPa	370.0	470.0	670.0
BACK PRESSURE, kPa	270.0	270.0	270.0
PORE PRESSURE PARAMETER "B"	0.95	0.95	0.95
EFFECTIVE CONSOLIDATION STRESS, σc, kPa	100.0	200.0	400.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.8	3.6	5.8
WATER CONTENT AFTER CONSOLIDATION, %	30.0	28.0	24.8
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	9.6	5.1	4.7
WATER CONTENT AFTER TEST, %	29.7	27.1	24.1
MAX. DEVIATOR STRESS, $(\sigma_1-\sigma_3)$, kPa	110.0	169.6	278.2
AXIAL STRAIN AT $(\sigma_1-\sigma_3)$ maximum, %	4.8	2.5	2.3
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1/σ'_3) maximum	3.0	3.1	3.4
DEVIATOR STRESS AT (σ'_1/σ'_3) maximum, kPa	106.8	135.5	267.7
AXIAL STRAIN AT (σ'_1/σ'_3) maximum, %	7.9	13.7	11.5
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1-\sigma_3)$ maximum	0.39	0.57	0.84
PORE PRESSURE PARAMETER, Af, AT (σ'_1/σ'_3) maximum	0.43	1.01	1.07
FILTER DRAINS USED, y/n	y	y	y

TEST NOTES:

Effective consolidation stresses are assigned by the client.

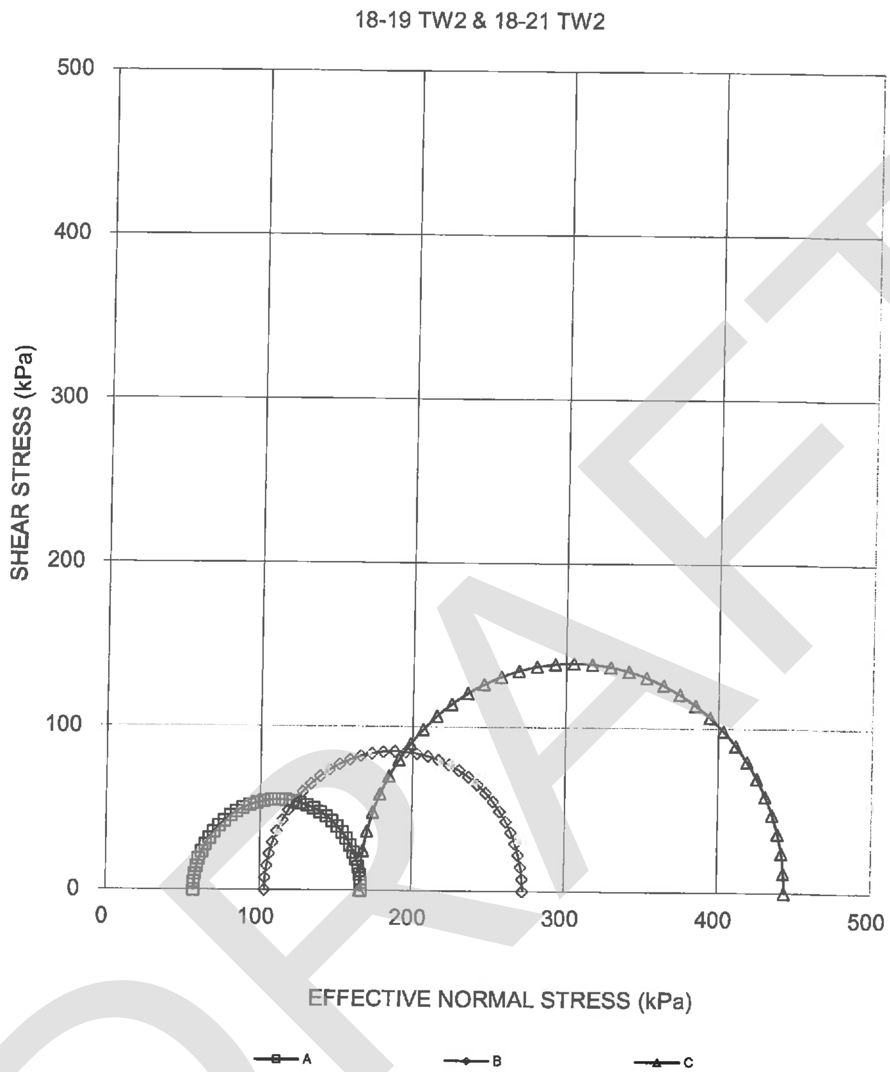
FAILURE PLANE NUMBER	1.0	-	-
ANGLE OF FAILURE PLANE, DEGREES	60.0	Bulged	Bulged

Date: 8/20/18
Project No. 1897138(2000)

Golder Associates

Prepared By LH
Checked By: [signature]

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 2 OF 4	FIGURE

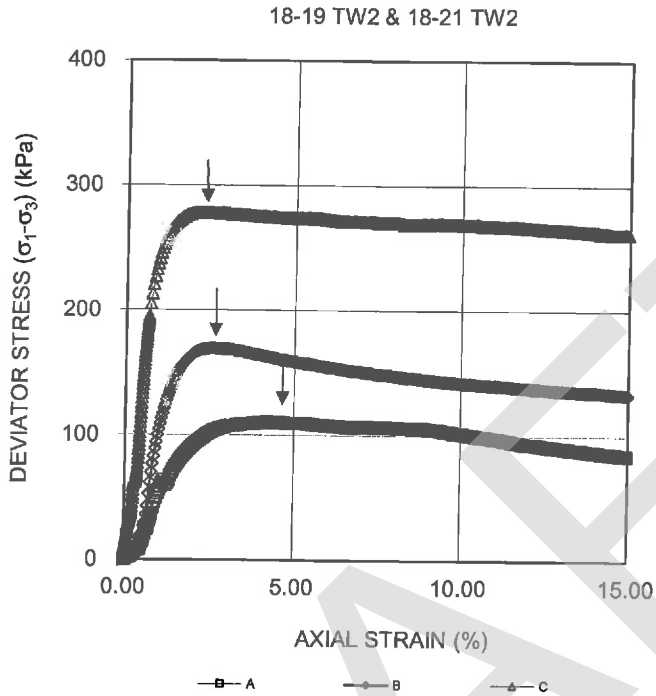


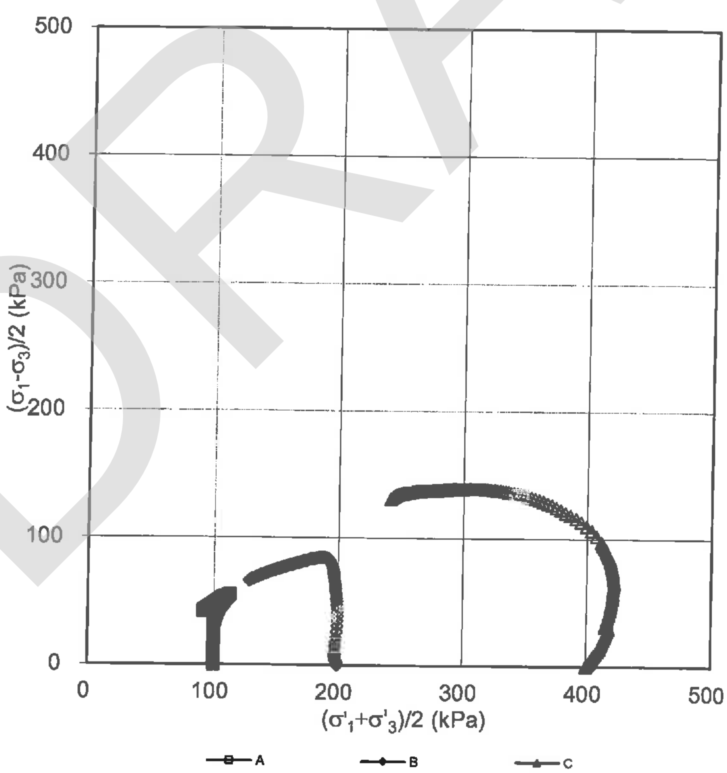
Date: 8/20/18
Project No. 1897138(2000)

Golder Associates

Prepared By LH
Checked By:

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 3 OF 4	FIGURE





Date: 8/20/18

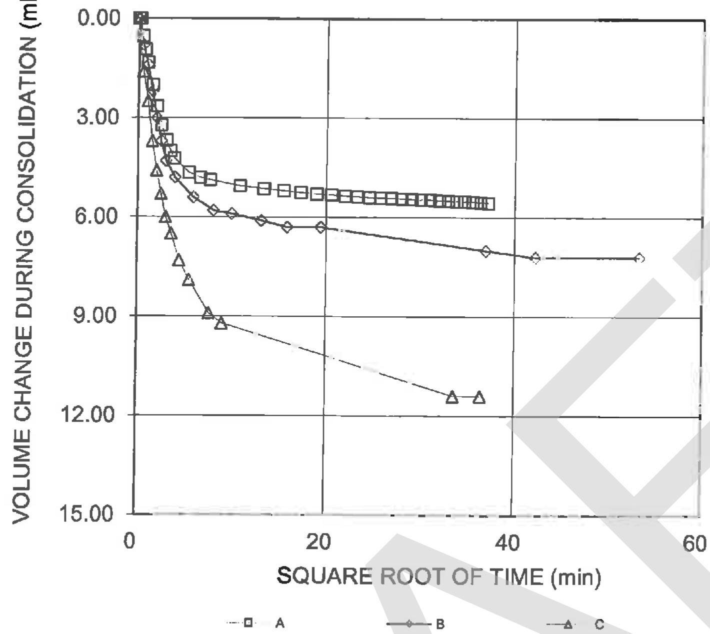
Project No. 1897138(2000)

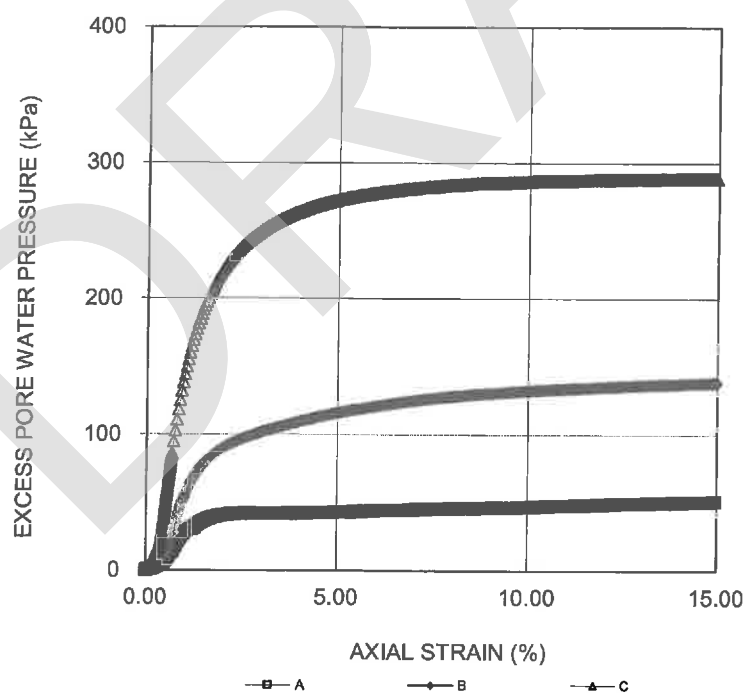
Golder Associates

Prepared By LH

Checked By: [signature]

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 4 OF 4	FIGURE





Date: 8/20/18

Project No. 1897138(2000)

Golder Associates

Prepared By LH

Checked By:

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 1 OF 4	FIGURE

TEST STAGE	A	B	C
BOREHOLE NUMBER	18-26		18-25
SAMPLE	T1		T1
DEPTH, m	7.62-8.23		12.20-12.80
SPECIMEN DIAMETER, cm	5.03	5.02	5.05
SPECIMEN HEIGHT, cm	10.12	10.13	10.14
NATURAL WATER CONTENT, %	26.9	31.0	29.1
DRY DENSITY, Mg/m^3	1.58	1.48	1.53
WATER CONTENT AFTER SATURATION, %	27.2	31.5	30.2
CELL PRESSURE, σ_3, kPa	205.0	280.0	500.0
BACK PRESSURE, kPa	130.0	130.0	200.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.96
EFFECTIVE CONSOLIDATION STRESS, σc, kPa	75.0	150.0	300.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	3.8	5.6	11.9
WATER CONTENT AFTER CONSOLIDATION, %	24.8	27.7	22.4
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, HOURS	7.5	13.1	3.5
WATER CONTENT AFTER TEST, %	27.1	28.6	24.5
MAX. DEVIATOR STRESS, $(\sigma_1-\sigma_3)$, kPa	89.4	126.1	183.0
AXIAL STRAIN AT $(\sigma_1-\sigma_3)$ maximum, %	3.8	6.5	1.7
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ'_1/σ'_3) maximum	2.4	2.3	3.1
DEVIATOR STRESS AT (σ'_1/σ'_3) maximum, kPa	86.9	123.7	156.5
AXIAL STRAIN AT (σ'_1/σ'_3) maximum, %	3.1	3.4	14.8
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1-\sigma_3)$ maximum	0.12	0.41	0.85
PORE PRESSURE PARAMETER, Af, AT (σ'_1/σ'_3) maximum	0.15	0.45	1.45
FILTER DRAINS USED, y/n	y	y	y

TEST NOTES:

Effective consolidation stresses are assigned by the client.

FAILURE PLANE NUMBER	1.0	1.0	-
ANGLE OF FAILURE PLANE, DEGREES	35.0	60.0	Bulged

Date: 8/17/18

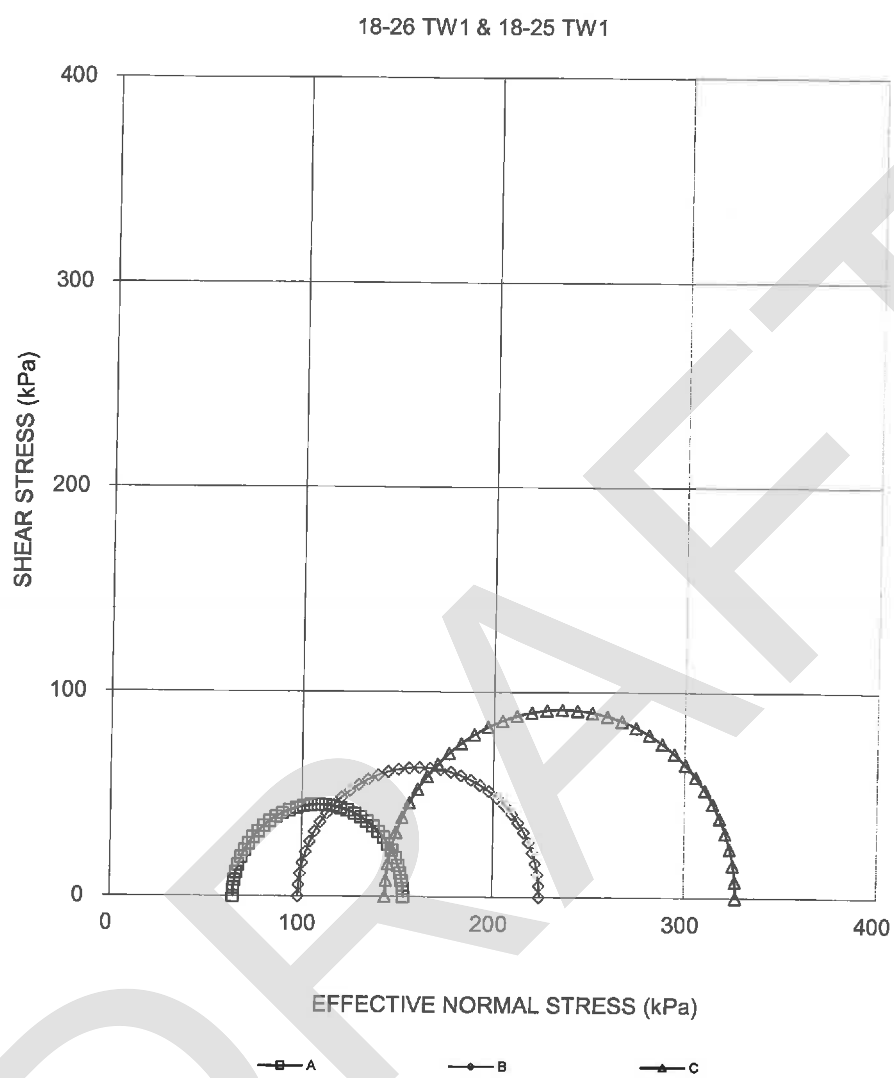
Project No. 1897138(2000)

Golder Associates

Prepared By LH

Checked By: [signature]

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 2 OF 4	FIGURE



Date: 8/17/18

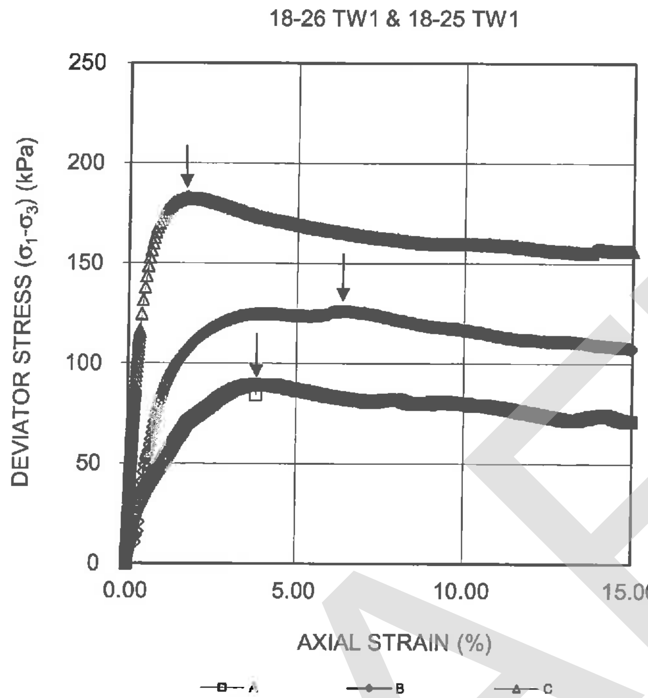
Project No. 1897138(2000)

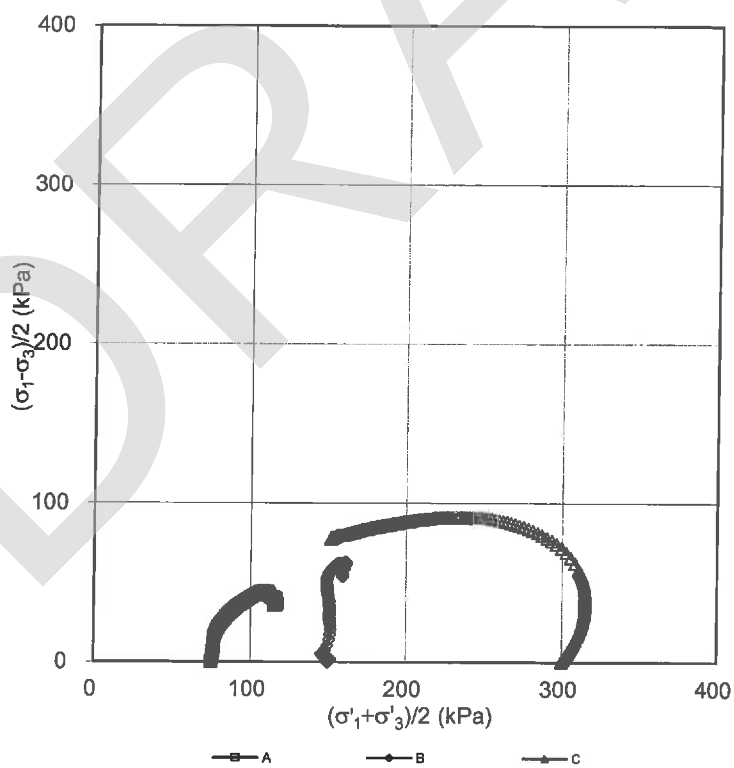
Golder Associates

Prepared By LH

Checked By: Mcl

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 3 OF 4	FIGURE





Date: 8/17/18

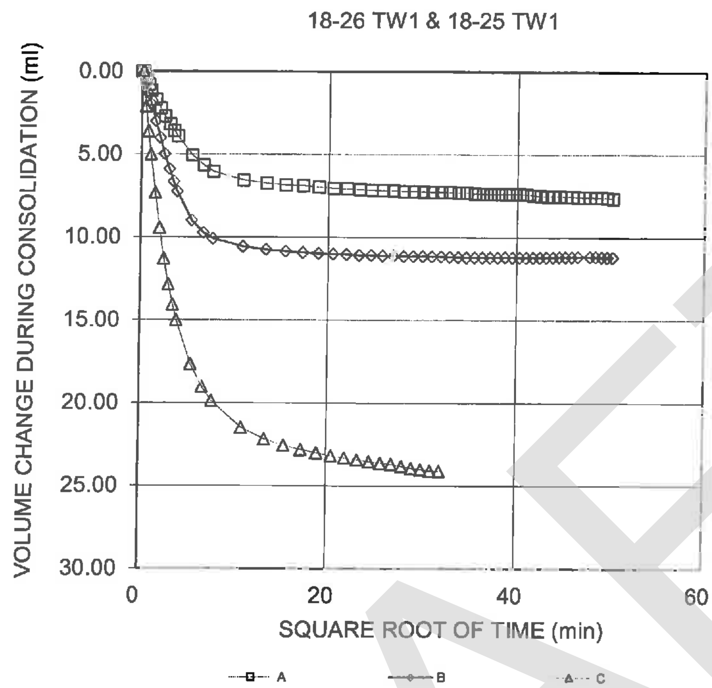
Project No. 1897138(2000)

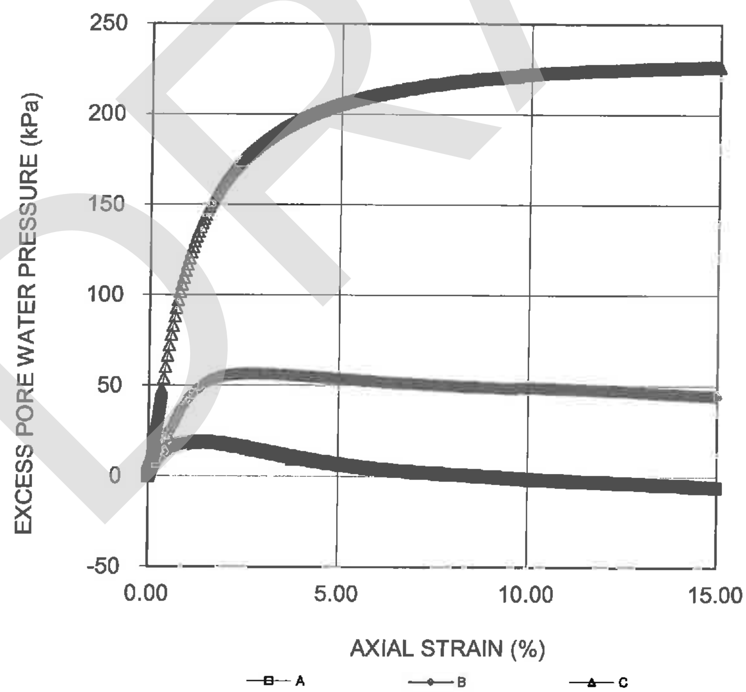
Golder Associates

Prepared By LH

Checked By:

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS ASTM D4767 SHEET 4 OF 4	FIGURE





Date: 8/17/18

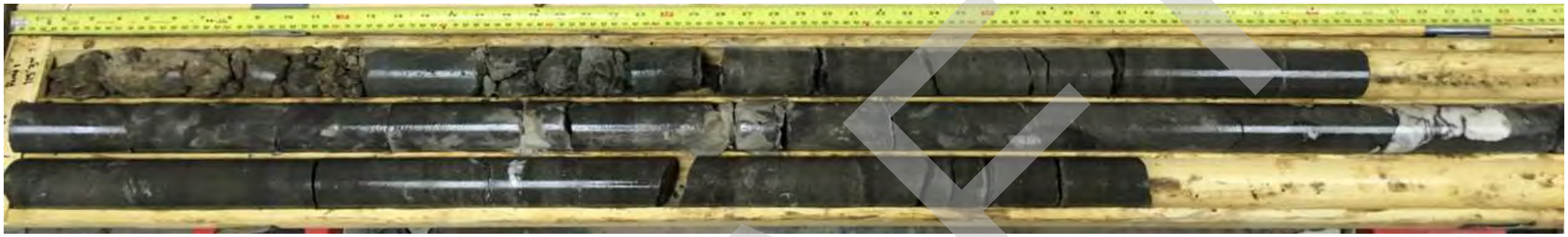
Project No. 1897138(2000)

Golder Associates

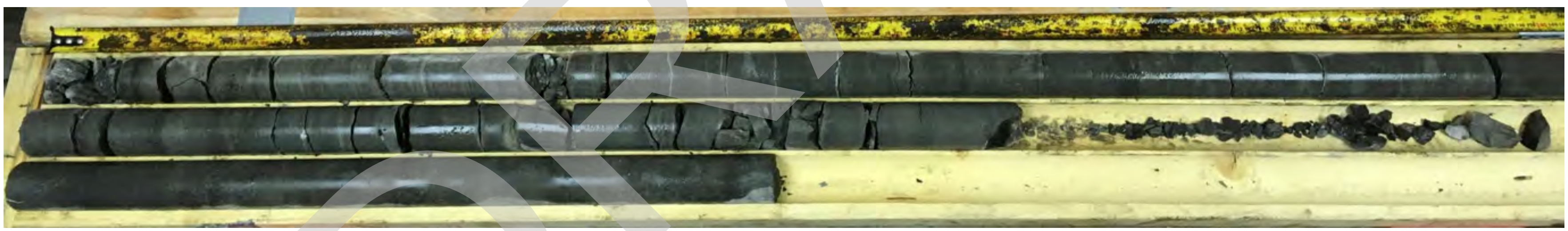
Prepared By LH

Checked By:

BOREHOLE:	**18-03**
CORE RUN #1:	115' 3" – 118' 9"
CORE RUN #2:	118' 9" – 123' 9"
CORE RUN #3:	123' 9" – 127' 0"



BOREHOLE:	**18-04**
CORE RUN #1:	115' 9" – 120' 9"
CORE RUN #2:	120' 9" – 123' 11"
CORE RUN #3:	123' 11" – 126' 2"



THURBER ENGINEERING LTD.

REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE:	**18-05**
CORE RUN #1:	115' 4" – 118' 8"
CORE RUN #2:	118' 8" – 121' 1"
CORE RUN #3:	121' 1" – 126' 1"
CORE RUN #4:	126' 1" – 130' 6"

BOREHOLE:	**18-06**
CORE RUN #1:	82' 6" – 83' 3"
CORE RUN #2:	83' 3" – 87' 11"
CORE RUN #3:	87' 11" – 92' 11"

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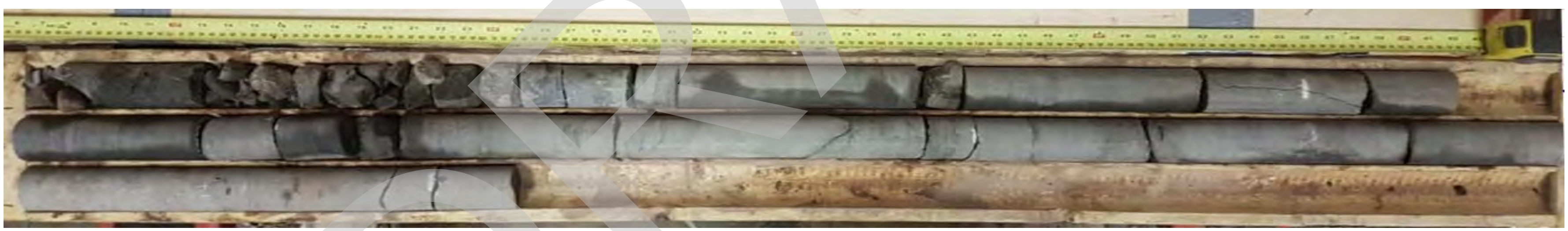
REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE:	**18-07**
CORE RUN #1:	82' 10" – 84' 5"
CORE RUN #2:	84' 5" – 89' 5"
CORE RUN #3:	89' 5" – 94' 0"

BOREHOLE:	**18-08**
CORE RUN #1:	81' 6" – 86' 6"
CORE RUN #2:	86' 6" – 91' 6"
CORE RUN #3:	91' 6" – 93' 0"



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REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE:	**18-09**
CORE RUN #1:	86' 2" – 91' 2"
CORE RUN #2:	91' 2" – 96' 2"
CORE RUN #3:	96' 2" – 97' 2"

BOREHOLE:	**18-10**
CORE RUN #1:	85' 7" – 90' 10"
CORE RUN #2:	90' 10" – 95' 3"
CORE RUN #3:	95' 3" – 97' 9"

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REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE: **18-11**
CORE RUN #1: 85' 7" – 89' 4"
CORE RUN #2: 89' 4" – 92' 3"
CORE RUN #3: 92' 3" – 96' 0"

BOREHOLE: **18-12**
CORE RUN #1: 70' 2" – 73' 10"
CORE RUN #2: 73' 10" – 78' 10"
CORE RUN #3: 78' 10" – 80' 11"

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REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE:	**18-13**
CORE RUN #1:	72' 6" – 77' 7"
CORE RUN #2:	77' 7" – 81' 7"
CORE RUN #3:	81' 7" – 83' 2"

BOREHOLE:	**18-14**
CORE RUN #1:	72' 8" – 74' 3"
CORE RUN #2:	74' 3" – 79' 3"
CORE RUN #3:	79' 3" – 83' 3"

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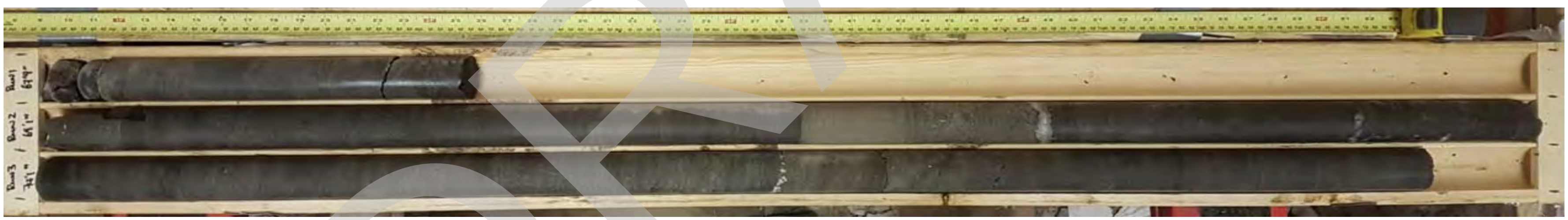
REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE: **18-15**
CORE RUN #1: 66' 8" – 69' 10"
CORE RUN #2: 69' 10" – 73' 1"
CORE RUN #3: 73' 1" – 77' 10"

BOREHOLE: **18-16**
CORE RUN #1: 67' 9" – 69' 1"
CORE RUN #2: 69' 1" – 74' 1"
CORE RUN #3: 74' 1" – 78' 8"



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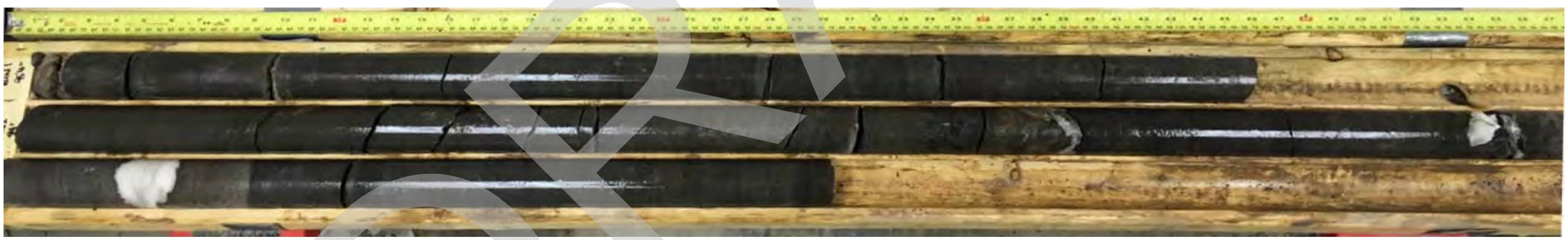
REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE:	**18-17**
CORE RUN #1:	65' 3" – 69' 7"
CORE RUN #2:	69' 7" – 74' 7"
CORE RUN #3:	74' 7" – 76' 11"

BOREHOLE:	**18-18**
CORE RUN #1:	95' 4" – 99' 0"
CORE RUN #2:	99' 0" – 104' 0"
CORE RUN #3:	104' 0" – 106' 5"



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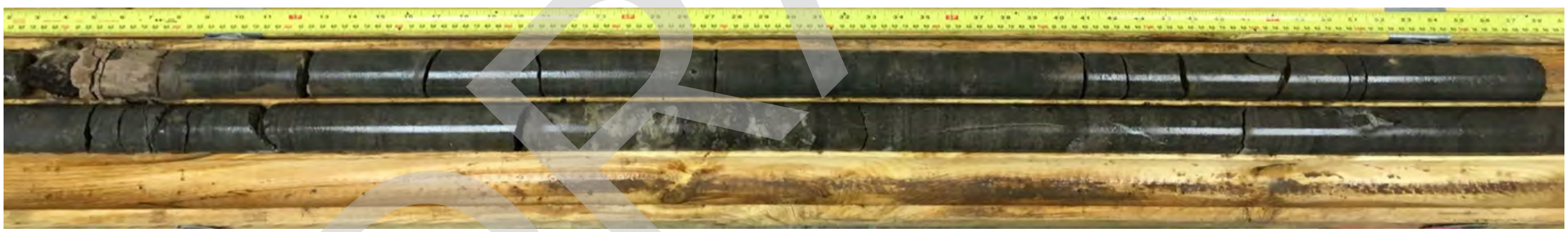
REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

BOREHOLE: **18-19**
CORE RUN #1: 95' 5" – 98' 11"
CORE RUN #2: 98' 11" – 103' 11"
CORE RUN #3: 103' 11" – 106' 11"

BOREHOLE: **18-20**
CORE RUN #1: 95' 9" – 100' 11"
CORE RUN #2: 100' 11" – 105' 11"



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REPLACEMENT OF WELLAND RIVER TWIN BRIDGE STRUCTURES
REGIONAL MUNICIPALITY OF NIAGARA
CITY OF NIAGARA FALLS, ON

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 18426 **Date Drilled:** 10-Apr-18

Client: WSP **Date Tested:** 18-Apr-18

Project Name: Welland River Twin Bridge Structures **Tester:** GA

Core Size: NQ **BH No :** 18-03 **Reviewed by:** GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	35.4	D	25.8	47.4	118.0	10.6	255.0	Dolostone	Extremely Strong
2	1	35.7	D	23.5	47.4	82.4	9.7	232.5	Dolostone	Very Strong
3	1	36.1	A	26.1	47.4	60.7	7.4	176.8	Dolostone	Very Strong
4	2	36.3	A	9.6	47.4	48.4	3.2	77.1	Dolostone	Strong
5	2	36.6	D	15.1	47.4	112.7	6.2	149.5	Dolostone	Very Strong
6	2	37.0	A	14.0	47.4	70.5	3.5	84.3	Dolostone	Strong
7	2	37.3	D	8.4	47.4	114.0	3.5	83.0	Dolostone	Strong
8	3	37.7	A	14.2	47.4	62.6	3.9	94.0	Dolostone	Strong
9	3	38.1	D	18.4	47.4	164.8	7.6	181.9	Dolostone	Very Strong
10	3	38.6	A	24.9	47.4	60.1	7.1	169.9	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	20-Apr-18
Client:	WSP	**Date Tested:**	02-May-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-04	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	35.4	A	11.5	47.3	58.3	3.3	80.3	Dolostone	Strong
2	1	35.6	D	13.2	47.3	128.6	5.5	131.3	Dolostone	Very Strong
3	1	35.9	A	20.9	47.4	60.4	5.9	142.0	Dolostone	Very Strong
4	1	36.2	D	21.9	47.4	171.2	9.0	216.5	Dolostone	Very Strong
5	1	36.6	A	21.7	47.4	50.6	7.0	168.8	Dolostone	Very Strong
6	2	36.8	A	13.0	47.4	53.0	4.1	97.4	Dolostone	Strong
7	2	37.2	A	27.4	47.4	69.1	7.0	167.9	Dolostone	Very Strong
8	2	37.7	D	10.7	47.4	131.7	4.4	105.6	Dolostone	Very Strong
9	3	37.8	A	20.2	47.3	60.3	5.7	137.4	Dolostone	Very Strong
10	3	38.1	D	22.5	47.3	140.7	9.3	222.7	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	08-Apr-18
Client:	WSP	**Date Tested:**	18-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-05	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	35.8	A	9.3	47.4	52.0	3.0	71.2	Dolostone	Strong
2	2	36.5	D	22.9	47.4	82.6	9.4	226.5	Dolostone	Very Strong
3	3	37.1	D	19.7	47.4	82.1	8.1	194.8	Dolostone	Very Strong
4	3	37.5	A	15.6	47.4	64.5	4.2	100.8	Dolostone	Very Strong
5	3	38.1	D	17.6	47.4	103.1	7.2	174.0	Dolostone	Very Strong
6	3	38.3	A	10.0	47.4	54.2	3.1	74.2	Dolostone	Strong
7	4	38.5	D	26.1	47.4	192.1	10.8	258.4	Dolostone	Extremely Strong
8	4	38.9	A	21.5	47.4	63.4	5.9	141.0	Dolostone	Very Strong
9	4	39.5	D	15.7	47.4	210.9	6.5	155.4	Dolostone	Very Strong
10	4	39.7	A	17.4	47.4	61.6	4.9	116.5	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	27-Mar-18
Client:	WSP	**Date Tested:**	02-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-06	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	25.3	D	11.8	47.0	160.5	4.9	118.5	Dolostone	Very Strong
2	2	25.8	D	21.5	47.0	97.6	9.0	215.0	Dolostone	Very Strong
3	2	26.5	D	16.2	47.0	93.2	6.8	162.5	Dolostone	Very Strong
4	3	27.0	D	21.5	47.0	324.4	9.0	215.8	Dolostone	Very Strong
5	3	27.8	D	14.0	47.0	223.4	5.9	140.6	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

Last Modified: September 14, 2016

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	28-Mar-18
Client:	WSP	**Date Tested:**	03-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-07	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	25.4	D	17.4	47.3	108.9	7.2	172.1	Dolostone	Very Strong
2	1	25.7	D	17.2	47.3	122.3	7.1	170.9	Dolostone	Very Strong
3	2	25.9	D	19.2	47.3	289.2	8.0	190.8	Dolostone	Very Strong
4	2	27.0	D	14.8	47.3	189.5	6.1	146.8	Dolostone	Very Strong
5	3	27.4	D	11.2	47.4	99.1	4.6	110.3	Dolostone	Very Strong
6	3	27.7	D	19.9	47.3	152.2	8.2	197.5	Dolostone	Very Strong
7	3	28.5	D	17.0	47.3	222.5	7.0	168.4	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	27-Mar-18
Client:	WSP	**Date Tested:**	03-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-08	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	25.4	D	13.9	47.2	112.7	5.8	138.1	Dolostone	Very Strong
2	1	26.1	D	11.6	47.2	156.5	4.8	115.3	Dolostone	Very Strong
3	2	26.6	D	18.1	47.2	136.3	7.5	180.3	Dolostone	Very Strong
4	2	27.3	D	13.9	47.2	105.7	5.7	137.9	Dolostone	Very Strong
5	3	28.3	D	19.4	47.0	116.5	8.1	193.9	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

Last Modified: September 14, 2016

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	18-Mar-18
Client:	WSP	**Date Tested:**	21-Mar-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-09	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	26.4	D	21.8	46.9	105.2	9.1	218.4	Dolostone	Very Strong
2	1	26.8	A	28.4	47.0	73.8	6.9	166.5	Dolostone	Very Strong
3	1	27.1	A	24.1	46.9	64.7	6.5	156.2	Dolostone	Very Strong
4	1	27.4	D	20.9	46.9	93.8	8.7	210.0	Dolostone	Very Strong
5	1	27.7	D	21.5	46.9	148.3	9.0	215.4	Dolostone	Very Strong
6	2	27.9	D	12.5	47.0	171.5	5.2	125.2	Dolostone	Very Strong
7	2	28.2	A	15.1	47.0	52.1	4.8	115.9	Dolostone	Very Strong
8	2	28.5	D	16.8	46.9	204.1	7.0	168.9	Dolostone	Very Strong
9	2	28.9	A	19.7	46.9	59.7	5.7	136.3	Dolostone	Very Strong
10	2	29.2	D	13.3	46.9	213.0	5.6	133.9	Dolostone	Very Strong
11	3	29.4	D	24.3	46.9	196.2	10.2	244.3	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	12-Mar-18
Client:	WSP	**Date Tested:**	21-Mar-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-10	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	26.3	D	18.5	46.9	120.0	7.8	186.1	Dolostone	Very Strong
2	1	26.5	A	17.1	46.9	62.9	4.7	113.5	Dolostone	Very Strong
3	1	26.8	A	29.9	46.9	66.3	7.9	190.5	Dolostone	Very Strong
4	1	27.1	D	13.3	46.9	153.2	5.6	133.5	Dolostone	Very Strong
5	1	27.6	A	22.7	46.9	65.7	6.1	145.7	Dolostone	Very Strong
6	2	27.8	D	16.8	46.9	242.1	7.0	168.8	Dolostone	Very Strong
7	2	28.2	A	24.4	46.9	69.6	6.2	149.8	Dolostone	Very Strong
8	2	28.4	D	14.0	46.9	144.7	5.9	140.7	Dolostone	Very Strong
9	2	28.7	D	13.6	46.9	189.2	5.7	136.6	Dolostone	Very Strong
10	2	29.0	A	22.4	46.9	65.3	6.0	144.4	Dolostone	Very Strong
11	3	29.1	D	10.5	46.9	128.9	4.4	105.4	Dolostone	Very Strong
12	3	29.6	A	17.3	46.9	58.9	5.0	120.7	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	15-Mar-18
Client:	WSP	**Date Tested:**	16-Mar-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-11	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	26.3	D	18.6	46.5	117.7	7.9	188.8	Dolostone	Very Strong
2	1	26.6	A	31.6	46.6	63.2	8.7	209.9	Dolostone	Very Strong
3	1	27.2	A	35.7	46.5	68.8	9.3	222.4	Dolostone	Very Strong
4	2	27.4	A	24.3	46.5	64.9	6.6	158.6	Dolostone	Very Strong
5	2	27.6	A	23.7	46.6	58.0	7.0	168.7	Dolostone	Very Strong
6	2	28.0	D	14.3	46.5	88.0	6.1	145.6	Dolostone	Very Strong
7	3	28.2	D	25.6	46.6	218.5	10.8	259.8	Dolostone	Extremely Strong
8	3	28.6	D	23.5	46.6	172.6	10.0	239.0	Dolostone	Very Strong
9	3	29.1	A	18.9	46.6	63.5	5.2	125.4	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	20-Mar-18
Client:	WSP	**Date Tested:**	02-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-12	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	21.8	D	10.0	46.7	116.2	4.2	100.8	Dolostone	Very Strong
2	1	22.5	D	14.0	46.4	86.3	6.0	143.1	Dolostone	Very Strong
3	2	22.7	D	22.8	46.9	143.5	9.5	228.7	Dolostone	Very Strong
4	2	23.8	D	22.2	46.8	508.1	9.3	223.4	Dolostone	Very Strong
5	2	23.9	D	24.8	46.9	254.6	10.4	249.6	Dolostone	Very Strong
6	3	24.4	D	11.8	46.8	538.4	5.0	119.0	Dolostone	Very Strong
7	3	24.5	D	19.3	46.7	278.4	8.1	195.3	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 18426 **Date Drilled:** 19-Mar-18

Client: WSP **Date Tested:** 21-Mar-18

Project Name: Welland River Twin Bridge Structures **Tester:** GA

Core Size: NQ **BH No :** 18-13 **Reviewed by:** GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	22.5	A	24.5	46.9	59.2	7.1	170.5	Dolostone	Very Strong
2	1	22.9	A	24.6	46.9	61.4	6.9	166.6	Dolostone	Very Strong
3	1	23.1	D	13.8	47.0	156.1	5.8	138.4	Dolostone	Very Strong
4	1	23.4	D	20.1	46.9	196.2	8.4	201.3	Dolostone	Very Strong
5	2	23.9	D	21.7	46.9	102.2	9.1	218.1	Dolostone	Very Strong
6	2	24.1	D	15.8	46.9	119.2	6.6	158.9	Dolostone	Very Strong
7	2	24.4	A	22.2	46.9	71.7	5.6	133.4	Dolostone	Very Strong
8	2	24.8	D	27.9	46.9	140.1	11.7	280.2	Dolostone	Extremely Strong
9	3	25.0	D	23.4	46.9	141.6	9.8	235.4	Dolostone	Very Strong
10	3	25.2	A	22.5	46.9	62.4	6.3	150.6	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	21-Mar-18
Client:	WSP	**Date Tested:**	02-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-14	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	22.4	D	12.7	46.4	262.7	5.4	130.0	Dolostone	Very Strong
2	2	22.9	D	13.1	46.1	145.5	5.6	135.4	Dolostone	Very Strong
3	2	24.0	D	14.6	46.3	231.3	6.3	150.1	Dolostone	Very Strong
4	3	24.3	D	5.6	46.2	155.0	2.4	57.2	Dolostone	Strong
5	3	25.3	D	15.2	46.3	329.9	6.5	155.4	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	23-Mar-18
Client:	WSP	**Date Tested:**	03-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-15	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	20.5	D	5.8	47.0	172.3	2.4	57.7	Dolostone	Strong
2	1	21.2	D	9.9	47.1	99.8	4.1	99.0	Dolostone	Strong
3	2	21.5	D	25.1	47.0	97.8	10.5	251.2	Dolostone	Extremely Strong
4	2	22.2	D	19.2	46.8	145.2	8.1	193.4	Dolostone	Very Strong
5	3	22.4	D	18.2	47.0	141.4	7.6	182.5	Dolostone	Very Strong
6	3	23.6	D	13.7	46.9	154.7	5.7	137.7	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

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POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	24-Mar-18
Client:	WSP	**Date Tested:**	03-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-16	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	20.8	D	13.4	46.9	63.1	5.6	134.3	Dolostone	Very Strong
2	1	21.0	D	16.4	47.0	78.9	6.8	164.1	Dolostone	Very Strong
3	2	21.2	D	21.1	47.0	168.0	8.8	211.2	Dolostone	Very Strong
4	2	21.9	D	14.1	47.0	240.5	5.9	141.0	Dolostone	Very Strong
5	3	22.7	D	21.0	47.0	119.3	8.7	210.0	Dolostone	Very Strong
6	3	23.1	D	17.4	47.0	242.1	7.2	173.9	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

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POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	24-Mar-18
Client:	WSP	**Date Tested:**	02-Apr-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	KF
Core Size:	NQ **BH No :** 18-17	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	26.3	D	11.6	46.8	135.6	4.9	117.4	Dolostone	Very Strong
2	1	26.5	D	17.3	47.1	127.2	7.2	173.1	Dolostone	Very Strong
3	2	26.8	D	23.9	47.0	153.7	10.0	239.0	Dolostone	Very Strong
4	2	27.1	D	22.1	47.2	526.8	9.2	219.7	Dolostone	Very Strong
5	3	27.6	D	18.4	47.1	706.4	7.6	183.5	Dolostone	Very Strong
6	3	27.8	D	19.2	47.1	585.5	8.0	191.7	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	18-Mar-18
Client:	WSP	**Date Tested:**	21-Mar-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-18	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	29.1	A	19.3	47.4	53.3	6.0	144.8	Dolostone	Very Strong
2	1	29.3	D	7.3	47.4	153.1	3.0	72.2	Dolostone	Strong
3	1	29.8	A	24.4	47.4	56.8	7.2	173.5	Dolostone	Very Strong
4	1	30.0	D	15.2	47.4	149.2	6.2	149.9	Dolostone	Very Strong
5	2	30.3	A	28.5	47.4	64.2	7.7	184.7	Dolostone	Very Strong
6	2	30.7	D	14.5	47.4	158.1	6.0	143.0	Dolostone	Very Strong
7	2	31.1	D	21.8	47.4	172.3	9.0	215.1	Dolostone	Very Strong
8	2	31.5	A	20.2	47.4	54.5	6.2	148.9	Dolostone	Very Strong
9	3	31.4	A	19.1	47.4	59.9	5.5	130.8	Dolostone	Very Strong
10	3	32.1	D	17.7	47.4	191.1	7.3	175.4	Dolostone	Very Strong
11	3	32.4	A	18.6	47.4	65.0	5.0	119.4	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	18426	**Date Drilled:**	18-Apr-18
Client:	WSP	**Date Tested:**	02-May-18
Project Name:	Welland River Twin Bridge Structures	**Tester:**	GA
Core Size:	NQ **BH No :** 18-19	**Reviewed by:**	GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	29.1	D	13.1	47.4	130.1	5.4	129.9	Dolostone	Very Strong
2	1	29.5	A	25.6	47.4	63.1	7.0	167.9	Dolostone	Very Strong
3	1	29.9	A	21.8	47.4	54.2	6.7	161.5	Dolostone	Very Strong
4	2	30.2	D	24.1	47.4	147.4	9.9	238.5	Dolostone	Very Strong
5	2	30.5	A	24.9	47.4	62.7	6.9	164.5	Dolostone	Very Strong
6	2	30.8	D	27.8	47.4	132.1	11.5	274.8	Dolostone	Extremely Strong
7	2	31.2	A	25.7	47.4	66.4	6.8	162.2	Dolostone	Very Strong
8	2	31.5	D	22.0	47.3	161.2	9.1	217.7	Dolostone	Very Strong
9	3	31.7	D	21.0	47.4	101.3	8.7	208.0	Dolostone	Very Strong
10	3	32.2	D	23.7	47.4	167.4	9.8	234.1	Dolostone	Very Strong
11	3	32.5	A	12.1	47.4	67.8	3.1	75.2	Dolostone	Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 18426 **Date Drilled:** 05-Apr-18
Client: WSP **Date Tested:** 18-Apr-18

Project Name: Welland River Twin Bridge Structures **Tester:** GA
Core Size: NQ **BH No :** 18-20 **Reviewed by:** GRL

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	29.4	D	21.5	47.5	102.2	8.9	212.6	Dolostone	Very Strong
2	1	29.7	A	12.2	47.4	57.6	3.6	86.3	Dolostone	Strong
3	1	30.0	D	22.7	47.4	136.2	9.3	223.9	Dolostone	Very Strong
4	1	30.3	A	23.2	47.4	56.5	6.9	166.2	Dolostone	Very Strong
5	1	30.6	D	20.8	47.4	158.6	8.6	206.0	Dolostone	Very Strong
6	2	30.8	D	26.6	47.4	110.4	11.0	263.4	Dolostone	Extremely Strong
7	2	31.1	A	24.1	47.4	68.6	6.2	148.6	Dolostone	Very Strong
8	2	31.4	D	10.8	47.4	152.3	4.4	106.6	Dolostone	Very Strong
9	2	31.8	A	11.8	47.4	64.0	3.2	76.9	Dolostone	Strong
10	2	32.1	D	10.4	47.4	144.5	4.3	102.6	Dolostone	Very Strong

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24. (Zhang, 2005)

Last Modified: September 14, 2016

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
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CLIENT NAME: THURBER ENGINEERING LTD
SUITE 103, 2010 WINSTON PARK DRIVE
OAKVILLE, ON L6H5R7
(905) 829-8666

ATTENTION TO: Abdul Nasri

PROJECT: 18426 Welland River Bridge

AGAT WORK ORDER: 18T339355

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: May 22, 2018

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.

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MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
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Certificate of Analysis

AGAT WORK ORDER: 18T339355

PROJECT: 18426 Welland River Bridge

CLIENT NAME: THURBER ENGINEERING LTD | **ATTENTION TO: Abdul Nasri**

SAMPLING SITE: | **SAMPLED BY:**

Corrosivity Package

DATE RECEIVED: 2018-05-15 | **DATE REPORTED: 2018-05-22**

			SAMPLE DESCRIPTION:	18-03 SS#5		18-10 SS#9	18-16 SS#12	18-20 SS#11
			SAMPLE TYPE:	Soil		Soil	Soil	Soil
			DATE SAMPLED:	2018-04-09		2018-03-16	2018-03-23	2018-04-05
Parameter	**Unit**	**G / S**	**RDL**	**9246635**	**RDL**	**9246636**	**9246637**	**9246638**
Sulfide (S2-)	%		0.05	<0.05	0.05	0.32	0.15	0.42
Chloride (2:1)	µg/g	NA	4	264	2	39	24	15
Sulphate (2:1)	µg/g		4	1190	2	197	260	310
pH (2:1)	pH Units		NA	7.66	NA	7.86	7.95	7.88
Electrical Conductivity (2:1)	mS/cm	1.4	0.005	1.35	0.005	0.331	0.352	0.407
Resistivity (2:1)	ohm.cm		1	741	1	3020	2840	2460
Redox Potential (2:1)	mV		5	196	5	185	175	164

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard: Refers to Table 3: Full Depth Generic Site Condition Standards in a Non-Potable Ground Water Condition - Soil - Industrial/Commercial/Community Property Use - Medium and Fine Textured Soils
Guideline values are for general reference only. The guidelines provided may or may not be relevant for the intended use. Refer directly to the applicable standard for regulatory interpretation.

9246635 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam

Samples were received and analyzed past hold time.

Elevated RDL indicates the degree of sample dilution prior to the analysis in order to keep analytes within the calibration range of the instrument and to reduce matrix interference.

9246636-9246638 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).
*Sulphide analyzed at AGAT 5623 McAdam

Samples were received and analyzed past hold time.

Certified By: Amanjot Bhela

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
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Quality Assurance

CLIENT NAME: THURBER ENGINEERING LTD **AGAT WORK ORDER: 18T339355**
PROJECT: 18426 Welland River Bridge **ATTENTION TO: Abdul Nasri**
SAMPLING SITE: **SAMPLED BY:**

Soil Analysis

RPT Date: May 22, 2018			DUPLICATE				REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD	Method Blank	Measured Value	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper	Recovery	Acceptable Limits Lower	Acceptable Limits Upper
Corrosivity Package															
Sulfide (S2-)	9246635	9246635	< 0.05	< 0.05	NA	< 0.05	95%	80%	120%						
Chloride (2:1)	9237761		1420	1450	2.1%	< 2	102%	80%	120%	104%	80%	120%	94%	70%	130%
Sulphate (2:1)	9237761		37	37	0.0%	< 2	97%	80%	120%	102%	80%	120%	106%	70%	130%
pH (2:1)	9252079		8.47	8.53	0.7%	NA	101%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	9245893		0.245	0.244	0.4%	< 0.005	95%	90%	110%	NA			NA		
Redox Potential (2:1)	9237761		184	181	1.6%	< 5	103%	70%	130%	NA			NA		

Comments: NA signifies Not Applicable.
Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Certified By: Amanjot Bhela

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Method Summary

CLIENT NAME: THURBER ENGINEERING LTD | **AGAT WORK ORDER: 18T339355**

PROJECT: 18426 Welland River Bridge | **ATTENTION TO: Abdul Nasri**

SAMPLING SITE: | **SAMPLED BY:**

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulfide (S2-)	MIN-200-12025	ASTM E1915-09	GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B,SSA #5 Part 3	CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE

1 Bag

AGAT Laboratories

SR6

5835 Coopers Avenue
Mississauga Ontario L4Z 1Y2
Ph: 905 712 5100 Fax: 905 712 5122
webearth agatlabs com

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Laboratory Use Only

Work Order #: 18T 339355

Cooler Quantity:

Arrival Temperatures: 2.9 2.9 2.7 / 3.1 3.0 2.7

Custody Seal Intact: ☐ Yes ☐ No ☐ N/A

Notes:

Report Information:

Company: Thurber Engineering Ltd.
Contact: Geoff Lay
Address: 103-2010 Winston Park Drive
Oakville, ON L6H 5R7
Phone: (905)829-8666 Fax: (905)829-1166
Reports to be sent to:
1. Email: glay@thurber.ca
2. Email:

Regulatory Requirements: ☐ No Regulatory Requirement
(Please check all applicable boxes)

☑ Regulation 153/04
Table 3 (Indicate One)
☑ Ind/Com
☐ Res/Park
☐ Agriculture
Soil Texture (Check One)
☐ Coarse
☑ Fine

☐ Sewer Use
☐ Sanitary
☐ Storm
Region ____ (Indicate One)
☐ MISA

☐ Regulation 558
☐ CCME
☐ Prov Water Quality Objectives (PWQO)
☐ Other ____ (Indicate One)

Is this submission for a Record of Site Condition? ☐ Yes ☐ No

Report Guideline on Certificate of Analysis ☐ Yes ☐ No

Turnaround Time (TAT) Required:

Regular TAT ☑ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)
☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

OR Date Required (Rush Surcharges May Apply):

Please provide prior notification for rush TAT
**TAT is exclusive of weekends and statutory holidays*

For 'Same Day' analysis, please contact your AGAT CPM

Project Information:

Project: 18426 Welland River Bridges
Site Location: QEW / Welland River
Sampled By: ES
AGAT Quote #: PO:
Please note: If quotation number is not provided, client will be billed full price for analysis

Invoice Information: Bill To Same: Yes ☑ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

- B Biota
- GW Ground Water
- O Oil
- P Paint
- S Soil
- SD Sediment
- SW Surface Water

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI (Y/N)	Metals and Inorganics	O. Reg 153: ☐ All Metals ☐ 153 Metals (excl Hydrides) ☐ Hydride Metals ☐ 153 Metals (incl Hydrides)	ORPs: ☐ B-HWS ☐ Cl ☐ CN ☐ Cr6+ ☐ EC ☐ FOC ☐ Hg ☐ pH ☐ SAR	Full Metals Scan	Regulation/Custom Metals	Nutrients: ☐ TP ☐ NH3 ☐ TKN ☐ NO3 ☐ NO2 ☐ NO3+NO2	Volatiles: ☐ VOC ☐ BTEX ☐ THM	CCME Fractions 1 to 4	ABNs	PAHs	PCBs: ☐ Total ☐ Aroclors	Organochlorine Pesticides	TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Sewer Use	Corrosivity
18-03 SS#5	04/09/2018		1	S																	☑
18-10 SS#9	03/16/2018		1	S																	☑
18-16 SS#12	03/232018		1	S																	☑
18-20 SS#11	04/05/2018		1	S																	☑

Samples Relinquished By (Print Name and Sign): Geoff Lay — Date: April 19/18 — Time:
Samples Relinquished By (Print Name and Sign): — Date: — Time:
Samples Relinquished By (Print Name and Sign): — Date: 05/15/18 — Time: 10 35
Samples Received By (Print Name and Sign): — Date: 05/15/18 — Time: 940
Samples Received By (Print Name and Sign): — Date: — Time:
Samples Received By (Print Name and Sign): — Date: — Time:

Page ____ of ____

Nº:

Pink Copy - Client I Yellow Copy - AGAT I White Copy- AGAT

Date Issued February 22 2017

DRAFT

PRESENTATION OF SITE INVESTIGATION RESULTS

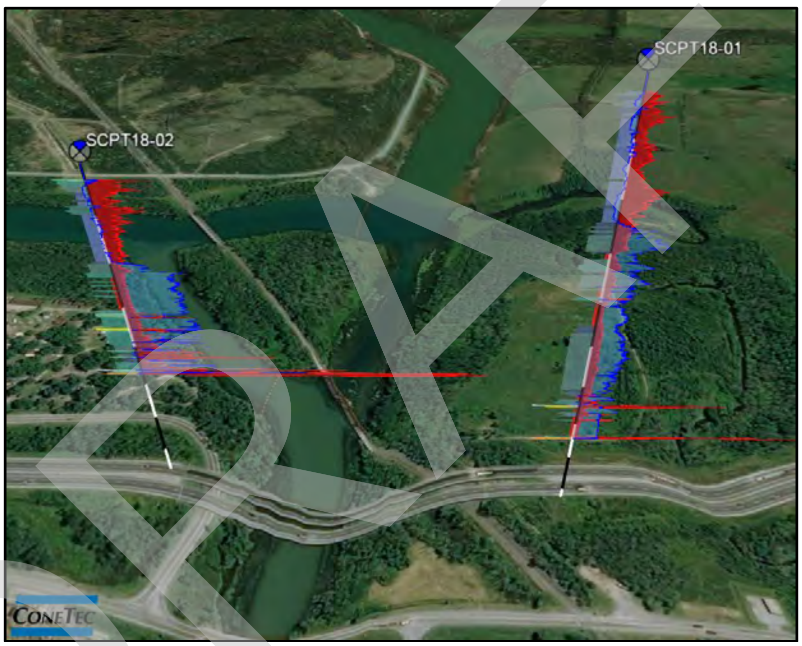
QEW at Welland River Bridge

Prepared for:

Thurber Engineering Ltd.

ConeTec Job No: 18-05043

Project Start Date: 13-Jul-2018
Project End Date: 13-Jul-2018
Report Date: 18-Jul-2018



Prepared by:

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Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Thurber Engineering Ltd. on Queen Elizabeth Way at the Welland River Bridge, Niagara Falls. The program consisted of two seismic cone penetration tests (SCPTs).

Project Information

Project	
Client	Thurber Engineering Ltd.
Project	QEW at Welland River Bridge
ConeTec project number	18-05043

A plan view from Google Earth including the CPT test locations is presented below.

Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
SCPT	Consumer grade GPS	32617

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Soil behaviour type (SBT) scatter plots, seismic plots and advanced plots are provided in the data release package.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
545:T1500F15U500	545	15	225	1500	15	500
Cone 545 was used for all CPT soundings.						

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Soils were classified as either drained or undrained based on the Normalized Soil Behaviour Type Chart (SBT Q_{tn}) (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures – clayey silt to silty clay (zone 4).

Limitations

This report has been prepared for the exclusive use of Thurber Engineering Ltd. (Client) for the project titled “QEW at Welland River Bridge”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

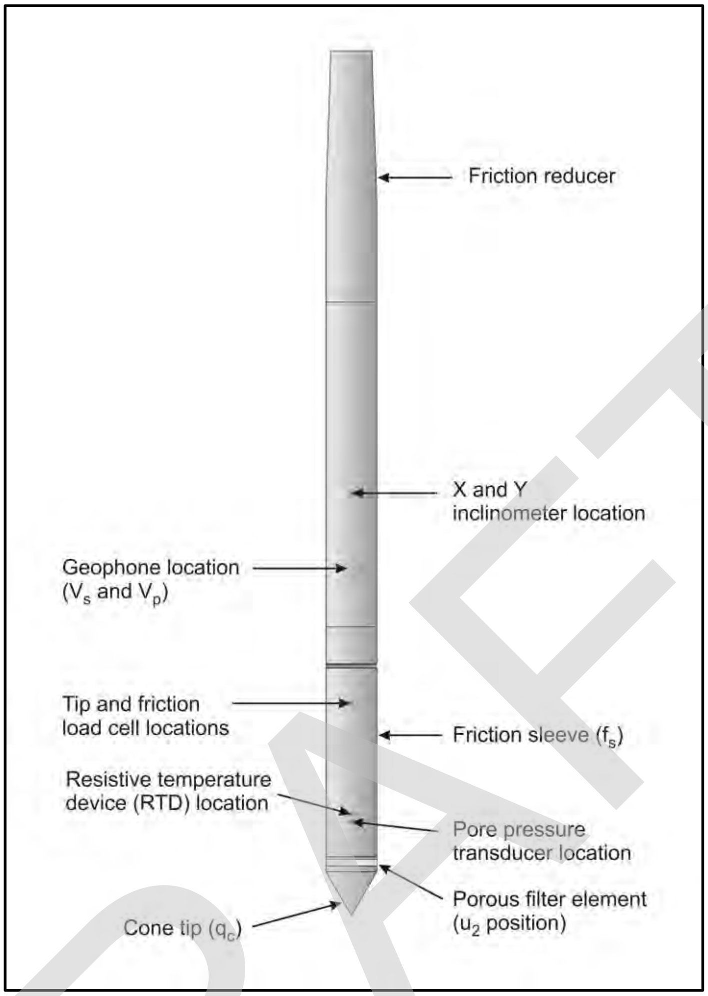

Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1\text{-}a) \bullet u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (Vp) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

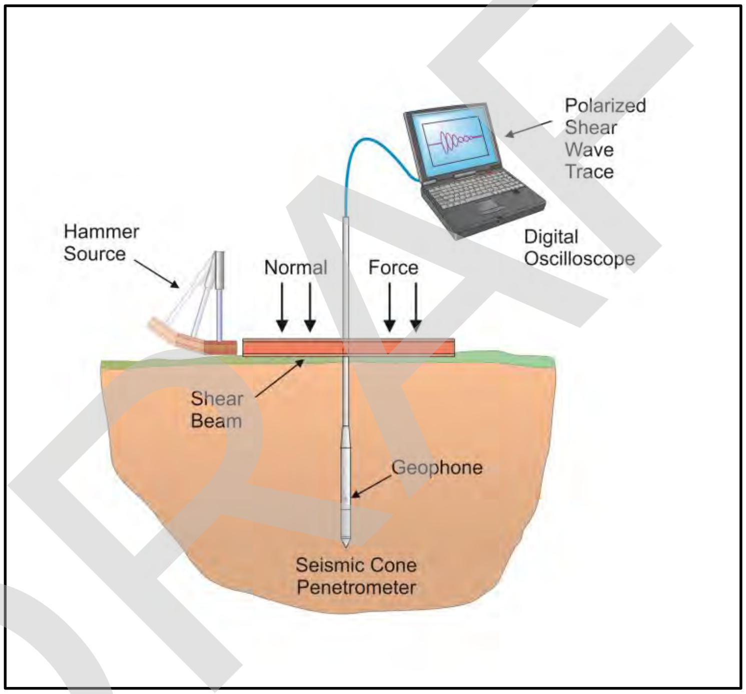

Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

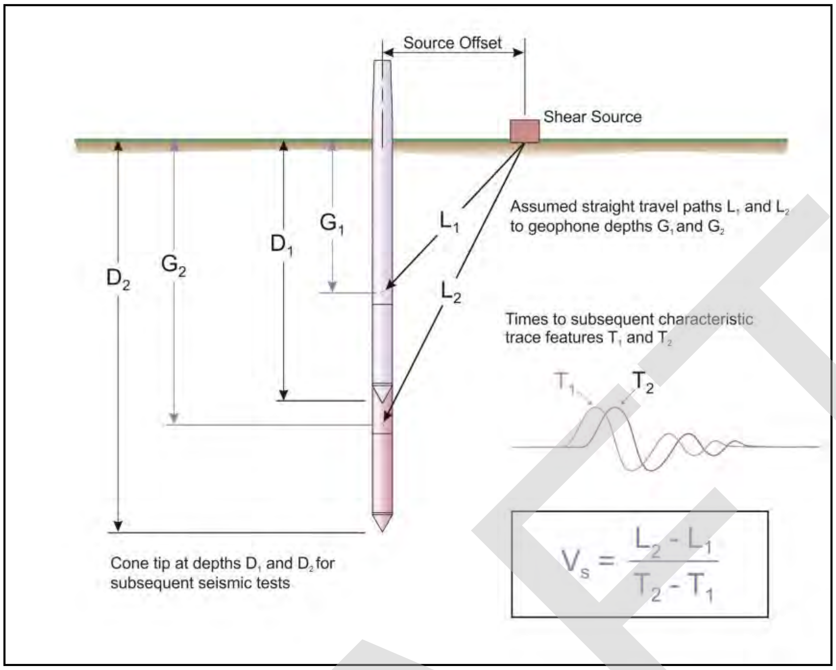

Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ (30m)}{\sum(layer\ traveltimes)}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

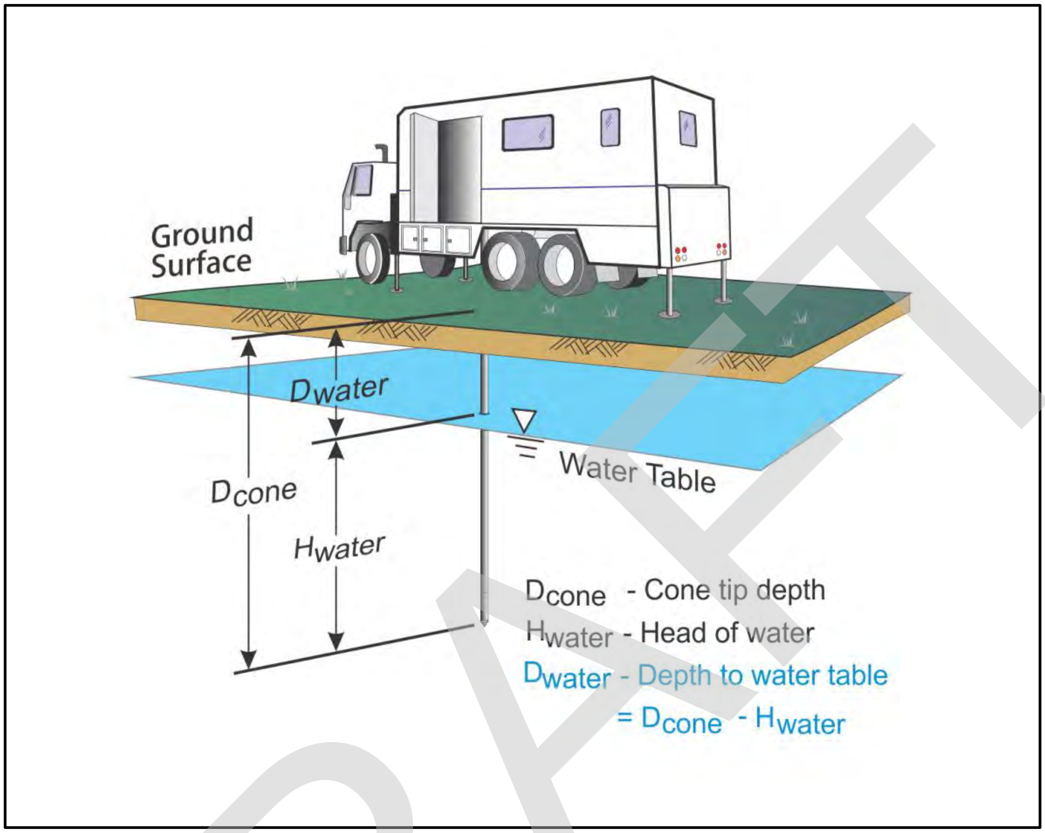

Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

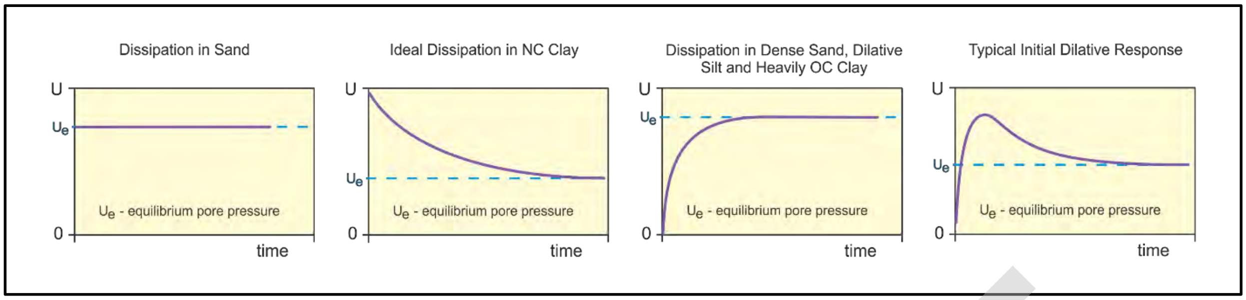

Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100}. In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100}. A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

T* is the dimensionless time factor (Table Time Factor)
a is the radius of the cone
I_r is the rigidity index
t is the time at the degree of consolidation

Table Time Factor. T* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
T* (u_2)	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50}, dissipation tests must be taken to a pressure less than u_{50}. The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100}. To estimate u_{50}, both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50}. In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r, the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50}, other methods should be applied to confirm the results for c_h.

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

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Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, “National shear wave measurement guidelines for Canadian seismic site assessment”, GeoManitoba 2012, Sept 30 to Oct 2, Winnipeg, Manitoba.

Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “Cone Penetration Testing in Geotechnical Practice”, Blackie Academic and Professional.

Mayne, P.W., 2013, “Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests”, Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420.

Mayne, P.W., 2014, “Interpretation of geotechnical parameters from seismic piezocone tests”, CPT’14 Keynote Address, Las Vegas, NV, May 2014.

Mayne, P.W. and Peuchen, J., 2012, “Unit weight trends with cone resistance in soft to firm clays”, Geotechnical and Geophysical Site Characterization *4,* Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27: 151-158.

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Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

Teh, C.I., and Houlsby, G.T., 1991, “An analytical study of the cone penetration test in clay”, Geotechnique, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), OCR and N1(60)Ic
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Time Domain Traces
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

Job No: 18-05043
Client: Thurber Engineering Ltd.
Project: QEW at Welland River Bridge
Start Date: 13-Jul-2018
End Date: 13-Jul-2018

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting (m)	Refer to Notation Number
SCPT18-01	18-05043_SP01	13-Jul-2018	545:T1500F15U500	12.2	37.075	4767443	652987	
SCPT18-02	18-05043_SP02	13-Jul-2018	545:T1500F15U500	6.9	23.675	4767761	652985	

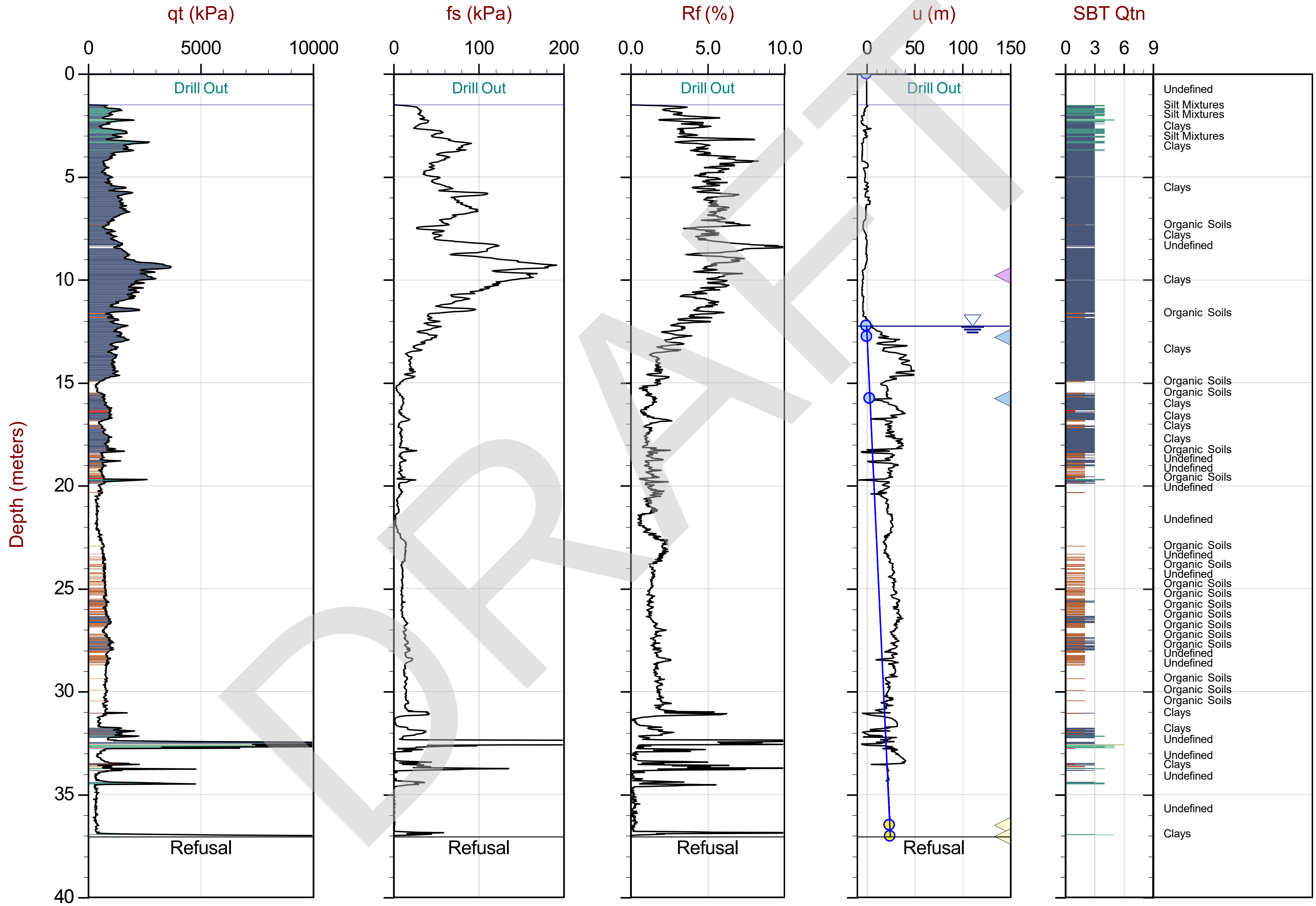
1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device in datum WGS84/UTM Zone 17 North.

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 2018-07-13 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500



Max Depth: 37.075 m / 121.64 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05043_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4767443m E:652987m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq assumed | Hydrostatic Line

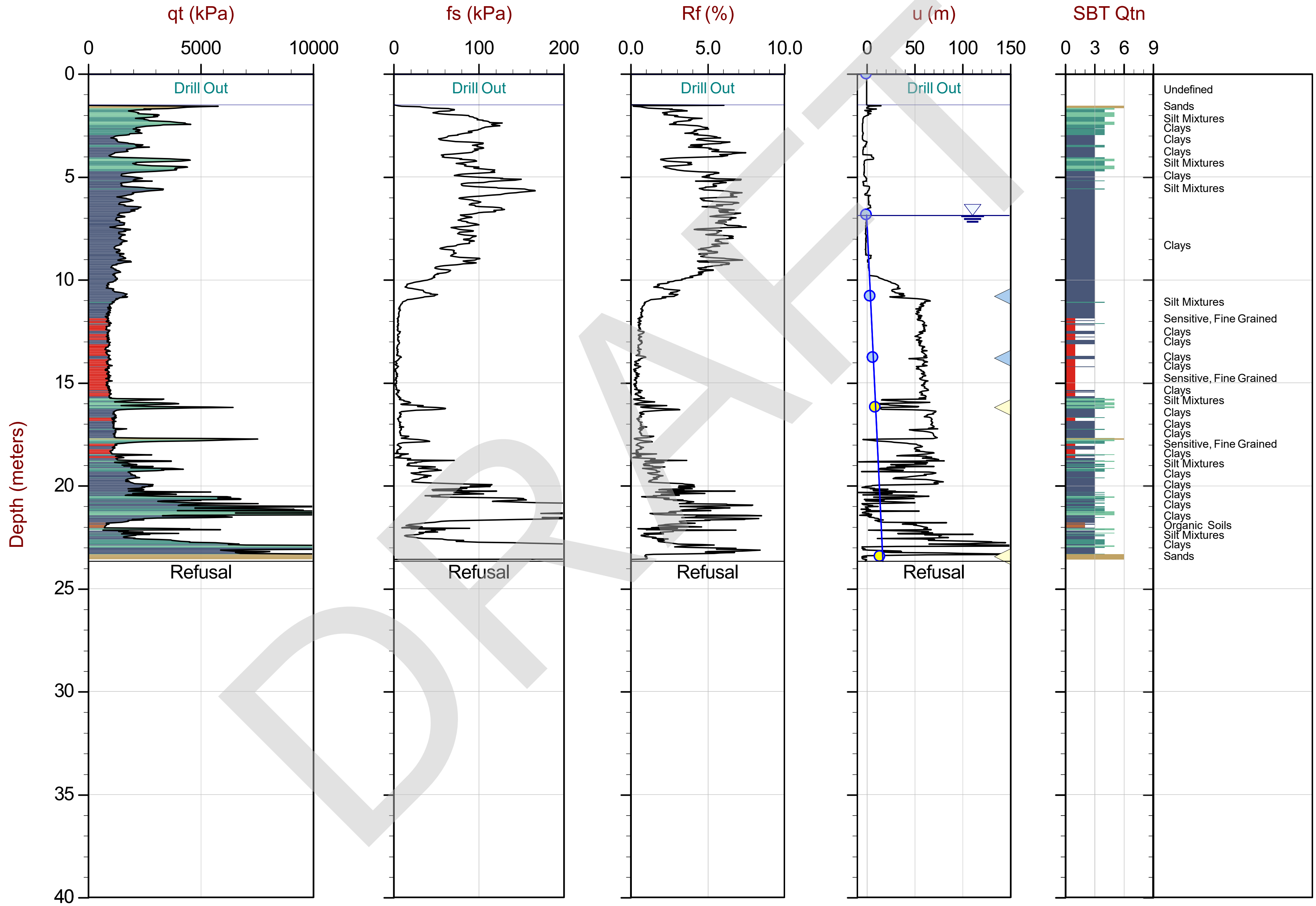
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 2018-07-13 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500



Max Depth: 23.675 m / 77.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05043_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4767761m E: 652985m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq assumed Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

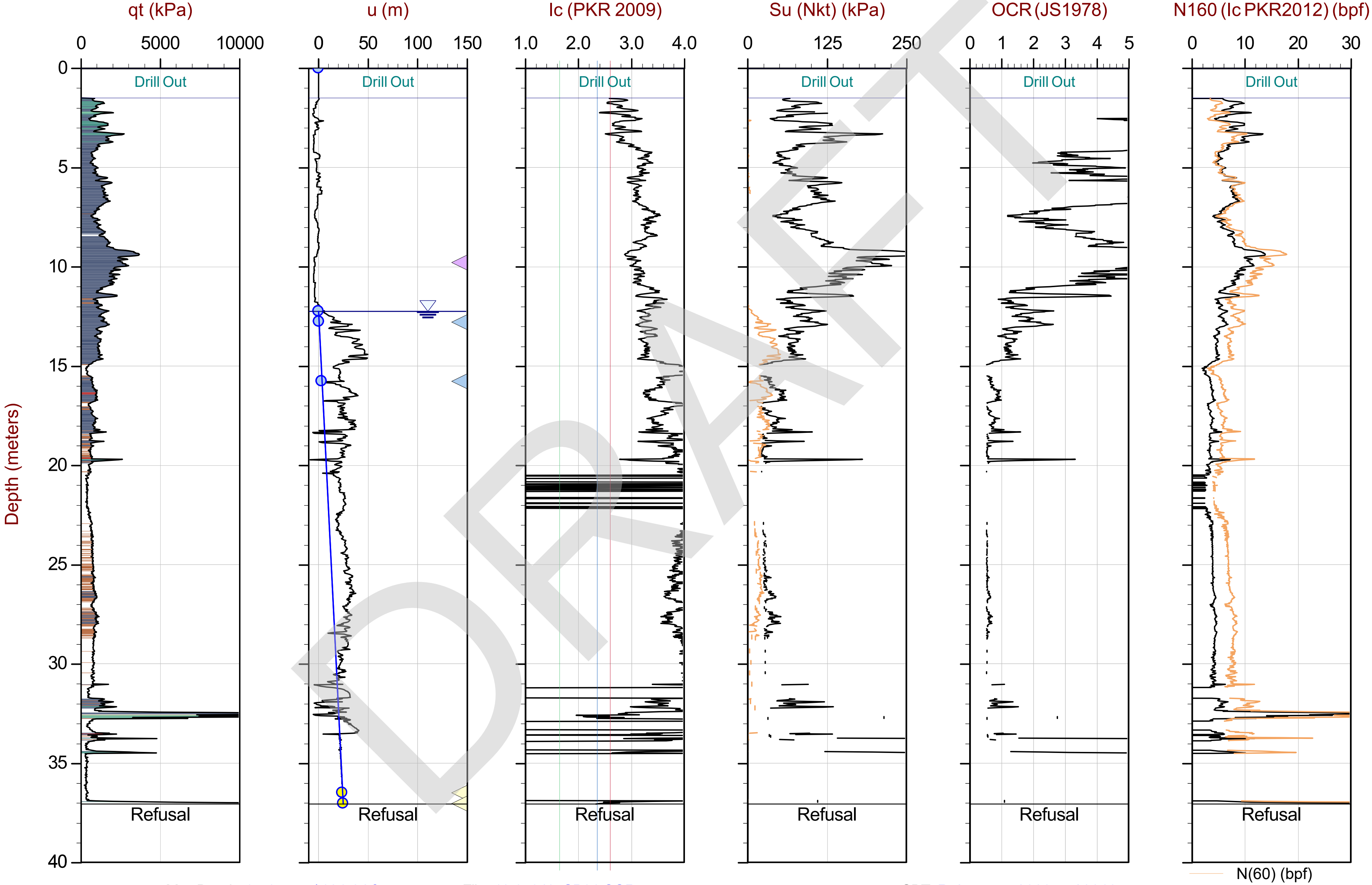
Advanced Cone Penetration Test Plots with Ic, Su (Nkt), OCR and N1(60)Ic

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 2018-07-13 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500



Max Depth: 37.075 m / 121.64 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05043_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 12.5 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N:4767443m E:652987m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq assumed Hydrostatic Line

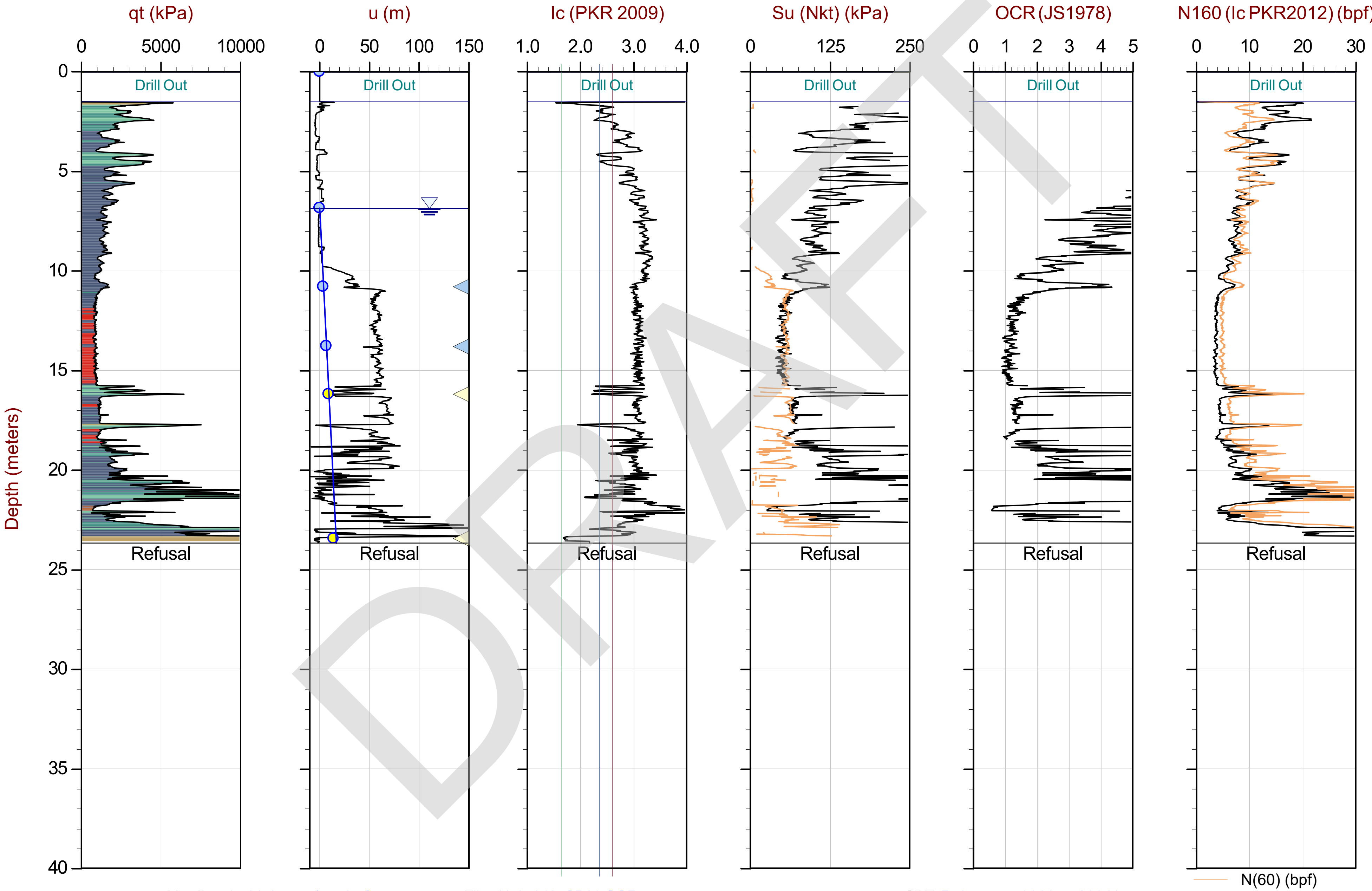
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

Thurber Engineering

Job No: 18-05043
Date: 2018-07-13 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500



Max Depth: 23.675 m / 77.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05043_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 12.5 / 9.0

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4767761m E: 652985m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq assumed | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Tabular Results

Job No: 18-05043
Client: Thurber Engineering Ltd.
Project: QEW at Welland River Bridge
Sounding ID: SCPT18-01
Date: 13-Jul-2018

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.78	1.58	1.67			
2.77	2.57	2.63	0.96	6.72	142
3.77	3.57	3.61	0.98	7.16	137
4.78	4.58	4.61	1.00	6.22	161
5.78	5.58	5.61	0.99	5.96	167
6.78	6.58	6.60	1.00	6.12	163
7.78	7.58	7.60	1.00	6.42	155
8.78	8.58	8.60	1.00	5.73	174
9.78	9.58	9.60	1.00	3.51	285
10.78	10.58	10.59	1.00	4.40	227
11.78	11.58	11.59	1.00	3.62	276
12.78	12.58	12.59	1.00	3.96	252
13.78	13.58	13.59	1.00	4.71	212
14.78	14.58	14.59	1.00	5.33	187
15.78	15.58	15.59	1.00	5.71	175
16.77	16.57	16.58	0.99	5.80	171
17.77	17.57	17.58	1.00	4.99	200
18.77	18.57	18.58	1.00	3.71	269
19.77	19.57	19.58	1.00	3.77	265
20.77	20.57	20.58	1.00	5.52	181
21.77	21.57	21.58	1.00	4.64	216
22.77	22.57	22.58	1.00	4.04	247
23.77	23.57	23.58	1.00	4.74	211
24.77	24.57	24.58	1.00	4.94	202
25.77	25.57	25.58	1.00	4.29	233
26.77	26.57	26.58	1.00	4.04	248
27.77	27.57	27.58	1.00	4.70	213
28.77	28.57	28.58	1.00	3.85	260
29.77	29.57	29.58	1.00	4.93	203

Job No: 18-05043
Client: Thurber Engineering Ltd.
Project: QEW at Welland River Bridge
Sounding ID: SCPT18-01
Date: 13-Jul-2018

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
30.77	30.57	30.57	1.00	3.15	317
31.77	31.57	31.57	1.00	1.76	570

Job No:	18-05043
Client:	Thurber Engineering Ltd.
Project:	QEW at Welland River Bridge
Sounding ID:	SCPT18-02
Date:	13-Jul-2018

Seismic Source:	Beam
Source Offset (m):	0.55
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.80	1.60	1.69			
2.80	2.60	2.66	0.97	6.69	144
3.80	3.60	3.64	0.98	6.80	145
4.80	4.60	4.63	0.99	6.03	164
5.80	5.60	5.63	0.99	5.47	182
6.80	6.60	6.62	1.00	5.38	185
7.80	7.60	7.62	1.00	5.86	170
8.80	8.60	8.62	1.00	6.05	165
9.80	9.60	9.62	1.00	6.07	164
10.80	10.60	10.61	1.00	6.27	159
11.80	11.60	11.61	1.00	5.94	168
12.80	12.60	12.61	1.00	5.16	193
13.80	13.60	13.61	1.00	4.60	217
14.80	14.60	14.61	1.00	5.27	190
15.80	15.60	15.61	1.00	4.66	214
16.80	16.60	16.61	1.00	4.22	237
17.80	17.60	17.61	1.00	4.38	228
18.80	18.60	18.61	1.00	3.42	292
19.80	19.60	19.61	1.00	2.46	406
20.80	20.60	20.61	1.00	2.41	415
21.77	21.57	21.58	0.97	1.84	526
22.80	22.60	22.61	1.03	2.05	502
23.68	23.48	23.49	0.88	1.26	699

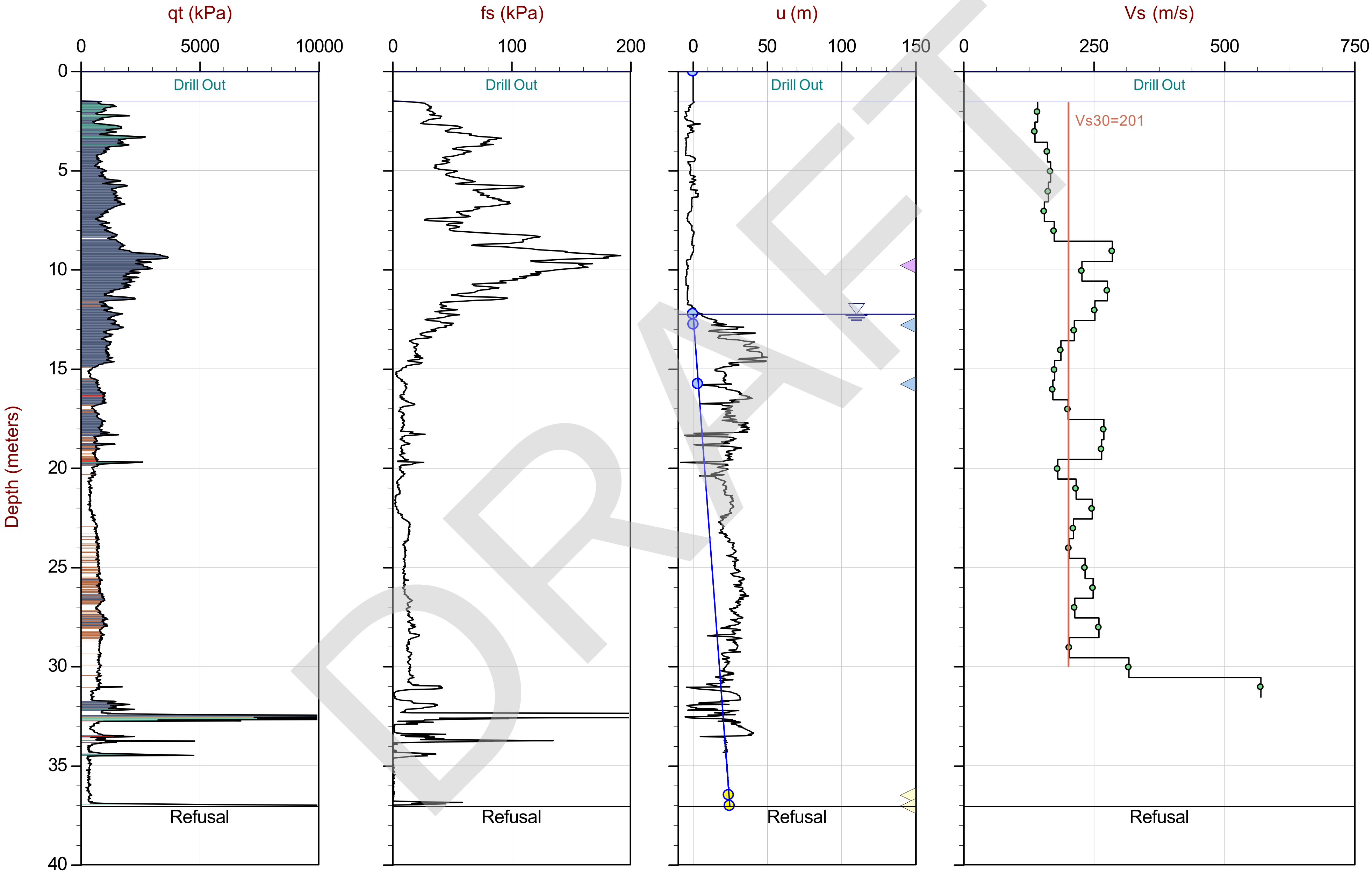
Seismic Cone Penetration Test Plots

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 2018-07-13 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500



Max Depth: 37.075 m / 121.64 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05043_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4767443m E: 652987m
Sheet No: 1 of 1

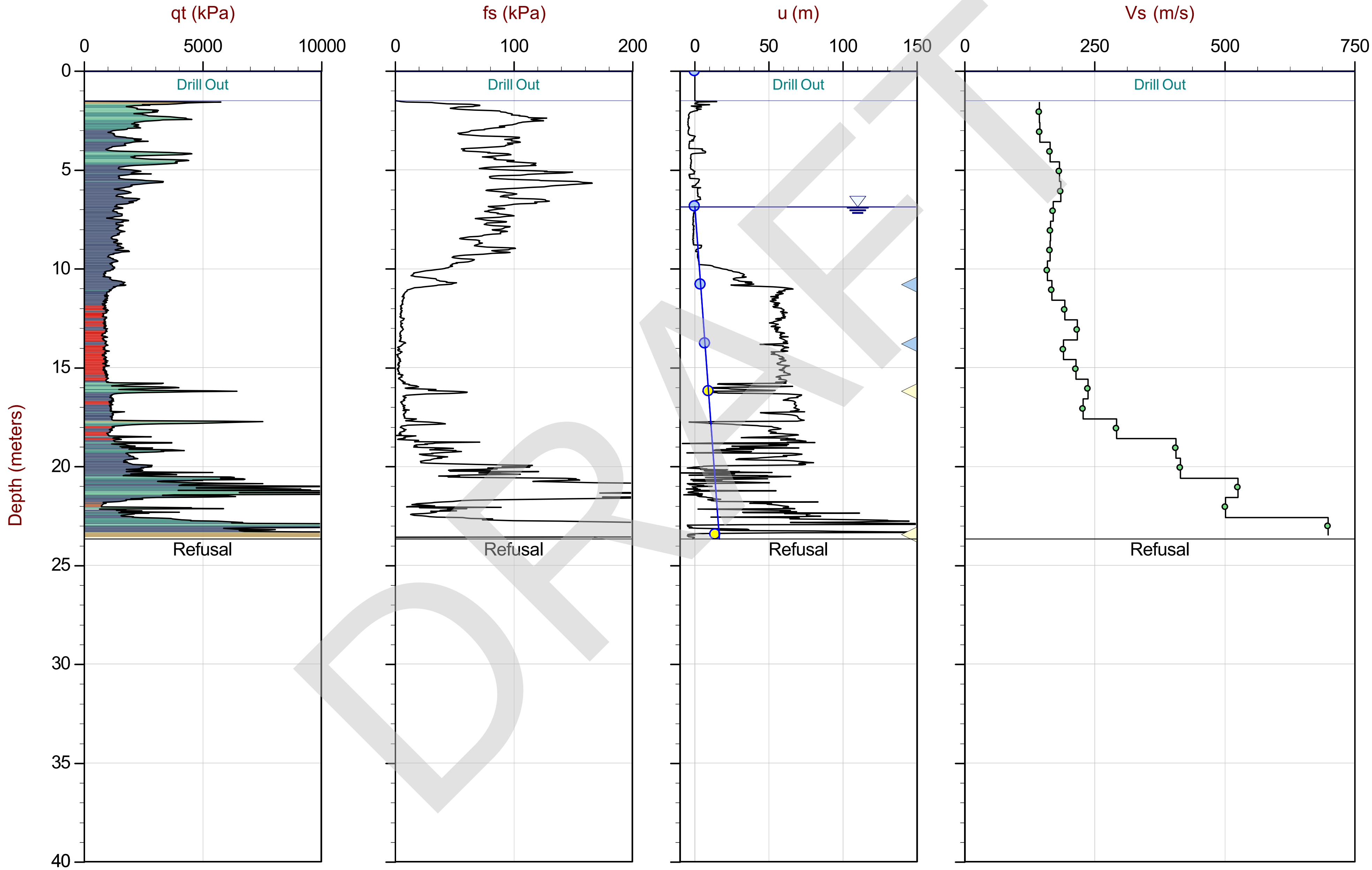
Equilibrium Pore Pressure (Ueq) Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq assumed Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Thurber Engineering

Job No: 18-05043
Date: 2018-07-13 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500



Max Depth: 23.675 m / 77.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05043_SP02.COR
Unit Wt: SBTQtn (PKR2009)

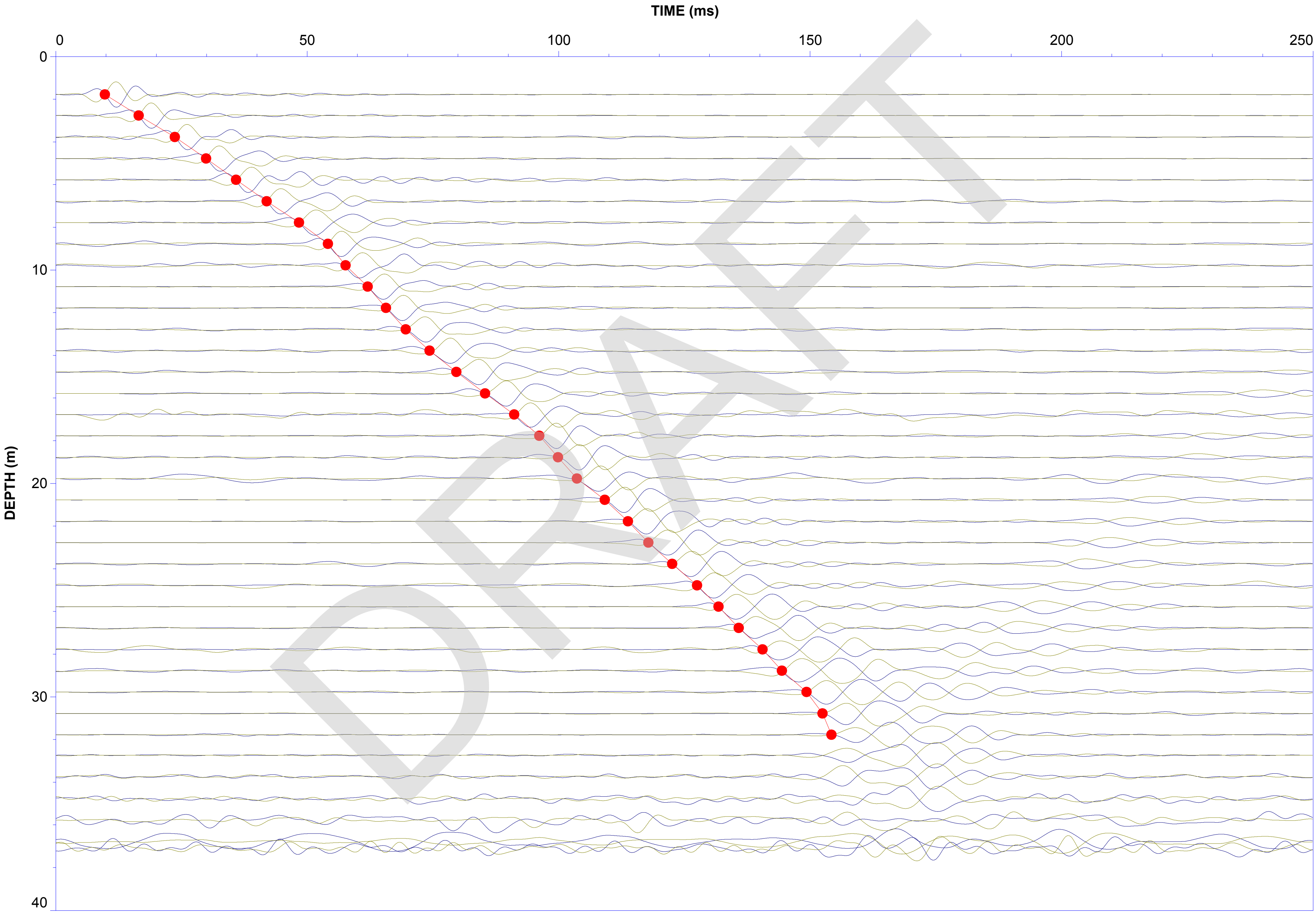
SBT: Robertson, 2009 and 2010
Coords: UTM 17N N: 4767761m E: 652985m
Sheet No: 1 of 1

Equilibrium Pore Pressure (Ueq) · Assumed Ueq · Dissipation, Ueq achieved · Dissipation, Ueq assumed · Hydrostatic Line

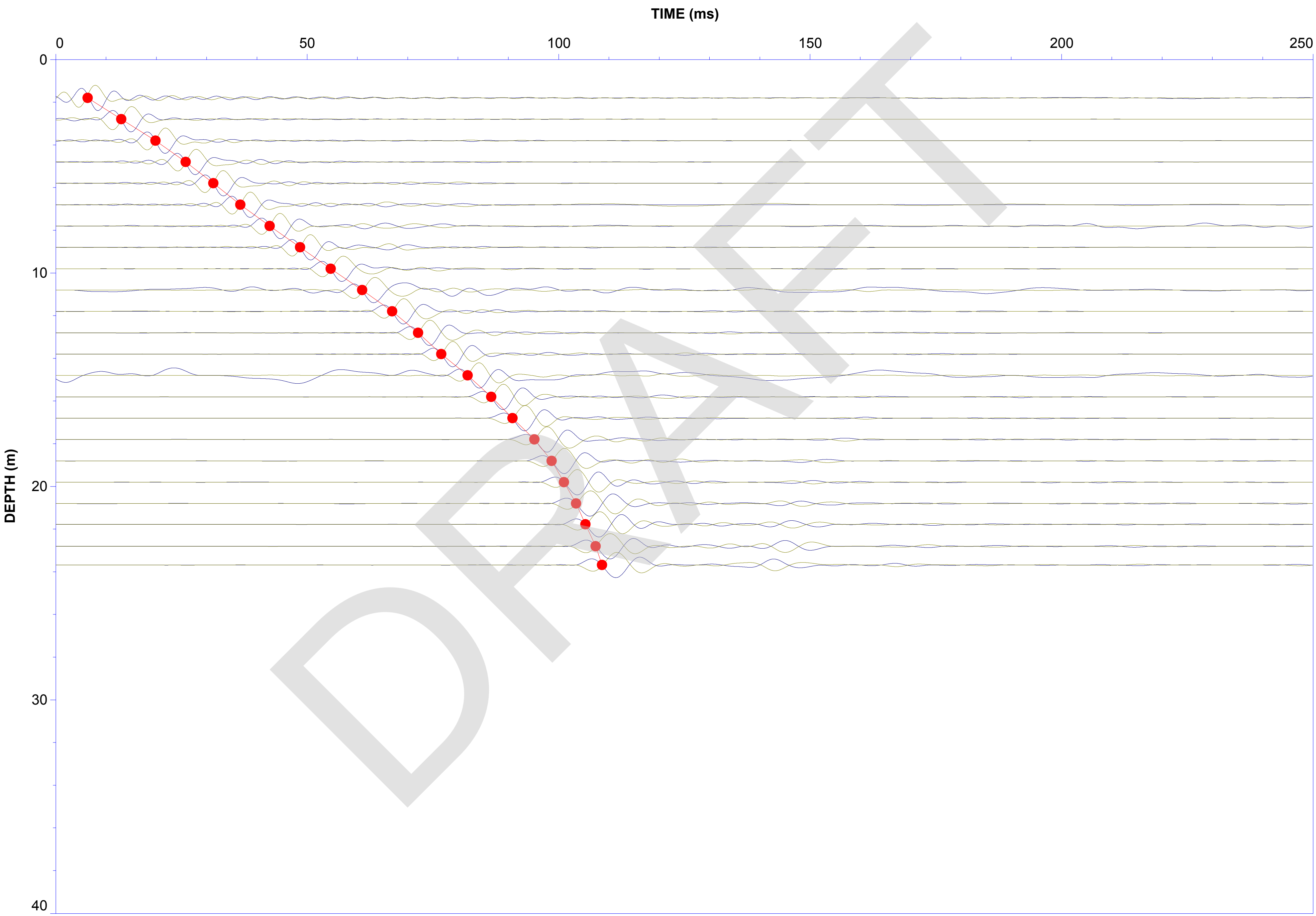
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Time Domain Traces

Job No: 18-05043	Client: Thurber Engineering Ltd.	Project Title: QEW at Welland River Bridge	Filter: 0-200 Hz	Souding: SCPT18-01	Date: 13-Jul-2018



CONETEC	Job No: 18-05043	Client: Thurber Engineering Ltd.	Project Title: QEW at Welland River Bridge	Filter: 0-200 Hz	Souding: SCPT18-02	Date: 13-Jul-2018



Soil Behaviour Type (SBT) Scatter Plots

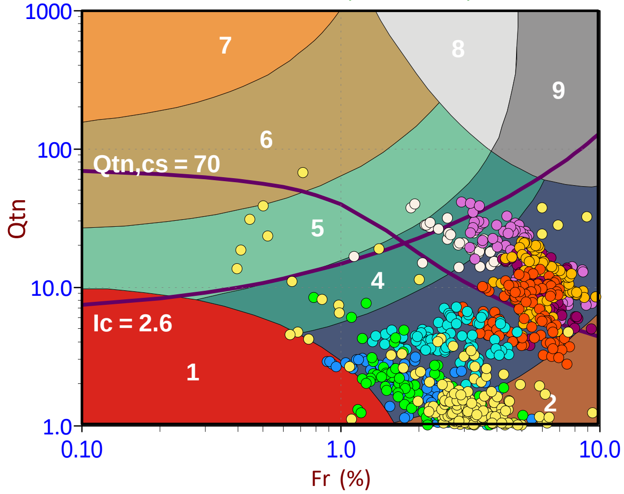
CONETEC

Thurber Engineering

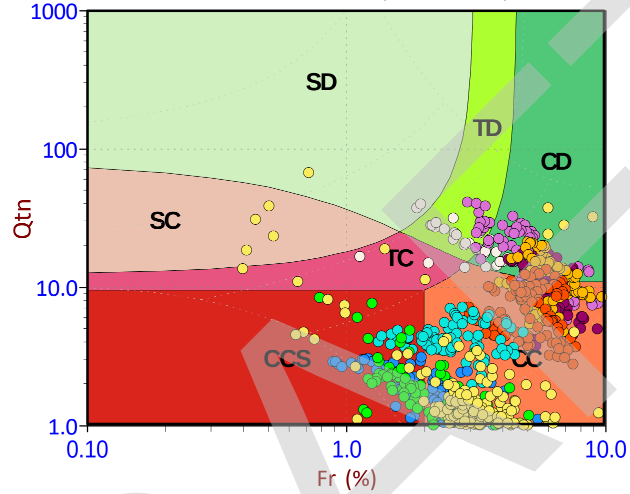
Job No: 18-05043
Date: 2018-07-13 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500

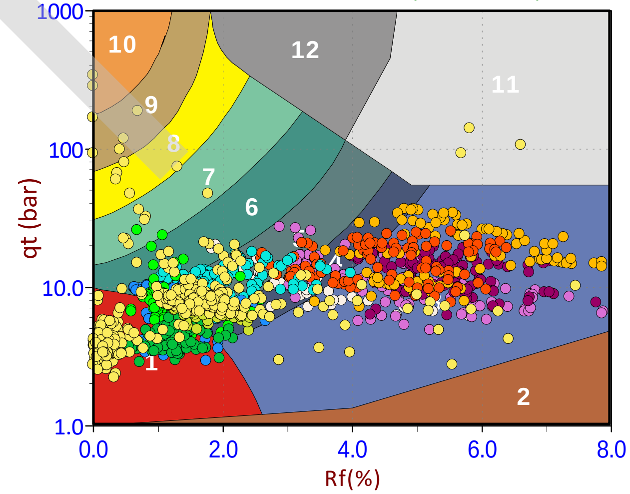
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



Depth Ranges

- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend

- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend

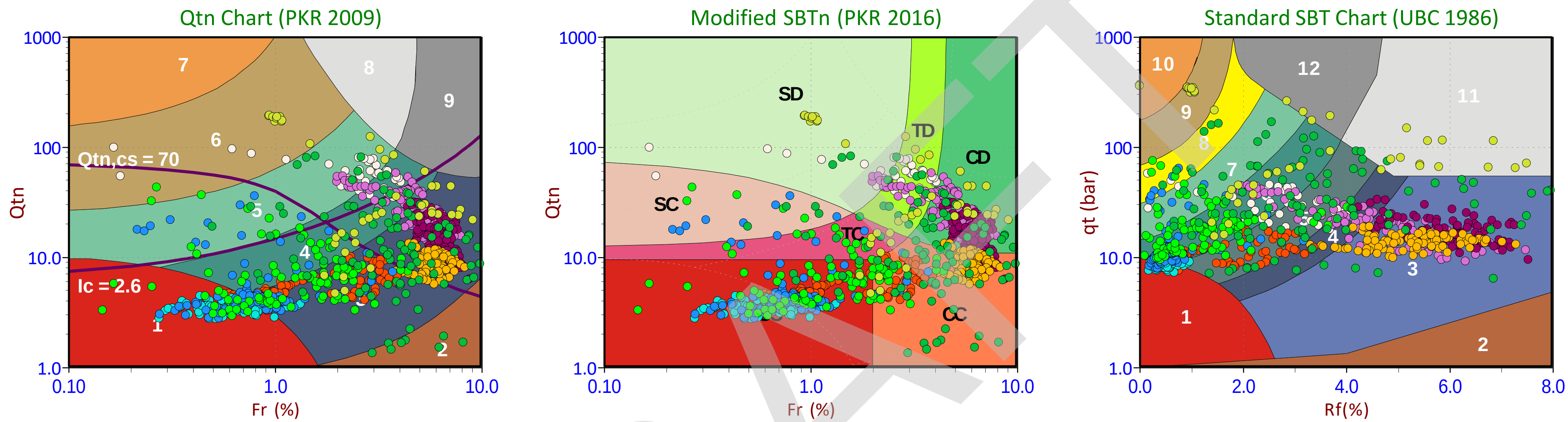
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend

- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Job No: 18-05043
Date: 2018-07-13 23:52
Site: Niagra Falls, ON
Sounding: SCPT18-02
Cone: 545:T1500F15U500

Thurber Engineering



Depth Ranges
- >0.0 to 2.5 m
- >2.5 to 5.0 m
- >5.0 to 7.5 m
- >7.5 to 10.0 m
- >10.0 to 12.5 m
- >12.5 to 15.0 m
- >15.0 to 17.5 m
- >17.5 to 20.0 m
- >20.0 to 22.5 m
- >22.5 to 25.0 m
- >25.0 m

Legend
- Sensitive, Fine Grained
- Organic Soils
- Clays
- Silt Mixtures
- Sand Mixtures
- Sands
- Gravelly Sand to Sand
- Stiff Sand to Clayey Sand
- Very Stiff Fine Grained

Legend
- CCS (Cont. sensitive clay like)
- CC (Cont. clay like)
- TC (Cont. transitional)
- SC (Cont. sand like)
- CD (Dil. clay like)
- TD (Dil. transitional)
- SD (Dil. sand like)

Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

Job No: 18-05043
Client: Thurber Engineering Ltd.
Project: QEW at Welland River Bridge
Start Date: 13-Jul-2018
End Date: 13-Jul-2018

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t_{50} [a] (s)	Assumed Rigidity Index (I_r)	c_h [b] (cm^2/min)
SCPT18-01	18-05043_SP01	15	300	9.775	Not Achieved					
SCPT18-01	18-05043_SP01	15	2605	12.775	Not Achieved		12.2	2429	100	0.3
SCPT18-01	18-05043_SP01	15	750	15.775	Not Achieved		12.2	494	100	1.4
SCPT18-01	18-05043_SP01	15	300	36.500	24.3	12.2				
SCPT18-01	18-05043_SP01	15	305	37.050	24.8	12.2				
SCPT18-02	18-05043_SP02	15	800	10.800	Not Achieved		6.9	486	100	1.4
SCPT18-02	18-05043_SP02	15	2150	13.800	Not Achieved		6.9	1555	100	0.5
SCPT18-02	18-05043_SP02	15	300	16.200	9.3	6.9				
SCPT18-02	18-05043_SP02	15	300	23.450	14.1	9.4				

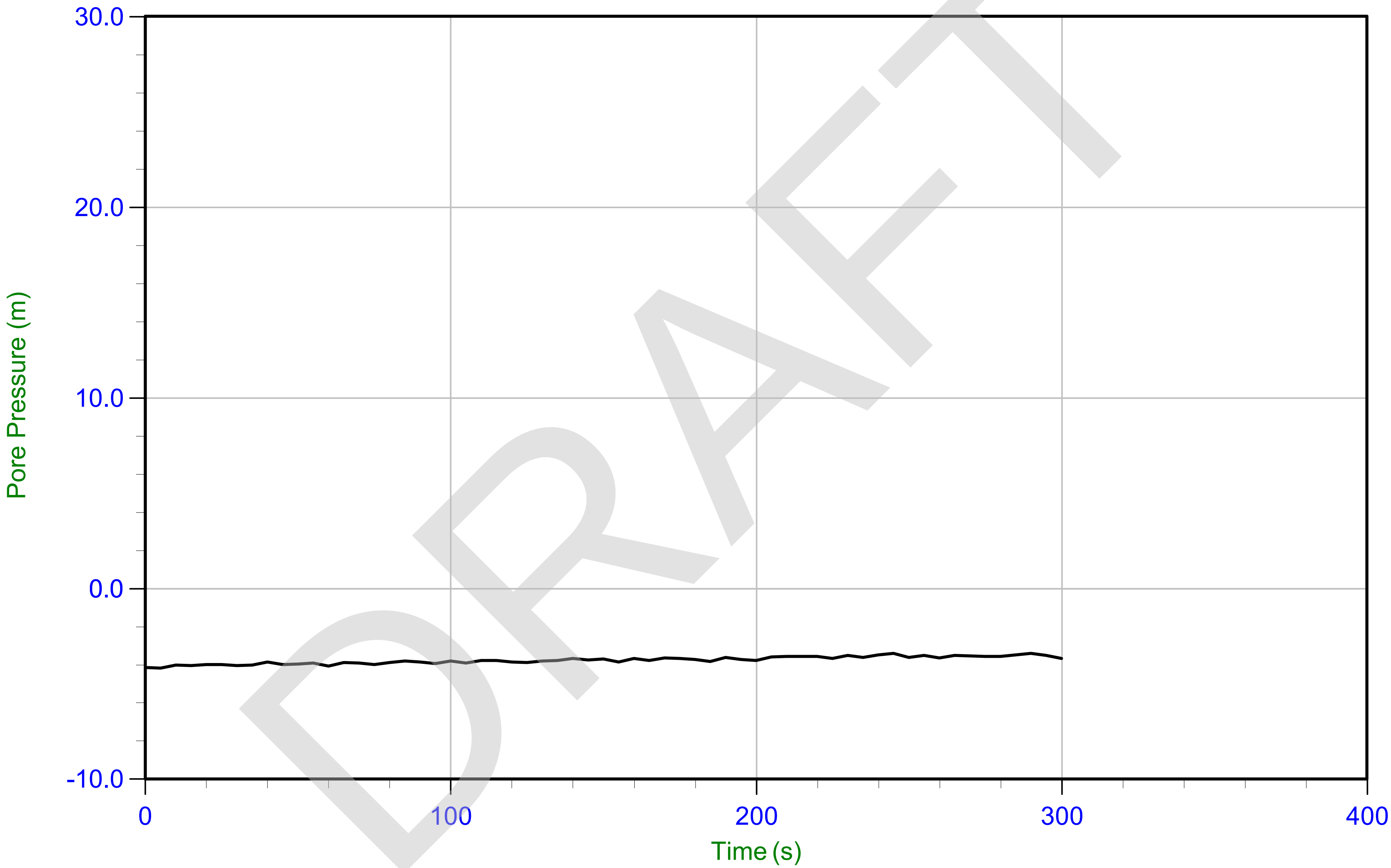
a. Time is relative to where umax occurred
b. Houlsby and Teh, 1991

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP01.PPF
Depth: 9.775 m / 32.070 ft
Duration: 300.0 s

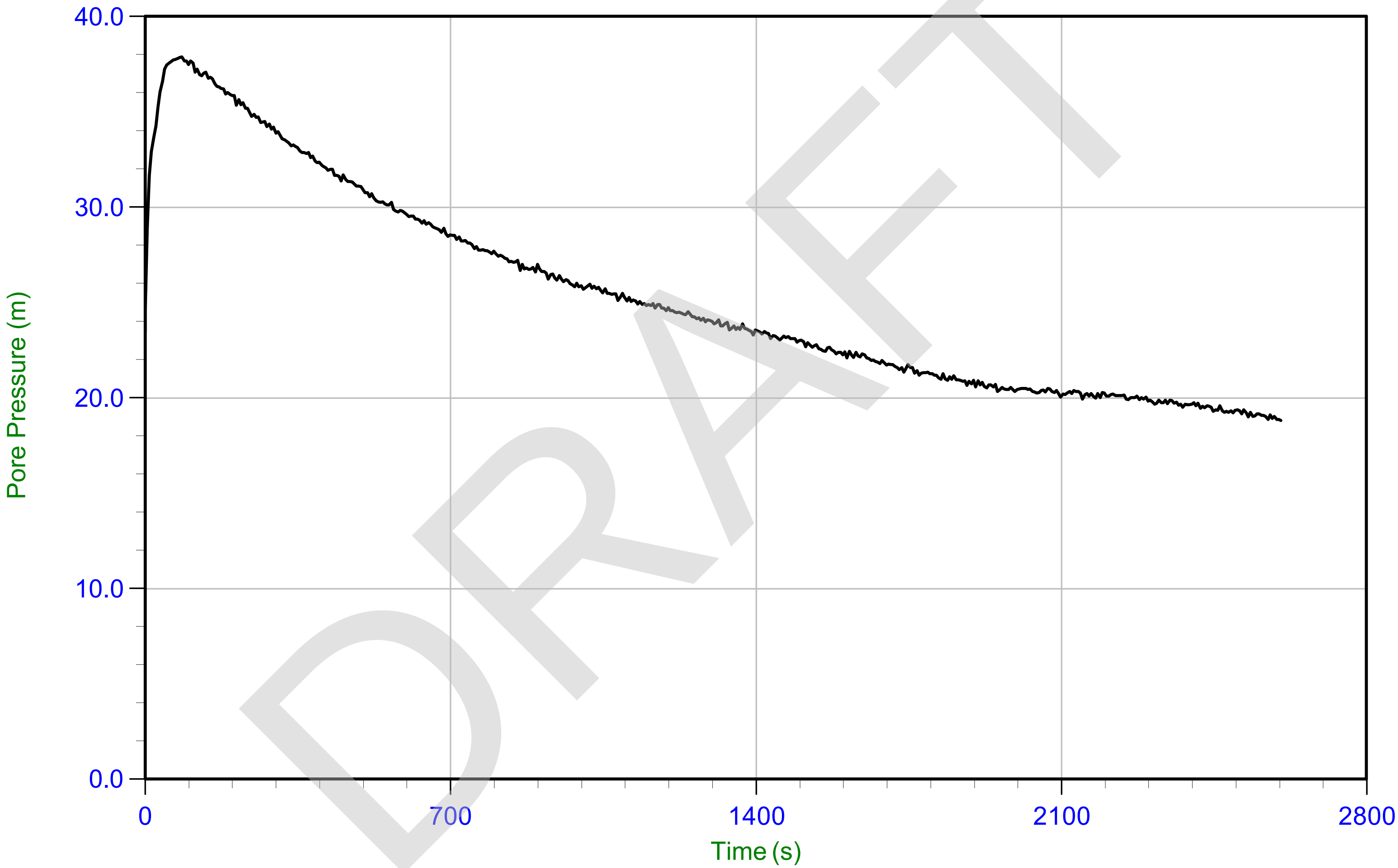
U Min: -4.2 m
U Max: -3.4 m

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP01.PPF
Depth: 12.775 m / 41.912 ft
Duration: 2605.0 s

U Min: 18.8 m
U Max: 37.9 m

WT: 12.244 m / 40.170 ft
Ueq: 0.5 m
U(50): 19.20 m

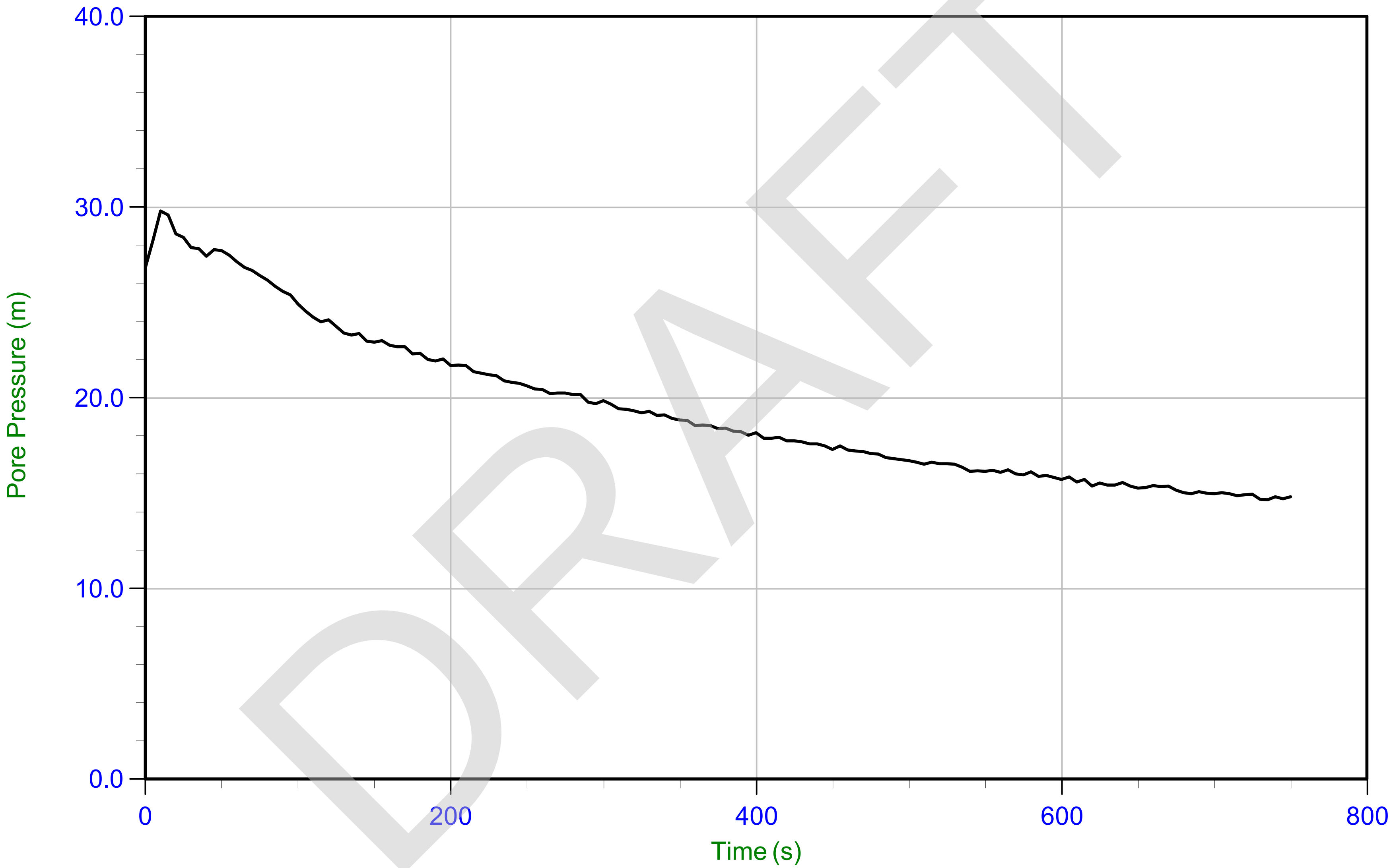
T(50): 2428.5 s
Ir: 100
Ch: 0.3 sq cm/min

ConeTec

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP01.PPF
Depth: 15.775 m / 51.755 ft
Duration: 750.0 s

U Min: 14.6 m
U Max: 29.8 m

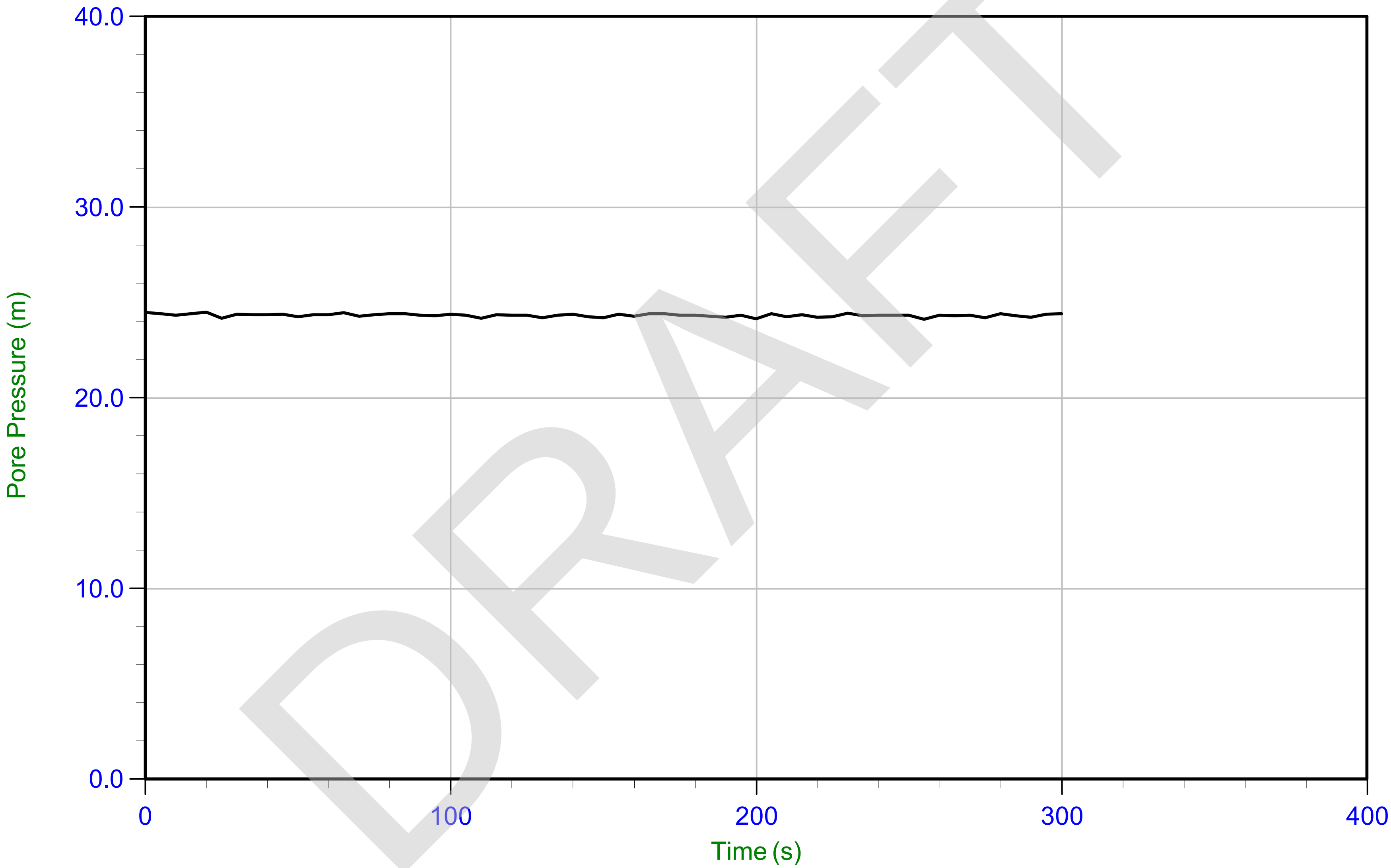
WT: 12.244 m / 40.170 ft
Ueq: 3.5 m
U(50): 16.66 m

T(50): 493.7 s
Ir: 100
Ch: 1.4 sq cm/min

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP01.PPF
Depth: 36.500 m / 119.749 ft
Duration: 300.0 s

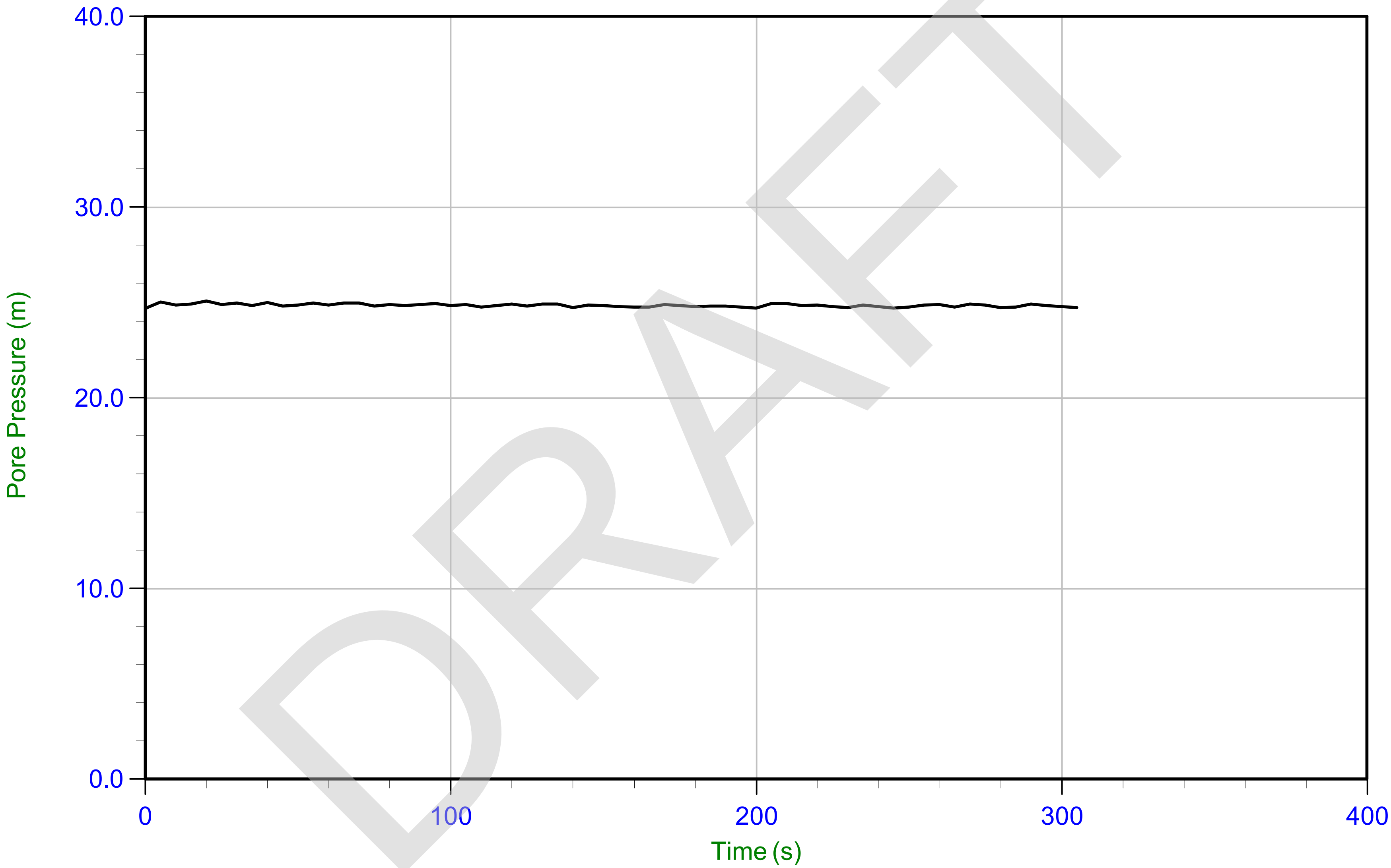
U Min: 24.1 m
U Max: 24.5 m

WT: 12.244 m / 40.170 ft
Ueq: 24.3 m

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 20:02
Site: Niagra Falls, ON

Sounding: SCPT18-01
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP01.PPF
Depth: 37.050 m / 121.554 ft
Duration: 305.0 s

U Min: 24.7 m
U Max: 25.1 m

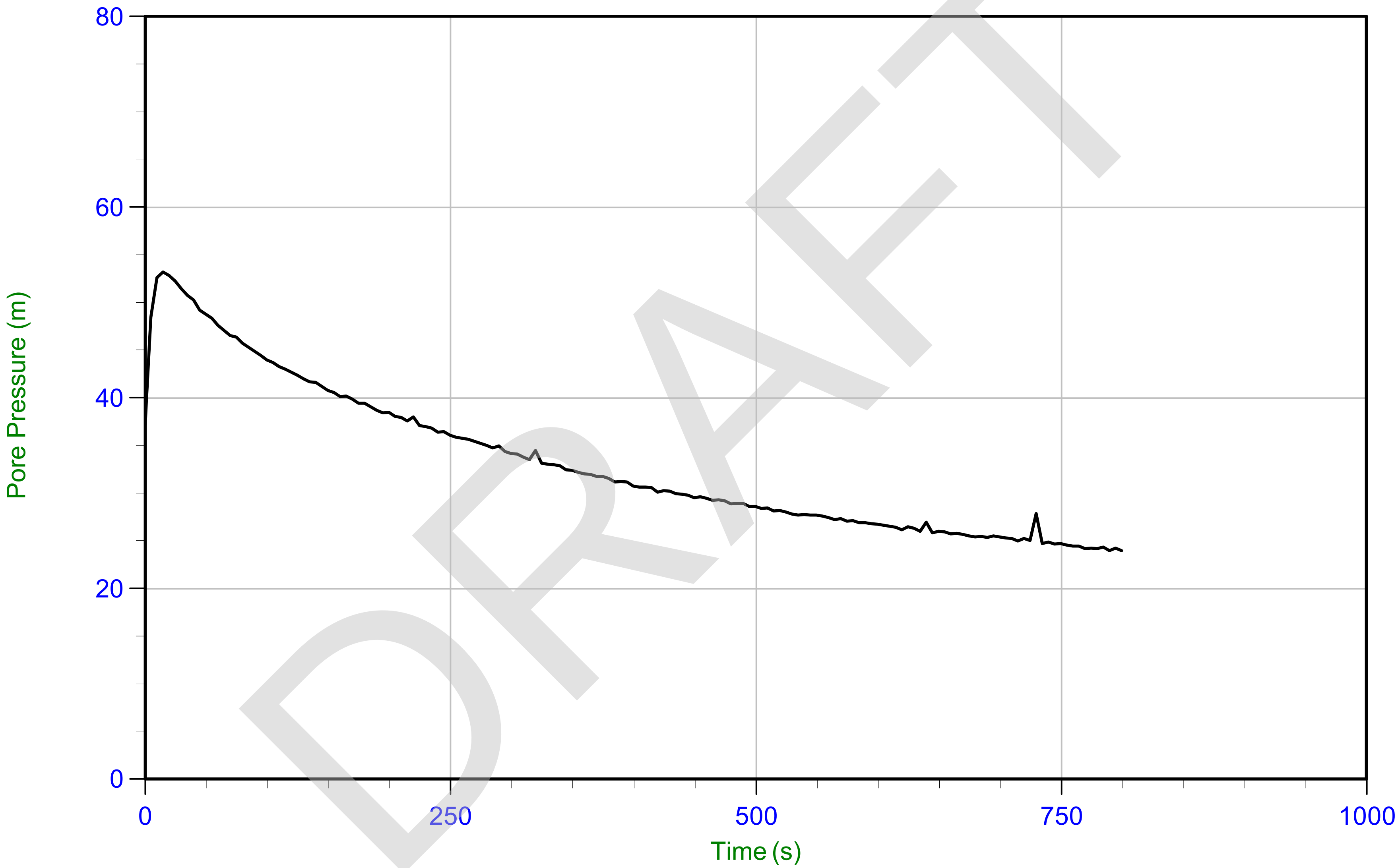
WT: 12.245 m / 40.173 ft
Ueq: 24.8 m

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP02.PPF
Depth: 10.800 m / 35.433 ft
Duration: 800.0 s

U Min: 24.0 m
U Max: 53.2 m

WT: 6.864 m / 22.519 ft
Ueq: 3.9 m
U(50): 28.57 m

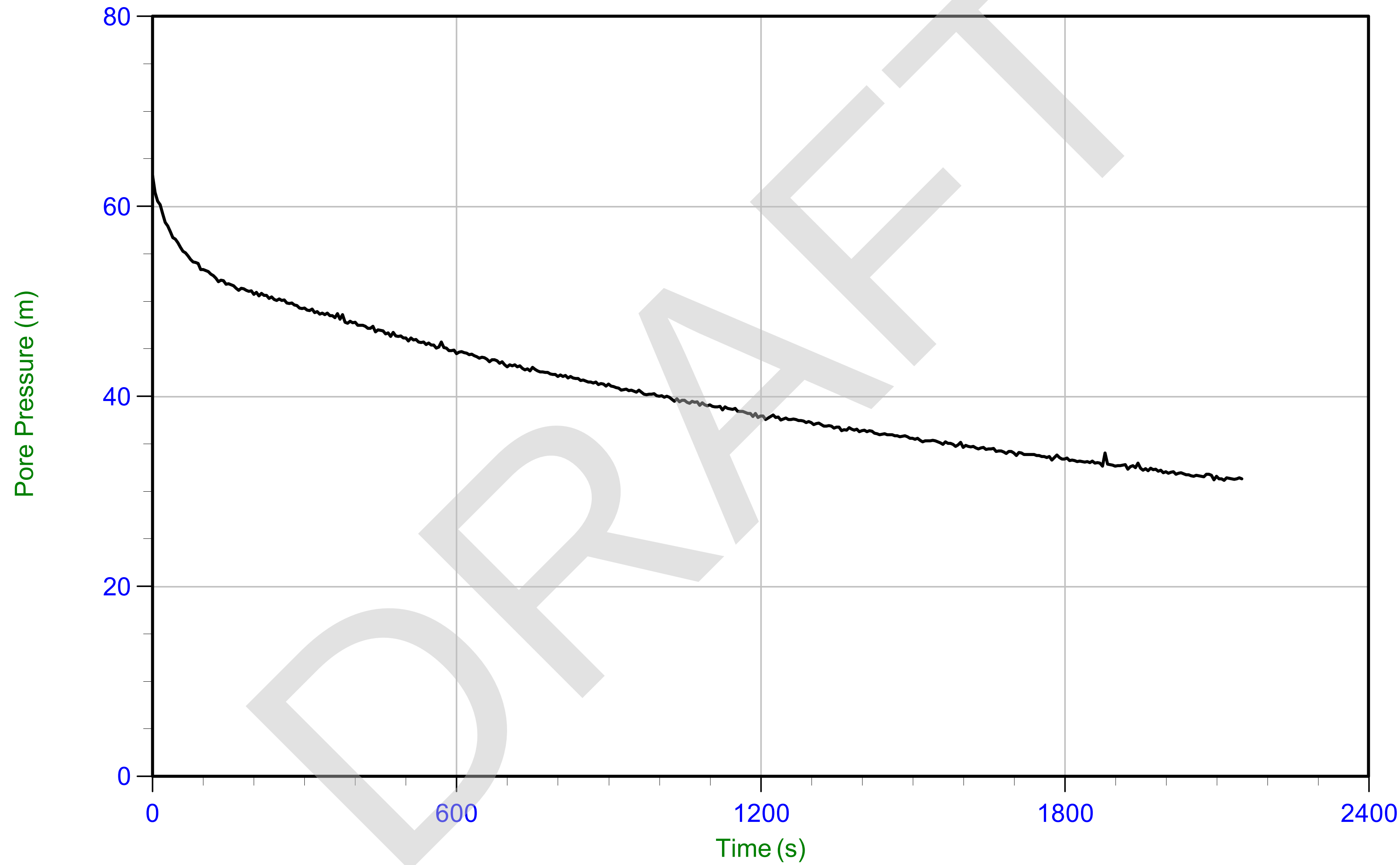
T(50): 485.9 s
Ir: 100
Ch: 1.4 sq cm/min

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

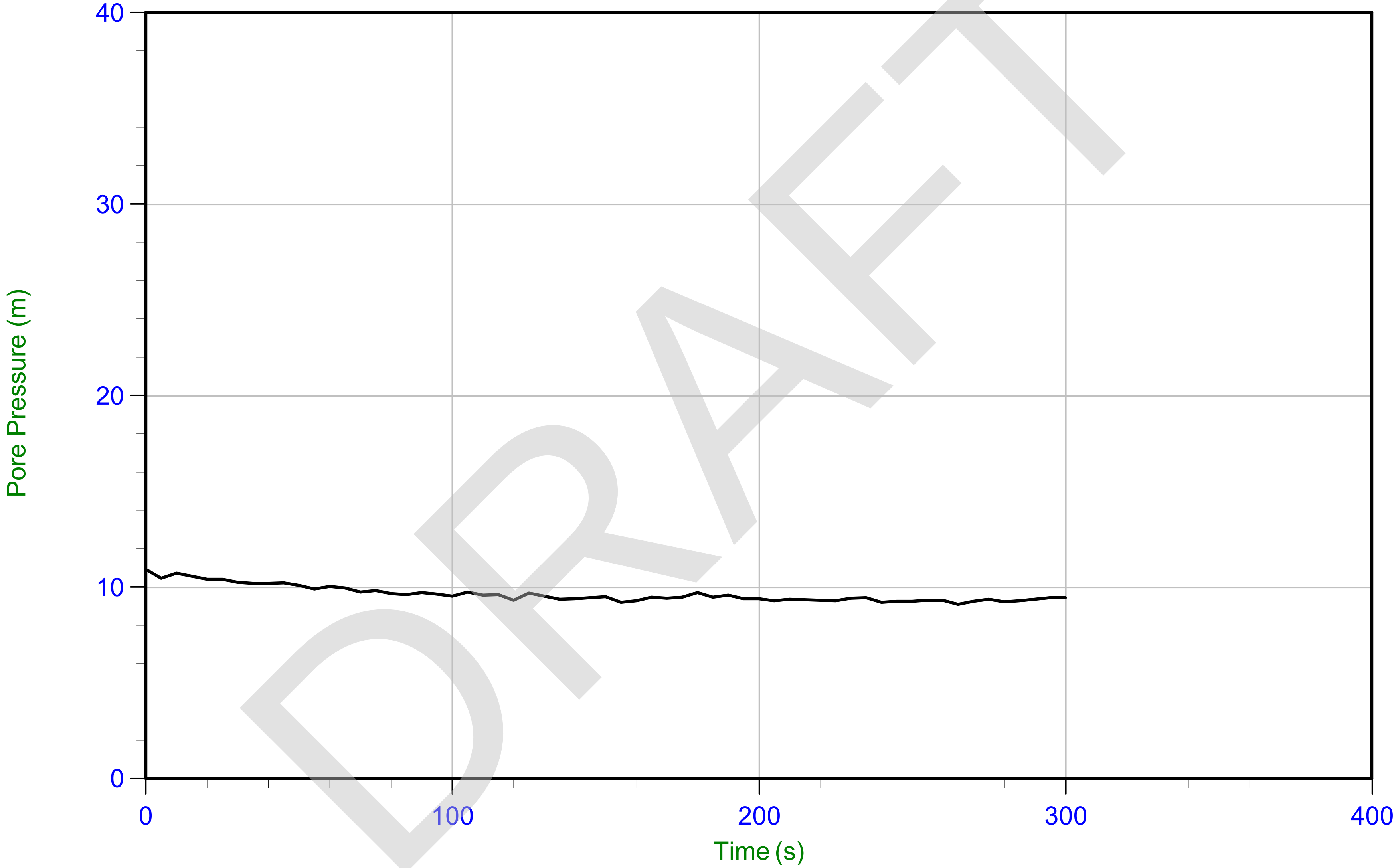
Filename: 18-05043_SP02.PPF	U Min: 31.2 m	WT: 6.864 m / 22.519 ft	T(50): 1555.0 s
Depth: 13.800 m / 45.275 ft	U Max: 63.3 m	Ueq: 6.9 m	Ir: 100
Duration: 2150.0 s		U(50): 35.11 m	Ch: 0.5 sq cm/min

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP02.PPF
Depth: 16.200 m / 53.149 ft
Duration: 300.0 s

U Min: 9.1 m
U Max: 10.9 m

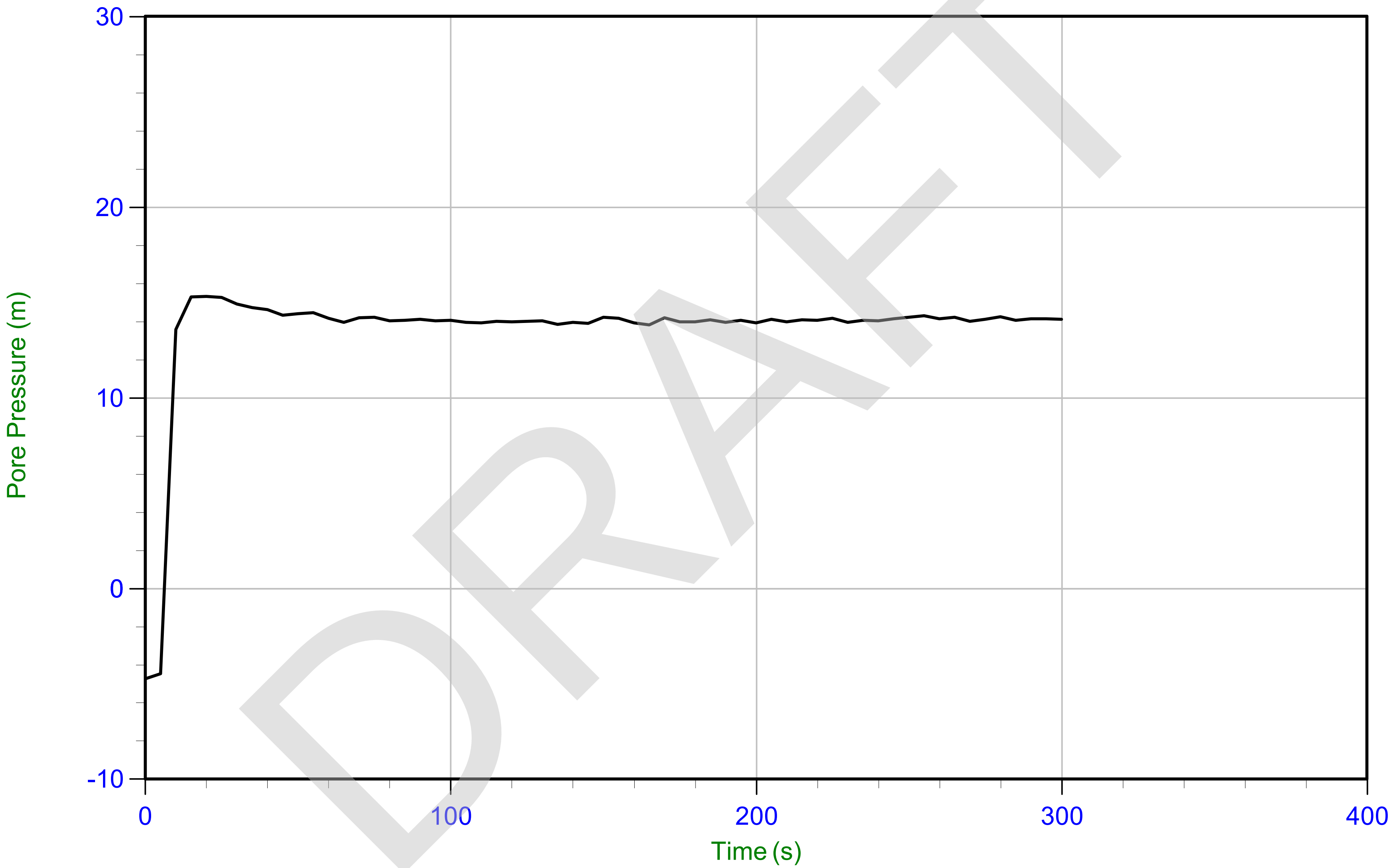
WT: 6.864 m / 22.519 ft
Ueq: 9.3 m

CONETEC

Thurber Engineering

Job No: 18-05043
Date: 07/13/2018 23:52
Site: Niagra Falls, ON

Sounding: SCPT18-02
Cone: 545:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 18-05043_SP02.PPF
Depth: 23.450 m / 76.935 ft
Duration: 300.0 s

U Min: -4.7 m
U Max: 15.3 m

WT: 9.377 m / 30.764 ft
Ueq: 14.1 m

APPENDIX C

Borehole and Drillhole Records – 2019 Investigation

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

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, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness Condition	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u 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

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90º angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

Foundation Design

GOLDER

RECORD OF BOREHOLE No 19-1

SHEET 1 OF 4

METRIC

PROJECT 18109622

W.P. 2430-15-00

LOCATION N 4767231.3; E 335626.2 NAD 83 MTM ZONE 10 (LAT. 43.044284; LONG. -79.121648)

ORIGINATED BY KN

DIST Central HWY QEW

BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with NQ Coring

COMPILED BY JM

DATUM Geodetic

DATE May 28, 2019

CHECKED BY MN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.0	GROUND SURFACE						
0.0	ASPHALT (150 mm)						
0.2	Sand and gravel (FILL) Brown Moist						
182.5 / 1.5	Silty clay, trace to some sand, trace to some organics (FILL) Firm Brown and grey Moist	1	SS	8	∇		
		2	SS	8			
		3	SS	7			
		4	SS	8		w_P–w–w_L	0 2 44 54
	- Organic inclusions and plant fragments from 7.1 m to 8.7 m depths						
		5	SS	8			
175.3 / 8.7	CLAYEY SILT to SILTY CLAY, trace to some sand Firm to very stiff Brown, grey and white to red Moist to wet						
		6	SS	15			

Elevation scale: 183, 182, 181, 180, 179, 178, 177, 176, 175

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No 19-1

SHEET 2 OF 4 — METRIC

PROJECT 18109622
W.P. 2430-15-00
LOCATION N 4767231.3; E 335626.2 NAD 83 MTM ZONE 10 (LAT. 43.044284; LONG. -79.121648)
ORIGINATED BY KN
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with NQ Coring
COMPILED BY JM
DATUM Geodetic
DATE May 28, 2019
CHECKED BY MN

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (Field Vane)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT to SILTY CLAY, trace to some sand Firm to very stiff Brown, grey and white to red Moist to wet						
		7	SS	22	173		0 1 59 40
	- Switch from augers to tricone/mud-rotary				172		
		8	SS	7			
					171	>96	
		9	SS	5	170		
						2.0	
					169		
		10	SS	WH			
					168	3.0	
		11	SS	WH	167		
					166	4.0	
		12	SS	WR			
					165	2.0	
	- Trace gravel below 19.8 m depth						

Continued Next Page

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

RECORD OF BOREHOLE No 19-1

SHEET 3 OF 4

METRIC

PROJECT 18109622
W.P. 2430-15-00
LOCATION N 4767231.3; E 335626.2 NAD 83 MTM ZONE 10 (LAT. 43.044284; LONG. -79.121648)
ORIGINATED BY KN
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with NQ Coring
COMPILED BY JM
DATUM Geodetic
DATE May 28, 2019
CHECKED BY MN

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	WATER CONTENT	GR	SA	SI	CL
	CLAYEY SILT to SILTY CLAY, trace to some sand Firm to very stiff Brown, grey and white to red Moist to wet	13	SS	WR			wP–wL with w	1	4	53	42
					163	+ 2.0					
		14	SS	WH	162						
					161	+ 2.0					
		15	SS	WH	160 / 159		o				
					158	+ 1.0					
156.1 / 27.9		16	SS	2	157						
	CLAYEY SILT with sand, some gravel (TILL) Stiff Red-brown Wet				156 / 155						
154.3 / 29.7											

Continued Next Page

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

RECORD OF BOREHOLE No 19-1

SHEET 4 OF 4 **METRIC**

PROJECT 18109622
W.P. 2430-15-00
LOCATION N 4767231.3; E 335626.2 NAD 83 MTM ZONE 10 (LAT. 43.044284; LONG. -79.121648)
ORIGINATED BY KN
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with NQ Coring
COMPILED BY JM
DATUM Geodetic
DATE May 28, 2019
CHECKED BY MN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---						
	SANDY SILT, trace clay (TILL) Very dense Red-brown Moist	17	SS	100/0.30		~20	0 21 77 2
	- Tricone grinding from ~31.1 m to 31.4 m depths - Contains gravelly layers				153 152 151		
150.5	- Spoon bouncing	18	SS	100/0.00			
33.5	Dolostone (BEDROCK) Bedrock cored from a depth of 33.5 m to 36.6 m For bedrock coring details, refer to Record of Drillhole 19-1	1	RC	REC 100%	150 149		RQD = 43%
		2	RC	REC 84%	148		RQD = 31%
147.4 36.6	END OF BOREHOLE						

NOTES:

1. Switched to mud rotary drilling at 9.1 m depth.
2. Borehole dry upon completion of hollow stem augering at 9.1 m depth.
3. Groundwater measured at a depth of 1.9 m (Elev. 182.1 m) upon completion of borehole drilling; and may be influenced by drilling mud.

Column headings also printed: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

RECORD OF DRILLHOLE: 19-1

PROJECT: 18109622

SHEET 1 OF 1

LOCATION: N 4767231.3 ;E 335626.2

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 28, 2019

DRILL RIG: CME 75 Truck Mount

DRILLING CONTRACTOR: Davis Drilling

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, MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

Columns: DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. / DEPTH (m) | RUN No. | RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.25 m (5 10 15 20) | DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION, Jr, Ja | ROCK STRENGTH INDEX (R4 R3 R2 R1) | WEATHERING INDEX (W1 W2 W3 W4 W5 W6) | FEATURES | R0/R1 ZONES

DESCRIPTION	ELEV. DEPTH (m)	RUN No.	TYPE AND SURFACE DESCRIPTION	Notes
Refer to Record of Borehole 19-1	150.49			
Dolostone (BEDROCK), with black shale partings Slightly weathered Thinly bedded Grey Fine to medium grained Faintly porous with occasional vuggs Strong to very strong - Moderate vuggy porosity between 33.7 m and 34.2 m depths - Lost core presumed due to core run 2 breaking off bottom along bedding at 36.3 m depth.	33.53	1 2	BD,PL,SM BD,PL,SM CO,UN,SM BD,UN,SM BD,UN,SM BD,CU,SM CO,UN,SM BD,UN,SM BD,PL,SM BD,UN,RO BD,UN,SM BD,UN,SM BD,PL,RO BD,UN,SM BD,PL,RO BD,PL,SM BD,UN,SM JN,PL,SM BD,UN,SM CO,PL,RO CO,PL,RO BD,PL,RO BD,PL,RO BD,UN,SM BD,UN,RO BD,CU,SM BD,UN,RO BD,PL,SM BD,PL,RO BD,PL,RO BD,UN,RO BD,UN,SM	UCS = 60.6 MPa UCS = 115.1 MPa UCS = 81.0 MPa UCS = 130.4 MPa
	147.44 36.58			
END OF DRILLHOLE				

Drilling record: NQ Coring

Depth scale: 34, 35, 36, 37, 38, 39, 40, 41, 42

FEATURES LEGEND

BROKEN CORE CLAY SEAM LIMESTONE LOST CORE

DEPTH SCALE

1 : 50

GOLDER

LOGGED: KN

CHECKED:

GOLDER

RECORD OF BOREHOLE No 19-2

SHEET 1 OF 3 **METRIC**

PROJECT 18109622
W.P. 2430-15-00
LOCATION N 4767546.6; E 335628.3 NAD 83 MTM ZONE 10 (LAT. 43.047122; LONG. -79.121605)
ORIGINATED BY KN
DIST Central HWY QEW
BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with Wash Boring
COMPILED BY JM
DATUM Geodetic
DATE May 27, 2019
CHECKED BY MN

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE / SHEAR STRENGTH	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
179.9	GROUND SURFACE												
0.0 / 179.7	ASPHALT (250 mm)												
0.3	Sand and gravel (FILL) Brown Moist												
179.0 / 0.9	Silty clay, trace to some sand, trace gravel, trace organics (FILL) Stiff to firm Reddish-brown Moist					179							
		1	SS	9		178							
		2	SS	9		177 / 176							
		3	SS	8		175		plotted		2	5	49	44
	- Layer of organics at 6.5 m depth	4	SS	7		174 / 173							
		5	SS	7		172		plotted					
	- Switch from augers to tricone/mud-rotary					171	+ >96						
	- Some organics between 9.1 m and 9.8 m depths	6	SS	11	∇	170							
170.1 / 9.8													

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTO\QEW\WELLANDRIVERBRIDGES\02_DATA\GINT\18109622.GPJ GAL-GTA.GDT 6/26/19 JM

GOLDER

PROJECT 18109622	**RECORD OF BOREHOLE No 19-2**	SHEET 2 OF 3	**METRIC**
W.P. 2430-15-00	LOCATION N 4767546.6; E 335628.3 NAD 83 MTM ZONE 10 (LAT. 43.047122; LONG. -79.121605)		ORIGINATED BY KN
DIST Central HWY QEW	BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with Wash Boring		COMPILED BY JM
DATUM Geodetic	DATE May 27, 2019		CHECKED BY MN

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa) / sensitivity	REMARKS GR SA SI CL
	CLAYEY SILT to SILTY CLAY, trace sand, trace gravel Firm to stiff Red-brown to grey Moist to wet					
		7	SS	2		
					+ 3.0	
		8	SS	3		0 0 67 33
					+ 3.0	
	- Silt seams at 13.7 m depth	9	SS	WH		
					+ 2.0	
164.3		10	SS	3		0 1 86 13
15.6	SILT, some clay, trace sand Very loose Red-brown Wet					
163.6 16.3	SILTY CLAY Stiff Brown Wet				+ 2.0	
		11	SS	4		
					+ 2.0	
161.6 18.3	SILTY SAND, some gravel (TILL) Very loose to compact Red-brown to brown Wet	12	SS	2		
					+ >96	

Elevation scale: 169, 168, 167, 166, 165, 164, 163, 162, 161, 160

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

GOLDER

PROJECT 18109622

RECORD OF BOREHOLE No 19-2

METRIC

W.P. 2430-15-00

LOCATION N 4767546.6; E 335628.3 NAD 83 MTM ZONE 10 (LAT. 43.047122; LONG. -79.121605)

ORIGINATED BY KN

DIST Central HWY QEW

BOREHOLE TYPE CME 75, 190mm O.D. Hollow Stem Augers, 'P' Casing with Wash Boring

COMPILED BY JM

DATUM Geodetic

DATE May 27, 2019

CHECKED BY MN

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND, some gravel (TILL) Very loose to compact Red-brown to brown Wet	13	SS	24		
					159	
		14	SS	29		
					158	
					157	
156.6 23.3	SAND, some silt to SILT and SAND Dense Red-brown Wet					
					156	
		15	SS	47		0 79 20 1
					155	
					154	
					153	
		16	SS	33		0 31 66 3
					152	
	- Spoon bouncing					
151.5 28.4	END OF BOREHOLE TRICONE and SAMPLER REFUSAL	~~17~~	~~SS~~	~~100/0.02~~		

NOTES:

1. Switched to mud rotary at 9.1 m depth.

2. Groundwater measured at a depth of 9.8 m (Elev. 170.1) upon completion of hollow stem augering.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

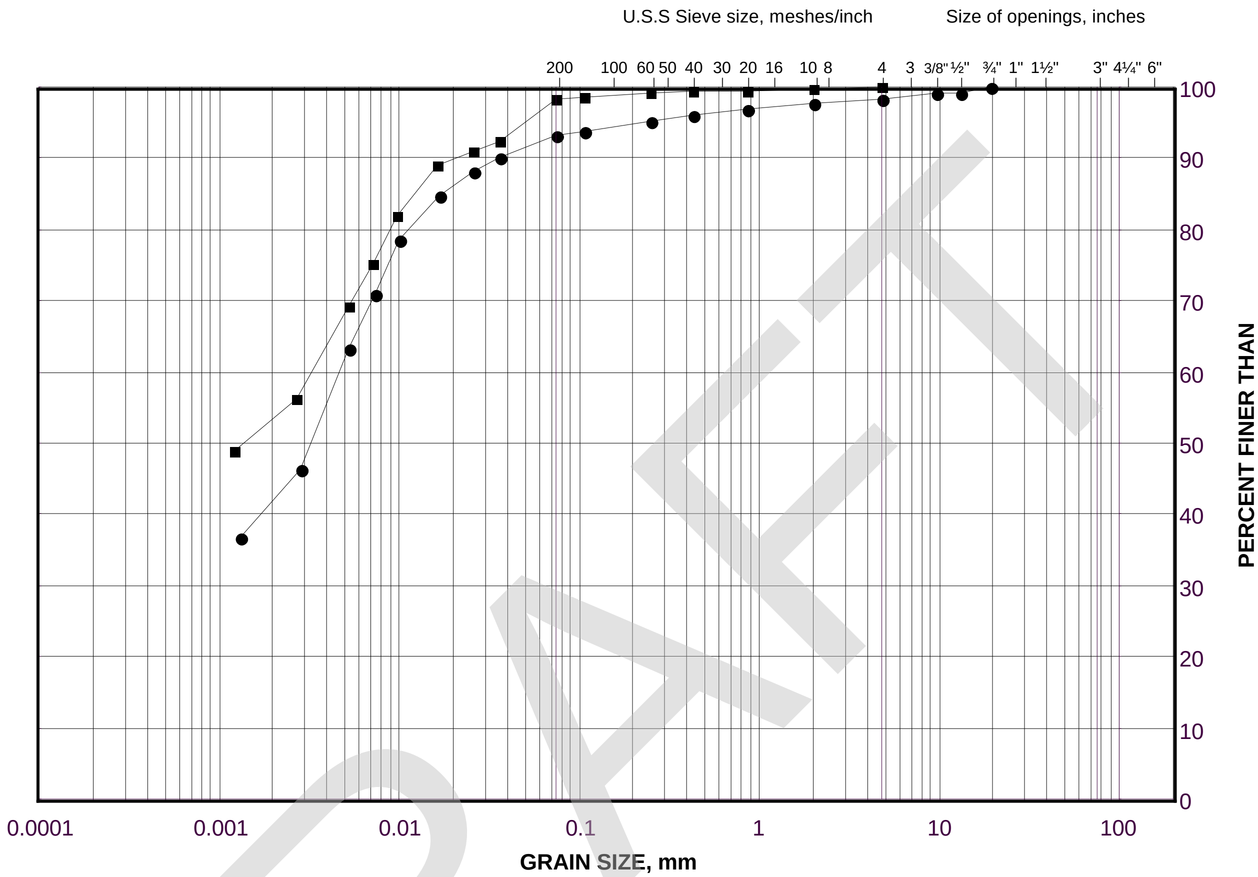
APPENDIX D

Geotechnical Laboratory Test Results and Bedrock Core Photographs

GRAIN SIZE DISTRIBUTION

Silty Clay (Fill)

FIGURE D1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-2	3	175.0
■	19-1	4	177.6

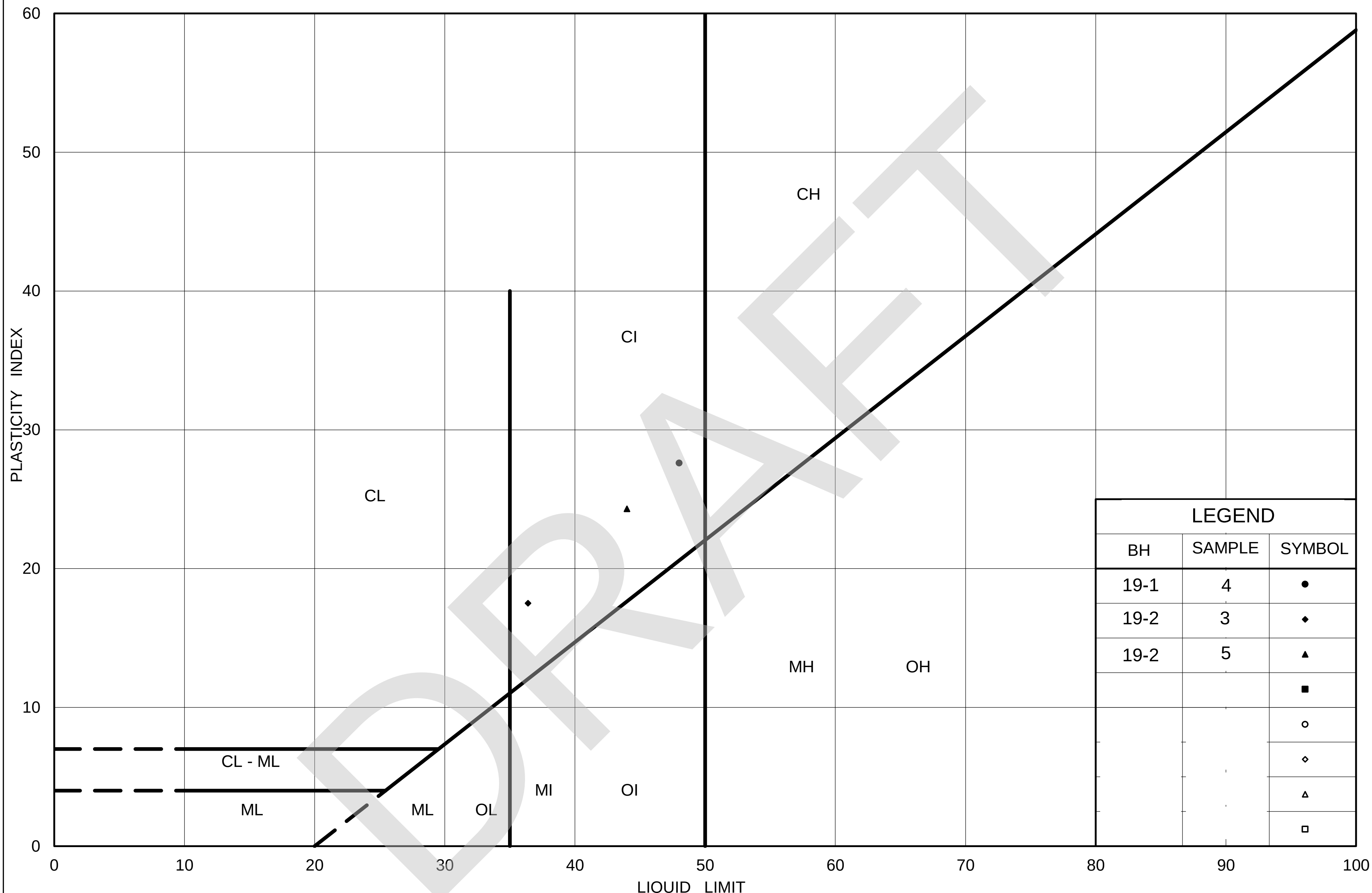
Project Number: 18109622

Checked By: SEMP

Golder Associates

Date: 13-Jun-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay (Fill)

Figure No. D2

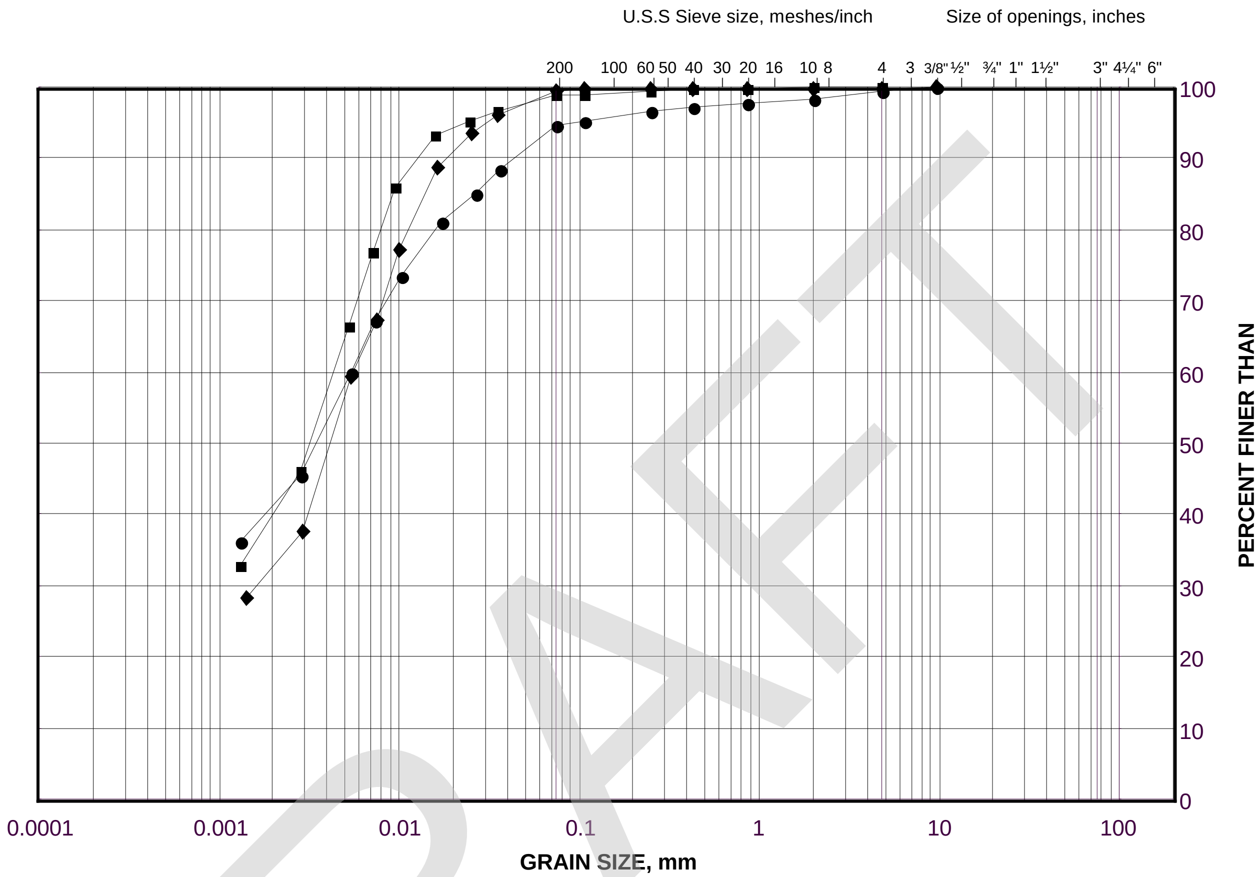
Project No. 18109622

Checked By: SEMP

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE D3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-1	13	163.9
■	19-1	7	173.0
◆	19-2	8	167.4

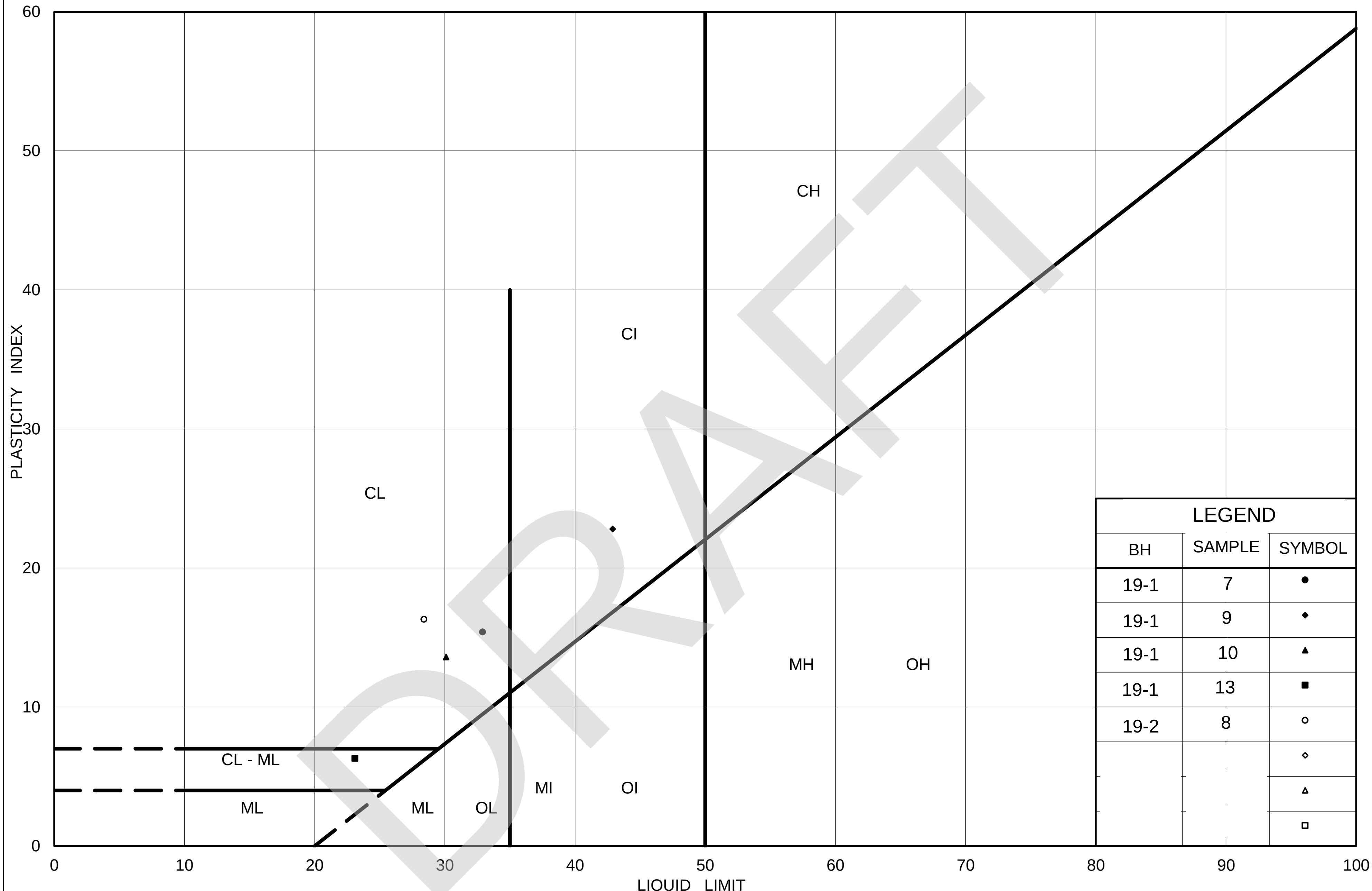
Project Number: 18109622

Checked By: SEMP

Golder Associates

Date: 13-Jun-19

Oct 75, FF-S-21

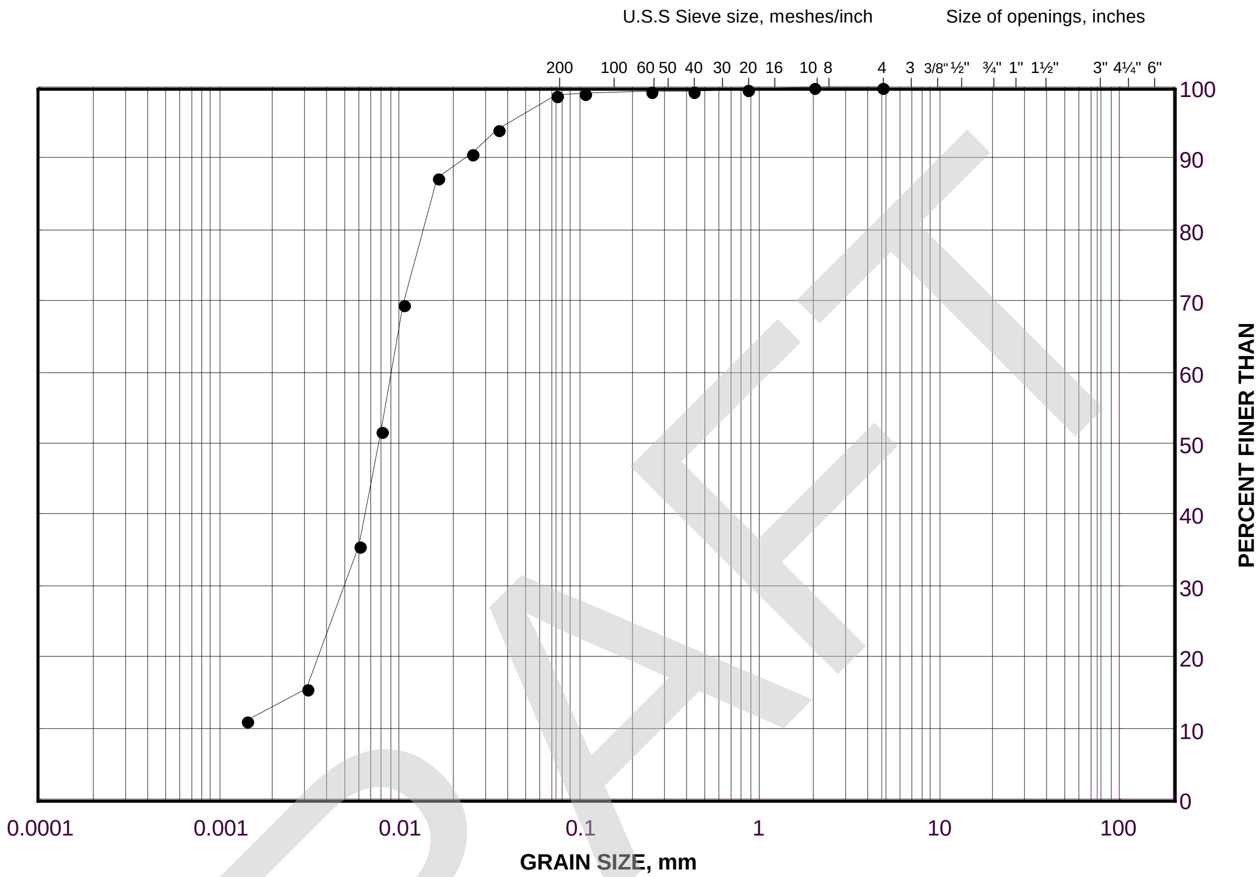


Ministry of Transportation Ontario	PLASTICITY CHART Clayey Silt to Silty Clay	Figure No. D4 Project No. 18109622 Checked By: SEMP

GRAIN SIZE DISTRIBUTION

Silt, Interlayer

FIGURE D5



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-2	10B	172.0

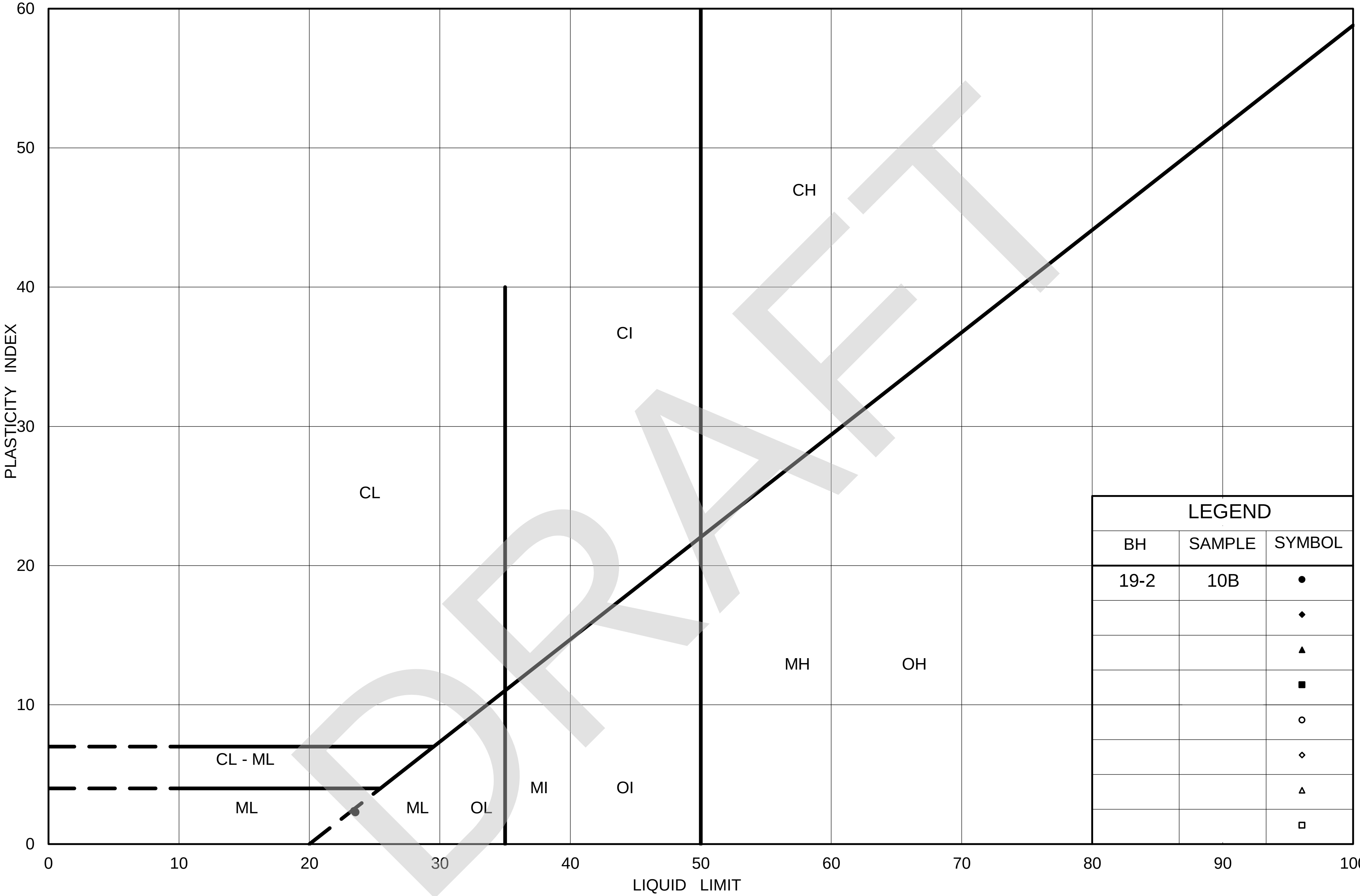
Project Number: 18109622

Checked By: SEMP

Golder Associates

Date: 13-Jun-19

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silt, Interlayer

Figure No. D6

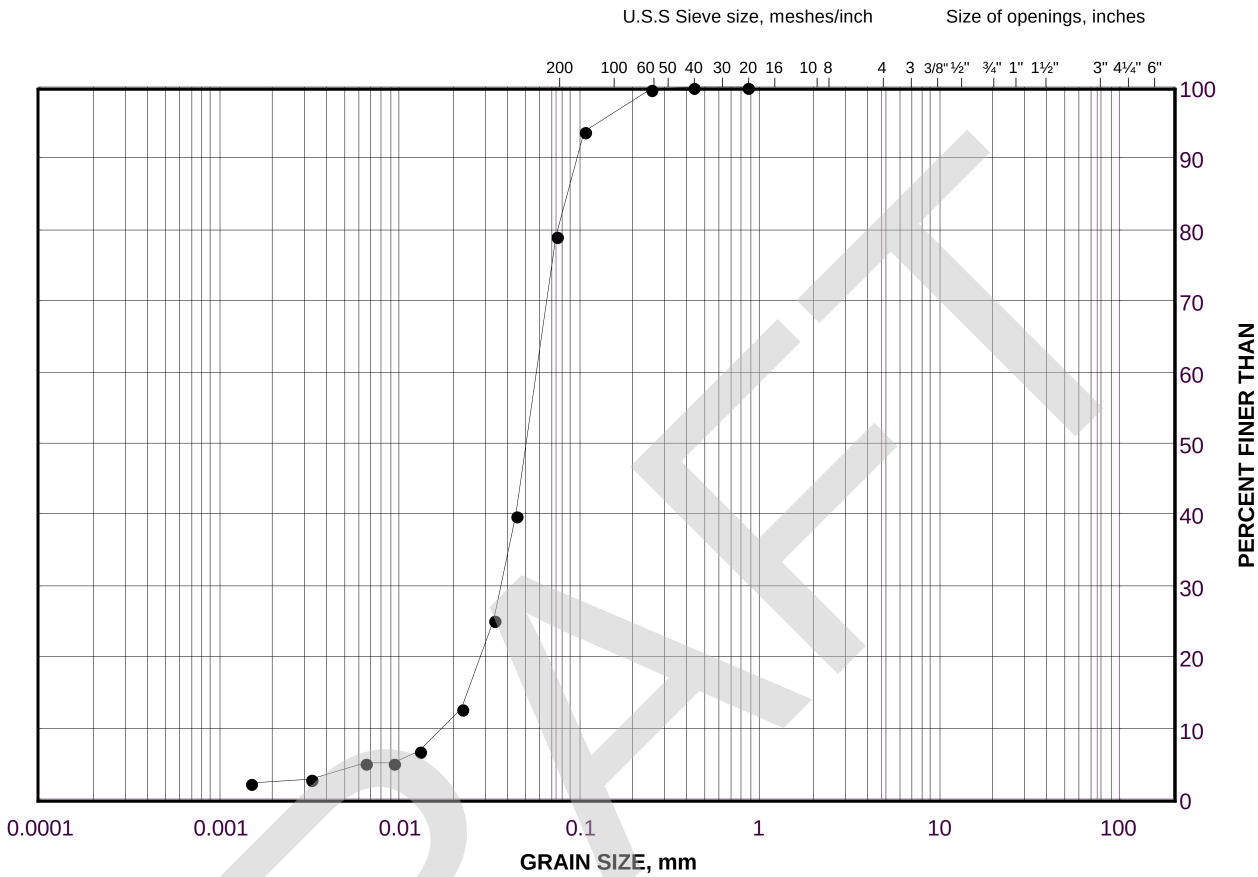
Project No. 18109622

Checked By: SEMP

GRAIN SIZE DISTRIBUTION

Sandy Silt (Till)

FIGURE D7



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-1	17	153.4

Project Number: 18109622

Checked By: SEMP

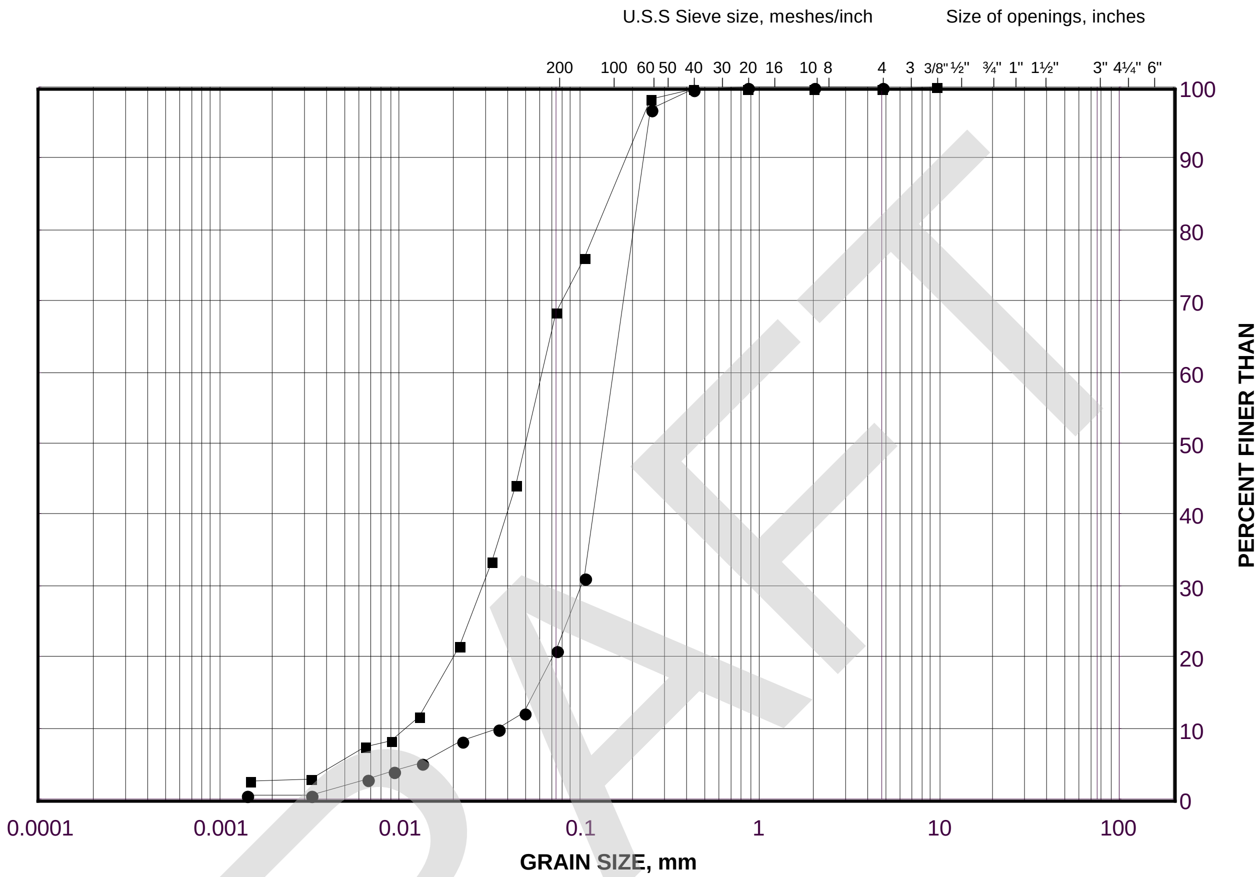
Golder Associates

Date: 13-Jun-19

GRAIN SIZE DISTRIBUTION

Sand to Silt and Sand

FIGURE D8



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	19-2	15	155.2
■	19-2	16	152.1

Project Number: 18109622

Checked By: SEMP

Golder Associates

Date: 13-Jun-19

Borehole 19-1

Start of Run No.1 (33.53 m)

Box 1: 33.53 m – 35.05 m

Start of Run No.2 (35.05 m)

Box 2: 35.05 m – 36.58 m

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m

0 ft	1 ft	2 ft	3 ft	4 ft	5 ft

Scale

PROJECT

Welland River Twin Bridge Replacement

TITLE

Bedrock Core Photographs
Borehole 19-1 (33.53 m – 36.58 m)

GOLDER

PROJECT No. 18109622			FILE No. BH-19-1		
DRAFT	CM	JUN 2019	SCALE	AS SHOWN	VER. 1.
CADD	--		**FIGURE D9**		
CHECK	MN				
REVIEW	SEMP				

GOLDER

FIGURE D10.1

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	18109622	SAMPLE NUMBER	19-1-1
PROJECT NAME	MTO/DB2018-2013/Welland RiverBridge	SAMPLE DEPTH, m	34.48-34.61
BOREHOLE NUMBER	19-1	DATE:	June 10, 2019

TEST CONDITIONS

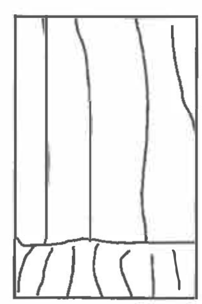
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.11

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.98	WATER CONTENT, (specimen) %	0.20
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	27.32
SAMPLE AREA, cm^2	17.62	DRY UNIT WT., kN/m^3	27.26
SAMPLE VOLUME, cm^3	175.74	SPECIFIC GRAVITY	-
WET WEIGHT, g	489.73	VOID RATIO	-
DRY WEIGHT, g	488.75		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	60.6

REMARKS:

Checked By: [signature]

Golder Associates

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

FIGURE D10.2

BEFORE COMPRESSION

AFTER COMPRESSION

Date June 10, 2019
Project 18109622

Golder Associates

Drawn Frank
Chkd.

GOLDER

FIGURE D11.1

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	18109622	SAMPLE NUMBER	19-1-2
PROJECT NAME	MTO/DB2018-2013/Welland RiverBridge	SAMPLE DEPTH, m	34.89-35.02
BOREHOLE NUMBER	19-1	DATE:	June 10, 2019

TEST CONDITIONS

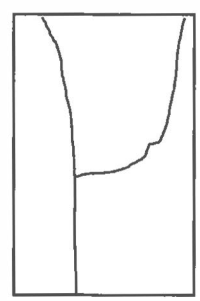
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.47	WATER CONTENT, (specimen) %	0.50
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	26.92
SAMPLE AREA, cm^2	17.59	DRY UNIT WT., kN/m^3	26.79
SAMPLE VOLUME, cm^3	184.13	SPECIFIC GRAVITY	-
WET WEIGHT, g	505.73	VOID RATIO	-
DRY WEIGHT, g	503.21		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	115.1

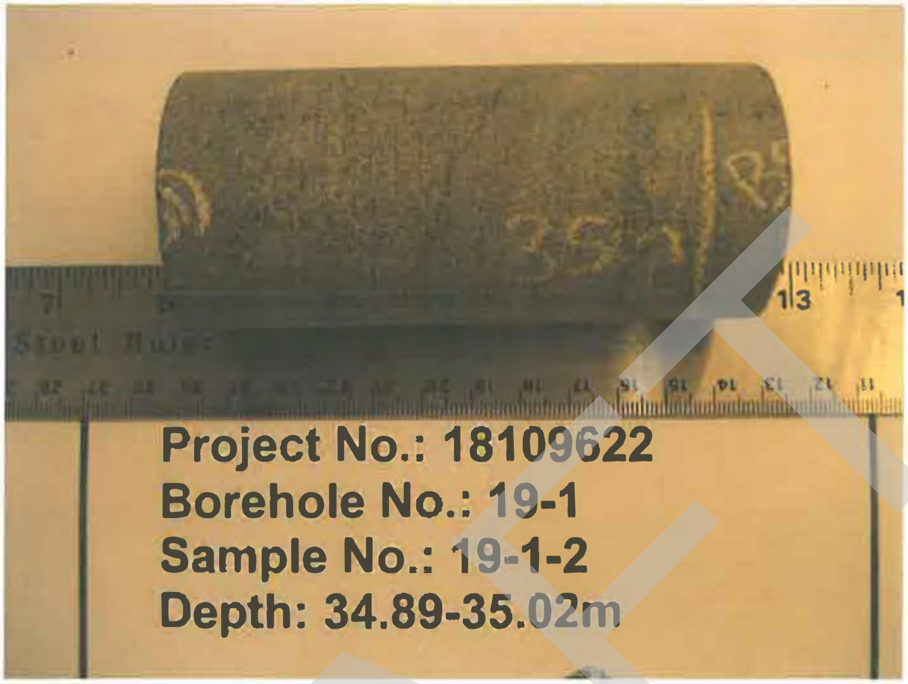
REMARKS:

Checked By:

Golder Associates

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

FIGURE D11.2



BEFORE COMPRESSION

AFTER COMPRESSION

Date June 10, 2019
Project 18109622

Golder Associates

Drawn Frank
Chkd.

FIGURE D12.1

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	18109622	SAMPLE NUMBER	19-1-3
PROJECT NAME	MTO/DB2018-2013/Welland RiverBridge	SAMPLE DEPTH, m	35.10-35.21
BOREHOLE NUMBER	19-1	DATE:	June 10, 2019

TEST CONDITIONS

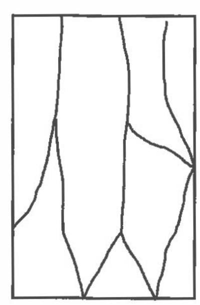
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	1.97

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.30	WATER CONTENT, (specimen) %	0.40
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	26.70
SAMPLE AREA, cm^2	17.56	DRY UNIT WT., kN/m^3	26.59
SAMPLE VOLUME, cm^3	163.31	SPECIFIC GRAVITY	-
WET WEIGHT, g	444.75	VOID RATIO	-
DRY WEIGHT, g	442.98		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	81.0

REMARKS:

L/D Ratio not in accordance with ASTM Standard

Checked By: *(signature)*

Golder Associates

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

FIGURE D12.2

BEFORE COMPRESSION

AFTER COMPRESSION

Date June 10, 2019
Project 18109622

Golder Associates

Drawn Frank
Chkd. LM

GOLDER

FIGURE D13.1

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	18109622	SAMPLE NUMBER	19-1-4
PROJECT NAME	MTO/DB2018-2013/Welland RiverBridge	SAMPLE DEPTH, m	35.84-35.95
BOREHOLE NUMBER	19-1	DATE:	June 10, 2019

TEST CONDITIONS

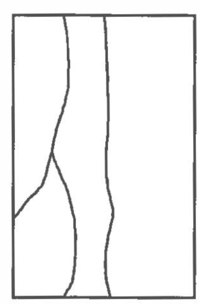
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.19

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.35	WATER CONTENT, (specimen) %	0.40
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	27.04
SAMPLE AREA, cm^2	17.57	DRY UNIT WT., kN/m^3	26.93
SAMPLE VOLUME, cm^3	181.94	SPECIFIC GRAVITY	-
WET WEIGHT, g	501.88	VOID RATIO	-
DRY WEIGHT, g	499.88		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	130.4

REMARKS:

Checked By:

Golder Associates

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

FIGURE D13.2

BEFORE COMPRESSION

AFTER COMPRESSION

Date June 10, 2019
Project 18109622

Golder Associates

Drawn Frank
Chkd. JM

APPENDIX E

Non-Standard Special Provisions (NSSP)

Item 4 – Chapter 3 – MTO Special Provisions, (Page 340), adds the following page:

Cellular Concrete - Item No.

Non Standard Special Provision

1.0 SCOPE

This specification specifies the requirements for the supply and placement of cellular concrete for use as lightweight fill at the locations and in accordance with the details shown in the plans. The cellular concrete shall be placed in the dry condition and above any groundwater table.

2.0 REFERENCES

This specification refers to the following standards, specifications, or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 517 Dewatering
OPSS 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

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 495-99a 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

3.0 DEFINITIONS

For the purpose of this specification 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.

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

Project Superintendent: means the cellular concrete's authorized representative in responsible charge of the construction of the cellular concrete

Cellular Concrete Representative: means an individual with continuous full-time employment with the cellular concrete supplier for a period of at least three (3) years, and who is knowledgeable in the design and construction of the cellular concrete.

4.0 DESIGN AND SUBMISSION 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

4.2.1 Project Superintendent

At least two weeks prior to commencement of construction of the cellular concrete, the name(s) of the project superintendent responsible for the placement of the cellular concrete in the Contract shall be submitted in writing to the Contract Administrator.

During construction of the cellular concrete, the project superintendent shall not change without written permission from the Contract Administrator. A proposal for a change in the project superintendent shall be submitted at least one week prior to the actual change in project superintendent.

4.2.2 Cellular Concrete Representative

At least two weeks prior to commencement of construction of the cellular concrete, the name(s) of the cellular concrete representative shall be submitted in writing to the Contract Administrator.

At least 48 hours written advance notice shall be provided to the Contract Administrator prior to each visit to the site by the cellular concrete representative. The advance notice shall include the dates and locations the cellular concrete representative will be on site.

4.3 Submission of Shop Drawings and Placement Procedures

The shop drawings and the proposed placement procedures shall be submitted to the Contract Administrator for review at least fifteen (15) business days prior to commencement of the work.

The submission shall include a description of the proposed method of installation including, as a minimum, the following:

- A work plan outlining the schedule, procedures and work site details;

- Proposed dewatering procedure (in accordance with OPSS 517);
- Environmental Protection procedures;
- Method for sealing cracks (if any) to prevent grout leakage;
- Method for bulkhead construction;
- List of equipment to be used;
- List of materials to be used;

The contractor must clearly identify how the grouting procedure will be monitored.

4.4 Submission of Environmental Protection Strategy

At least fifteen (15) business days 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.

5.5 Cellular Concrete Properties

Cellular concrete shall have the following properties:

- Minimum unconfined compressive strength at 28 days of 0.5 MPa.
- Wet cast density of 475 kg/m³ (4.66 kN/m³) (+/-5%)
- Must not contain any fly ash or any other waste or process by-product.

5.6 Prequalification of Cellular Concrete Product

Prior to use, the Cellular concrete product must be “Prequalified” for use as lightweight fill by MTO (through the MTO Lightweight Fill Committee Prequalification Process).

6.0 EQUIPMENT

Cellular concrete shall be produced utilizing specialized automated proportioning, mixing, and foam producing equipment, which is capable of meeting the specified properties.

Dry-mix equipment must be able to receive bulk cement and process it continuously from one piece of equipment, and pump through hoses or pipes up to a flat lineal distance of 1000 metres. 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. 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, consistent and predictable volumetric rate of foam with stable uniform microbubbles.

7.0 CONSTRUCTION

7.1 Excavation and Subgrade Preparation

Foundation excavation shall be carried out to the design elevations and the horizontal and vertical limits shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be sub excavated.

The prepared subgrade shall be good competent level ground and 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.

7.3 Roadway Protection System

The construction of all protection schemes shall be according to OPSS 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 Placement

The construction of cellular concrete shall be scheduled such that it is at all times under the responsible charge of the project superintendent who has been advised on site by the manufacturer's representative as to the required procedures and schedule for the placement of the cellular concrete.

The cellular concrete representative shall be on site to oversee the placement of the cellular concrete and to verify that the cellular concrete is being supplied and placement in accordance with the contractual requirements.

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.

Where required, formwork should be designed and installed to withhold cellular concrete. When working near surface water, formworks shall be lined with an impermeable liner to prevent any leakage.

Cellular concrete shall not be allowed on frozen ground. 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. Cold weather protection shall be provided in accordance with OPSS.PROV 1350.

Cellular concrete must not be placed during heavy or prolonged precipitation.

Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling.

The Constructor shall determine the maximum lift thickness based on density and any other considerations that may impact placement. Cellular concrete shall be cast in a formed area within 1 to 2 hours, to permit an undisturbed setting.

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

Vehicles, equipment, backfills or other loadings on the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfill can commence on the cellular concrete when the cellular concrete has attained sufficient strength such that foot traffic can be supported 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 ASSURANCE

8.1 Field Sampling and Testing

The fresh cellular concrete shall be collected for density testing once per production run, or once for every 50 cubic metres, or once per 30 minutes, whichever is more frequent.

Cellular concrete samples shall be captured, cured, and tested 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 is comprised of one set of four cellular concrete cylinders. One sample should be taken for each placement, or every 100m3, whichever is more frequent. Cylinders are cast in 75 mm by 150 mm cylindrical plastic molds. Cellular concrete cylinders shall be cured and tested for compressive strength as per ASTM C495-07 and LS 407.

9.0 MEASUREMENT FOR PAYMENT – NOT USED

10.0 BASIS OF PAYMENT – NOT USED

Ed Marcon
Team Lead, Contract Tendering
February 15, 2019

golder.com