LOWER BREWERS SWING BRIDGE REPLACEMENT

GEOTECHNICAL INVESTIGATION

PARKS CANADA

PROJECT NO.: 19M-01599-00
DATE: NOVEMBER 23, 2021

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November 23, 2021

Parks Canada
Ontario Waterways Unit
34 Beckwith Street
Smiths Falls, ON K7A 3A8

Attention: Mr. Tyler Atkinson

Subject: Geotechnical Investigation Report, Lower Brewers Swing Bridge Replacement

Dear Sir:

We are pleased to submit our geotechnical investigation report addressing subsurface conditions to support the design and construction of the currently proposed full replacement of the Lower Brewers Swing Bridge located on Washburn Road at Lock 45 of the Rideau Canal near Washburn, north of Kingston, Ontario.

It is WSP's understanding that the proposed construction shall include the replacement of the Lower Brewers Swing Bridge, refurbishing and/or replacing the mechanical components, and replacing/rehabilitating the existing substructure.

A geotechnical soils investigation within the current project limits was completed by WSP between November 18th and 20th, 2019. The investigation comprised of exploring the subsurface conditions by means of advancing and sampling a total of four (4) boreholes and advancing one (1) dynamically-coned probe. A track mounted drill rig was used and drilling was completed using direct push technology with standard penetration testing (SPT) and/or dynamic cone penetration testing (DCPT). A groundwater monitoring well previously installed on this site by others was used to obtain a groundwater depth measurement during our fieldwork.

In addition to the geotechnical information obtained from boreholes drilled by WSP as part of this assignment, previous geotechnical reports by others were reviewed by WSP and the relevant portions thereof are included (as Appendices) and discussed in this report within the context of the current project.

This report summarizes the procedures and findings of WSP's geotechnical investigation for this project, including results of the drilling and laboratory testing program, and our general recommendations with regards to design and construction of the bridge replacement.

We trust that the information in this report is straightforward and meets with your present requirements.

Yours sincerely,

Pete Hynes, P.Eng.
Project Engineer

WSP ref.: 19M-01599-00

REVISION HISTORY

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March 3, 2020	DRAFT	
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November 23, 2021	FINAL	
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Date

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1 INTRODUCTION

WSP Canada Group Ltd. (WSP) was retained by Parks Canada (the Client) to undertake a geotechnical site investigation to support the design and reconstruction of the Lower Brewers Swing Bridge located on Washburn Road at Lock 45 of the Rideau Canal near the Community of Washburn, to the north of Kingston (city centre), Ontario (the "Site"). The Site is located at the boundary of the City of Kingston and the Township of South Frontenac; the Rideau Canal is the boundary of these municipalities, with the east bank being located in the City of Kingston and the west bank being located in the Township of Frontenac. Site location mapping showing the site and borehole locations is included as **Figures 1 and 2** of this report.

It is WSP's understanding that the Lower Brewers Swing Bridge, including its foundations, will be replaced as part of this project. This will consist of constructing new foundations for abutments on the west and east banks, new foundations for a pivot pier immediately west of the canal's west edge, and reconstruction of the approaches (which may include up to a 0.3 metres grade raise).

This geotechnical report provides information on subsurface conditions at the Site, including a description of the existing soil profile and groundwater conditions. Included as appendices are selected portions of reports summarizing subsurface conditions encountered during previous geotechnical works performed by other firms – such information was obtained from the following documents:

- report prepared by SNC Lavalin GEM Ontario Inc. (SNC), entitled "Geotechnical Investigation, Lower Brewers and Brass Point Swing Bridges, Rideau Canal, North of the City of Kingston, Ontario", dated November 15, 2018; and
- report prepared by Butts, Ross, Magwood & Hall Ltd. (BRMH), entitled "Report of a Site Investigation, Lower Brewers Lock Station", dated June 28, 1968.

Based on our investigation findings, WSP herein provides geotechnical recommendations for consideration in the design and reconstruction of the proposed new swing bridge structure.

Details are provided in the following sections of this report.

2 INVESTIGATION METHODOLOGY

2.1 FIELD INVESTIGATION

WSP completed a borehole investigation for the site between November 18th and 20th, 2019. Buried utility clearances were completed prior to equipment mobilization. A total of five (5) boreholes (including a probe advanced using dynamic cone penetration testing) were advanced, designated as BH19-01 to BH19-05. The boreholes extended to depths ranging from approximately 1.5 metres (m) to 17.7 m below existing grade (mbeg) and were located as illustrated on **Figures 1 and 2**. A monitoring well previously installed by SNC in their borehole BH2 (see **Appendix D**) was used to obtain a groundwater level measurement during our investigation fieldwork.

WSP field personnel supervised the drilling operations and recorded the subsurface conditions encountered in our boreholes. The boreholes were advanced using a commercial track-mount drill rig. Four of the boreholes (excluding BH19-04) were advanced by the rig using direct push technology and continuous sampling method, with soil samples recovered at regular intervals (approximately 0.75 m and 1.5 m) using a 51 millimetre (mm) outside diameter split-spoon sampler, driven in accordance with the SPT procedures (i.e. ASTM D1586). Borehole BH19-04 was advanced using dynamic cone penetration testing (DCPT); this test involves driving a 51 mm diameter, 60-degree apex cone attached to the tip of the drill rods through the soils using similar energy as the SPT technique, allowing blows per 0.3 m to be recorded as the cone advances through the strata. It should be noted that the DCPT values do not necessarily represent equivalent SPT N-values but rather reflect a continuous qualitative record of the relative densities/consistencies of the materials penetrated. Further, the friction between the soil and the drill rods (skin friction) can affect the DCPT values, especially within cohesive soils. The increase in skin friction can result in elevated DCPT values with depth and can be seen as a steady upwards creep of the DCPT values on the logs (i.e., between depth of approximately 1.2 m to 9.0 m below the existing ground surface within BH19-04). The SPT (N-value) and DCPT values are presented on the borehole logs. The results of the SPTs in terms of ‘N’ values are referred to in this report as consistency for cohesive soils and relative density for non-cohesive materials. Representative disturbed samples were also obtained directly from the auger cuttings. One (1) sample of soil was obtained from BH19-03 using a thin-walled Shelby tube sampler. Bedrock core samples were obtained from BH19-02 and BH19-03 using NQ sized diamond coring equipment. Soil samples recovered from the boreholes were placed in moisture proof bags and transported to our CCIL-certified laboratory for detailed classification and testing. Bedrock core samples were placed in bedrock core boxes and transported to our CCIL-certified laboratory for further assessment and classification.

The boreholes were backfilled on completion of the investigation using a mixture and layering of bentonite and compacted cuttings, with cold-patch asphalt used to cap the paved surface. Traffic control was achieved through the Client’s execution of a road closure with associated signage throughout the fieldwork.

Borehole elevations were measured utilizing a laser level, and in reference to a benchmark located on a metal plate situated south of the bridge/road, on the east bank, within concrete at grade and having the following coordinates (as provided by WSP’s Kingston team):

- Northing: 4916610.568 m;
- Easting: 318758.589 m; and
- Elevation: 93.634 m.

Borehole elevations are summarized on the Borehole Logs provided in **Appendix A**. Borehole elevations are for geotechnical engineering analytical purposes only and should be verified prior to finalizing any design or construction parameters upon which they are based.

2.2 LABORATORY ANALYSIS

Upon completion of drilling, all recovered soil and bedrock core samples were transported to WSP's CCIL-certified geotechnical laboratory for more detailed visual examinations and engineering classifications. All recovered soil samples were subjected to Moisture Content Tests (per ASTM 2216), three (3) soil samples were subjected to Particle Size Distribution Analyses (per ASTM D422) to assess gradation, textural descriptions and engineering classification, three (3) soil samples were subjected to Atterberg Limit testing (per ASTM D4318), and one (1) soil sample obtained using Shelby tube sampling methods was subjected to 1-dimensional consolidation testing (per ASTM D2435/D2435M-11).

Results of the laboratory tests are incorporated into the borehole logs in **Appendix A**, and the laboratory test reports are presented in **Appendix B**.

2.3 SAMPLE STORAGE

Unless requested in advance, the soil samples from the investigation will be stored in our laboratory facility for a period of no longer than three (3) months after the issuance of the final report.

3 SUBSURFACE CONDITIONS

3.1 GENERAL

This section presents a summary of the subsurface conditions encountered in the boreholes WSP advanced for this investigation. Also included is a summary of the conditions encountered in previous investigations performed by others.

Based on WSP's borehole information, the subsurface materials in the boreholes on the west bank generally consisted of topsoil or asphalt, over fill, over native soils consisting predominantly of silts, sands and/or clays (in varying compositions), underlain by granitic bedrock at about 1.8 metres below existing ground surface (mbeg) (approx. 91.9 m elev.). The subsurface conditions in the boreholes on the east bank generally consisted of asphalt over fill over layers of varying sandy, silty, and clayey soils, over granitic bedrock at about 12.9 mbeg to 13.6 mbeg (approx. 80.7 to 80.6 m elev.). Groundwater in the open boreholes was observed as shallow as about 2.1 mbeg (approx. 92.1 m elev.).

Previous boreholes performed and reported by others generally note topsoil or asphalt over fill with native soils consisting of clay and silty clay (with silt and sand seams) over granitic bedrock. It is noted that the log of one previous borehole in the east bank area notes a layer of boulders and cobbles approximately 3.3 m thick, within the silty clay. Groundwater in these previous boreholes ranged in depth from about 1.8 mbeg to 2.1 mbeg.

Individual soil units encountered in the boreholes are described in the following Sections.

3.2 TOPSOIL

A surficial layer of topsoil was encountered in borehole BH19-02, which was about 170 mm thick. The topsoil generally had a silty texture. This soil is expected to be devoid of any structural properties.

3.3 ASPHALT

Boreholes BH19-01, BH19-03, and BH19-05 encountered surficial asphalt ranging from about 50 mm to 100 mm in thickness. Surficial asphalt was present at borehole BH19-04, however due to the nature of the DCPT testing conducted, the thickness of the asphalt was not confirmed in this borehole.

SNC's previously advanced boreholes BH2 and BH3 encountered surficial asphalt ranging from 50 mm to 60 mm in thickness.

3.4 FILL

Immediately beneath the asphalt in BH19-01, BH19-03, and BH19-05 a layer of granular-like material was encountered. This material consisted of gravelly sand that ranged in thickness from about 150 mm to 650 mm (final depth of about 0.3 mbeg in BH19-01 and BH19-03). In borehole BH19-05, this layer extended to a depth of about 0.8 mbeg, beneath which a layer of fill was observed that is described as a sandy silt fill, extending to a depth of about 1.3 mbeg. Due to borehole BH18-04's location within the roadway, it is assumed that fill is present beneath the asphalt at this location, however due to the nature of the DCPT testing performed, the presence and thickness of any fill was not confirmed in this borehole.

The fill materials were light brown in colour, and in a moist (occasionally frozen) in-situ state – it is noted that the frozen condition will tend to correspondingly heighten N-counts obtained within such frozen material. Laboratory moisture content tests performed on samples of the fill yielded values ranging from approximately 7% to 24%

moisture by weight. A gradation test performed on sample BH19-01 CC-1 suggests the following gradation: 33% gravel, 60% sand, and 7% silt and clay.

All three of SNC's previously advanced boreholes encountered fill materials, to depths ranging from 0.3 mbeg to 1.3 mbeg. Their logs describe the fill immediately beneath the asphalt as "sand and gravel, trace silt, trace clay, compact, moist". Beneath this fill in their borehole BH2, and beneath the topsoil in their BH1, a layer of fill described as "silty clay, some sand, some gravel, firm, moist" was noted, which extended to depths of 1.2 mbeg to 1.3 mbeg.

3.5 SILTS AND CLAYS

3.5.1 WSP BOREHOLES

In boreholes BH19-01 and BH19-02 (located on the west bank), a layer of sandy silt was observed immediately below the fill (BH19-01) or topsoil (BH19-02) and extended to a depth of 1.5 mbeg. At this depth BH19-01 was terminated, while BH19-02 encountered a layer of silt that extended to 1.8 mbeg, at which depth bedrock was encountered. These soils are described as light brown to grey, with trace clay and occasional gravel, in a soft to very soft in-situ state of consistency and based on visual-tactile examination are described as at (or near) their plastic limit (APL). Moisture content tests performed on samples of these soils yielded values of 22 % to 24% moisture by dry weight.

In boreholes BH19-03 and BH19-05 (located on the east bank), the fill was underlain by a layer of sandy silt (BH19-03) or clayey silt (BH19-05). Borehole BH19-03 exhibited layers of sandy silt interbedded with thicker layers of clayey silt and silty clay. Combined, these layers extended to a depth of 12.2 mbeg in this borehole. In borehole BH19-05, similarly interbedded layers of clayey silt, silty clay, and sandy silt were observed, combined which extended to a depth of 11.0 mbeg. These soils are described as light brown to grey, in a very soft to stiff in-situ state of consistency (blow counts ranging from about 1 to 8 blows per 0.3 m, undisturbed shear strength from a vane test yielding greater than 74 kPa) and based on visual-tactile examination are described typically as about plastic limit (APL) to wetter than plastic limit (WTPL), and occasionally as drier than plastic limit (DTPL). Moisture content tests performed on samples of these soils yielded values of 21% to 36% moisture by dry weight. Gradation tests performed on samples of these soils yielded the following compositional ranges: 0% gravel, 3% to 10% sand, and 90% to 97% silt and clay-sized particles (18% to 69% less than 2 µm in size). A 1-dimensional consolidation test performed on a shelby tube sample obtained from BH19-03 between 2.3 mbeg and 2.9 mbeg yielded a specific gravity of 2.702, and analysis of the test results suggest a Compression Index (Cc) of 0.25.

3.5.2 SNC BOREHOLES

SNC's previously advanced boreholes BH1 and BH2 (located on the west bank) encountered a layer of sandy silty clay immediately beneath the fill and extending to practical refusal/bedrock at about 2.3 mbeg to 2.9 mbeg. Their borehole BH3 observed sandy silty clay beneath the fill extending to 5.2 mbeg, where a layer of red/black boulders and cobbles was present down to 8.5 mbeg, then was underlain by silty clay that extended to at least 9.8 mbeg. SNC reported moisture content levels ranging from 19 % to 29 % moisture by dry weight. Blow counts recorded by SNC within these soils ranged from 4 blows to 18 blows per 0.3 m.

3.5.3 BRMH BOREHOLES

BRMH's borehole BH No. 11 was located in the canal, near its west edge; its log records layers of dark grey clay with silt, occasional pockets of sand, and seams of silt, extending to the bedrock. They describe this clay as generally having a stiff consistency with some variation into firm or very stiff. This log displays unconfined compressive strength test results from within the clay, whose values range from 1.07 tons per square foot (tsf) to 1.90 tsf (102 kilopascals (kPa) to 182 kPa).

BRMH's borehole BH No. 7 was located in the canal, near its east edge; its log records layers of dark grey, fissured silty clay, occasional pockets of sand or silt at depth, extending to the bedrock. They describe this silty clay as generally having a stiff to very stiff consistency. This log displays results of vane testing performed at about 28.3 ft (8.6 m) depth, with values of 1.53 tsf (147 kPa) for undisturbed, 0.22 tsf (21 kPa) for remoulded, and a resulting sensitivity ratio (undisturbed/remoulded) of 7.0. This log also reports unconfined compressive strength test results from within the silty clay, whose values range from 1.55 tsf to 2.36 tsf (148 kPa to 226 kPa).

3.6 SAND AND SILTY SAND

A layer of soil described as either sand or silty sand was encountered in boreholes BH19-03 and BH19-05. These soils were first observed at depths ranging from about 11.0 mbeg (BH19-05) to 12.2 mbeg (BH19-03), and in both boreholes extended to the depth where practical refusal/bedrock was encountered.

These soils exhibited a grey colouration, and were in a saturated, loose to very loose in-situ state of relative density (based on visual-tactile examination and N-counts of 3 blows per 0.3 m).

3.7 BOULDERS AND COBBLES

SNC's borehole BH3 recorded a layer of red/black boulders and cobbles between depths of about 5.2 mbeg and 8.5 mbeg. Their report and log explain that this borehole was advanced through this material by coring.

It is noted that our boreholes BH19-03, BH19-04, and BH19-05 did not encounter evidence of this layer in any of these three boreholes.

3.8 DYNAMIC CONE PENETRATION TESTING

DCPTs were performed in borehole BH19-04 from surface to 13.7 mbeg, where the advancement of the DCPT testing was terminated due to practical refusal to further cone advancement. The presence of denser soils or boulder/cobbles or bedrock was inferred as the cause of the practical refusal, but this was not confirmed by diamond coring in this borehole. Because no soil samples were obtained during advancement of the DCPT, the type of materials penetrated by the DCPT is unknown. The blows per 0.3 m values obtained by the DCPT do reflect the relative densities of the materials penetrated.

SNC similarly advanced a DCPT in their borehole BH3 from 9.8 mbeg to the depth this borehole was terminated at (12.8 mbeg) where they reported practical refusal occurred.

3.9 BEDROCK (INFERRED AND CONFIRMED)

3.9.1 WSP BOREHOLES

In the west bank, borehole BH19-01 advanced to a depth of 1.5 mbeg without encountering practical refusal. Borehole BH19-02 encountered practical refusal at a depth of 1.8 mbeg (91.9 m elev.), and the presence of bedrock was confirmed at this depth by diamond coring down to 4.6 mbeg (89.1 m elev.). This bedrock consisted of red and black granitic bedrock of fair to good quality, with Rock Quality Designations (RQDs) of 58% to 78%.

In the east bank, borehole BH19-03 advanced to a depth of 12.9 mbeg (80.7 m elev.) where practical refusal was encountered. Diamond coring from that depth down to 17.7 mbeg (75.9 m elev.) confirmed light to dark grey granitic bedrock of very poor to fair quality, with RQDs of 19% to 67%. Borehole BH19-04 was advanced using DCPT methods (see **Section 3.8** for details), which encountered effective refusal at a depth of 13.7 mbeg (80.5 m elev.), at which depth the presence of either denser soils or boulder/cobbles or bedrock was inferred. Borehole

BH19-05 was advanced to a depth of 13.6 mbeg (80.6 m elev.) where practical refusal to further advancement was encountered and the presence of bedrock was inferred (but not confirmed by further diamond coring).

Laboratory testing was carried out on select sections. A summary of this testing is presented in the following Table.

Table 3.1 Results of Rock Core Testing for Unconfined Compressive Strength (UCS)

BOREHOLE NO.	CORE NUMBER	CORE INTERVAL (m)	SAMPLE INTERVAL (m)	DESITY (kg/m^3)	UNCONFINED COMPRESSIVE STRENGTH (MPa)
BH 19-02	RC1	1.78 to 3.07	2.46 to 2.67	2710	101.5
BH 19-03	RC6	17.09 to 17.73	17.53 to 17.68	2633	110.0
			Average	2672	105.8

3.9.2 SNC BOREHOLES

SNC's borehole BH1 (west bank) recorded red and black granite bedrock of excellent quality at a depth of 2.3 mbeg (91.5 m elev.), which they cored to 4.2 mbeg (89.6 m elev.). Their BH2 recorded practical refusal at 2.9 mbeg (90.9 m elev.). In the east bank, their BH3 was terminated at 12.8 mbeg (81.2 m elev.) without encountering practical refusal.

3.9.3 BRMH BOREHOLES

BRMH's borehole BH No. 11 was located in the canal, near its west edge; it records the water surface at "304.25 ft at a waterboard reading of 7 ft", bedrock at a depth of 28.6 ft (8.7 m) below the water surface and describes the bedrock as medium-grained grey granite.

Their borehole BH No. 7 was located in the canal, near its east edge; it records the water surface at "304.24 ft at a waterboard reading of 7 ft", bedrock at a depth of 32.4 ft (9.9 m) below the water surface and describes the bedrock as coarse-grained red, black and green granite traces of phlogopite.

3.10 GROUNDWATER

During drilling operations, open borehole BH19-01 was open to its full depth (1.5 mbeg) and remained free (effectively dry) of any groundwater accumulation. Open borehole BH19-05 exhibited groundwater accumulation at a depth of 2.1 mbeg (92.1m elev) during drilling. Due to the nature of the other three boreholes advanced by WSP (ie, coring operations or DCPT), groundwater measurements were not obtained from them.

SNC's boreholes BH1 and BH3 logs describe groundwater depths of 1.8 m and 2.1 m (respectively) during drilling. Their borehole BH2 log shows a monitoring well was installed to a depth of 2.9 m. On November 20, 2019 a groundwater depth of 2.28 mbeg was obtained from this well by WSP's field staff.

Note that groundwater levels are subject to seasonal fluctuations, specifically in response to extreme precipitation events and the spring thaw, as well as any changes in response to water level changes in the canal. As such variable levels should be anticipated, and the presence of groundwater is anticipated during construction, depending on site location and depth of construction works.

4 DISCUSSION AND RECOMMENDATIONS

4.1 BACKGROUND FROM SNC REPORT

An SNC report dated November 2018 summarizes a geotechnical investigation they performed in August 2018 for a then-proposed rehabilitation design of this bridge. Their report describes how the original Lower Brewers swing bridge was built in the 1870's, and underwent major reconstruction in 1984. Their Borehole Location Plan and Borehole Logs for their boreholes BH1 to BH3 are included as Appendix D herein. Section 6.1.1 of their report states "*Bedrock was found at a depth of 2.3 m bgs at BH1 and was inferred at 2.9 m bgs at BH2 at the west bank. However, bedrock was not contacted at BH3 on east bank up to 12.8 m bgs. Based on the results of the field investigation, new foundations could consist of spread or strip footings constructed on sound granitic bedrock on west bank and on competent native sandy silty clay soils on east bank. However, differential settlement can be expected to occur between foundations placed on bedrock (as encountered in BH1 and inferred in BH2) and relatively weak native clay overburden in BH3 and this aspect will need to be considered*."

Based on the differential settlement expected if footings are used for both west and east foundations, as requested WSP has performed further boreholes and testing (as described herein) to support provision of recommendations regarding a possible alternative foundation system for the proposed bridge replacement.

4.2 GENERAL

It is WSP's understanding that the Lower Brewers Swing Bridge, including its foundations, will be replaced as part of this project. This will consist of constructing new foundations for abutments on the west and east bank, new foundations for a centre pivot pier immediately west of the canal's west edge, and reconstruction of the approaches (which may include up to a 0.3 m grade raise).

Based on the results of our boreholes, and also considering the results of boreholes previously drilled by SNC and BRMH, from a foundation design perspective the subsurface conditions on the west bank, in the canal, and on the east bank can be generally summarized as follows:

- West bank, based on BH19-01, BH19-02, and SNC BH1 and BH2: asphalt/topsoil/fill over native soils consisting of silt and sandy silt over granitic bedrock at relatively shallow depths (1.8 to 2.9 mbeg), with groundwater at 1.8 to 2.3 mbeg;
- Canal (west side), based on BRMH BH-11: water over grey, stiff to very stiff clay to silty clay with some layers of sand or silt at depth, over granitic bedrock at a depth of 28.6 ft (8.7 m) below the water surface which was recorded by BRMH's log as being at "304.24 ft at a waterboard reading of 7 ft".
- Canal (east side), based on BRMH BH-7: water over grey, stiff to very stiff silty clay with some layers of sand or silt at depth, over granitic bedrock at a depth of 32.4 ft (9.9 m) below the water surface which was recorded by BRMH's log as being at "304.24 ft at a waterboard reading of 7 ft".
- East bank, based on BH19-03, BH19-04, BH19-05, and SNC BH3: asphalt/fill over native soils consisting of sandy silt, clayey silt, sandy silty clay, or silty clay, over bedrock (confirmed or inferred) at depths of 12.8 mbeg to 13.7 mbeg, with groundwater at about 2.1 mbeg. Note that SNC's borehole BH3 encountered a layer of boulders and cobbles between 5.2 mbeg and 8.5 mbeg (beneath which the borehole then extended through overburden soil to 12.8 mbeg).

It is important to note the bedrock depths as encountered in the boreholes. From west to east; the bedrock is relatively shallow in the proposed area of the west bank abutment, its depth increases in the area of the west bank pier, and it is at greater depth in the proposed area of the east bank abutment. The foundation design and construction must consider such sloped bedrock and depth variations across this project site, as well as the potential presence of obstructions such as boulders and cobbles within the overburden soils.

The following recommendations for design and construction of the proposed new bridge are based on the borehole information provided in **Section 3** which includes WSP's borehole data as well as supplementary borehole data from other Consultant reports and included in this report as **Appendices D** and **E**. While we believe our findings are reasonably representative, conditions may vary between and beyond the investigated borehole locations. If significant differences in the subsurface conditions described above are found at a later time, WSP should be contacted immediately to review and, if necessary, update our findings and recommendations.

These recommendations are intended for the Designers only, and should not be construed as instructions to Contractors, who should form their own opinions about Site conditions for tendering purposes, and to determine appropriate equipment, construction methods, and their costs. It is WSP's understanding that final grading of the approaches will not exceed 0.3 m higher than existing grade.

4.3 SITE PREPARATION

In the area of the west bank abutment and from beneath the reconstructed approaches; the asphalt, fill, existing foundations, as well as all organics and organic-bearing materials should be stripped/removed properly from the site. The existing foundations should be removed to the limits shown on the contract drawings, prior to preparing these areas for advancement of the new foundations.

Prepared subgrade for pavement areas should be proof-rolled using a self-propelled vibratory compactor or smooth drum roller. Care must be taken to ensure the subgrades for pavement areas does not become overly wet or loosened (i.e., liquified) in isolated zones through overly aggressive compaction of the subgrade or other construction operations, or through exposure to the climatic elements.

Proof-rolling should be completed in the presence of a qualified Geotechnical Engineer or qualified personnel working under the direct supervision of a Geotechnical Engineer, who must approve the founding subgrade prior to placement of any fill. Any loose or soft subsoils encountered within the new approach areas should be subexcavated and replaced with suitable fill approved by the Geotechnical Engineer, and placed in maximum 200 mm lifts, subsequently compacted to 100% Standard Proctor Maximum Dry Density (SPMDD) standards (ASTM D698). Moisture adjustments may be required to compact materials to the required design standards, as directed by the Geotechnical Engineer.

Silt and/or clay soils may become soft/weak or otherwise unstable when construction loads are applied in wet weather conditions. This material may require stabilization or full removal from beneath the new approaches and/or abutments, subject to the moisture conditions at the time of construction. This material may also be frost susceptible and should be removed from below footings that are exposed to freezing.

The native subsoils are susceptible to strength loss or deformation if saturated or disturbed by construction traffic. The contractor is responsible for the techniques and methods they utilize, including during the subgrade preparation stage during which the subgrade can be sensitive and susceptible to strength losses if inappropriate equipment and/or techniques are used. Care must be taken to protect the exposed subgrade from excess moisture and from construction traffic. The contractor and WSP should assess the subgrade soils as they become exposed and given the prevailing climatic conditions at that time decide then whether application of a 75 mm thick lean concrete mud mat (or other strategy) is warranted to protect the exposed subgrade from the elements and construction traffic.

4.4 EXCAVATIONS, DEWATERING, BACKFILL

Temporary excavations should be carried out to conform to the manner specified in Ontario Regulation 213/91 and the Occupational Health and Safety Act and Regulations for Construction Projects (OHSA). All excavations above the water table not exceeding 1.2 m in depth may be constructed with unsupported slopes. In general, the soils are classified as Type 4 material in accordance with OHSA, requiring temporary excavation side-slopes to be sloped at 3 Horizontal to 1 Vertical (3H:1V) or flatter, or they must be properly supported (shored). These classifications must be reviewed and confirmed by a qualified person during excavation.

Stockpiling of soil beside the excavations should be avoided; the weight of the stockpiled soil could lead to basal instability of braced excavations or slope instability of unsupported excavations.

Excavations should be protected from exposure to precipitation and associated ground surface runoff and should be inspected regularly for signs of instability. If localized instability is noted during excavation, or if wet conditions are encountered, side slopes should be flattened to maintain safe working conditions. Stockpiling of excavated soils adjacent to the excavation must be avoided, to prevent causing related instabilities in the excavation's sidewalls and/or base.

Excavations extending deeper than the groundwater table at this site will encountered groundwater infiltration into open excavations. See **Section 3.10** for information about the groundwater levels encountered during this investigation. The degree of groundwater inflow and resulting dewatering, will depend on the depth of excavation required below the groundwater table, and the final design and construction methods it entails. If excavations remain above the groundwater table, then this will minimize the construction dewatering required. If excavations extend below the groundwater table, then the amount of construction dewatering and/or groundwater control required will increase in relation to the depth of excavation below the groundwater table.

Based on excavations only advancing to the base of the abutments, this should minimize the degree of groundwater control and dewatering required. The west abutment excavation will extend to the bedrock (1.8 to 2.9 mbeg), with bedrock encountered at 1.8 to 2.3 mbeg. The west bank pier and east abutment excavations will extend to an appropriate depth to allow advancement of the deep foundation elements and construction of the pier and abutment at a suitable depth. In both excavations it is expected there will be some amount of groundwater infiltration but should be manageable with a system of sheet piling, cofferdams, and localized dewatering within such elements. It is strongly recommended that the contractor retain a specialist in design and installation of such shoring and groundwater control elements under these conditions. This assessment does not represent an engineering design of a dewatering operation, but a preliminary analysis based on the available data. The selection of the dewatering method(s) and actual design of the dewatering operation will be the responsibility of the contractor.

It should be noted that groundwater control measures that extract more than 50,000 L/day of water are subject to a dewatering permit as regulated by the Ministry of the Environment, Conservation and Parks (MECP). It may be necessary to obtain an Environmental Activity and Sector Registry (EASR) for construction dewatering between 50,000 to 400,000 L/day or a Permit To Take Water (PTTW) for dewatering greater than 400,000 L/day. Pumping discharges should conform to the required regional or municipal by-laws. The PTTW would require that a hydrogeologic investigation be completed and submitted along with the PTTW application and the corresponding fee from the applicant. MECP typically requires at least three (3) months to review PTTW applications.

The soils present at the site are expected to be sensitive to disturbance and proper control of the groundwater infiltration will be required to prevent excessive disturbance. Failure to adequately control groundwater inflows may result in disturbance of the subgrade and a need for over-excavation and replacement of disturbed subgrade soil.

Some excavated inorganic soils may be suitable for use as approach pavement subgrade backfill. These will generally consist of more granular or sandy soils. The finer-grained soils (including silty and clays) as well as organics will not be suitable for reuse as backfill. The reuse of any existing excavated soils is conditional on it being workable, at a suitable moisture content, and receiving final review and approval for such reuse from a Geotechnical Engineer or representative at the time of construction. Some soils will require prior processing (such as aeration) to lower its moisture content before being considered for approval as backfill material. If site soils cannot be reused as backfill, then an OPSS Granular B Type 1 material is recommended for general backfilling.

Footings and walls exposed to frost action should be backfilled with free-draining, non-frost susceptible material such as OPSS 1010 Granular B Type II, or an approved equivalent. Imported material should be screened and approved by the Geotechnical Engineer or representative before being delivered to the Site. Screening should also confirm that any imported fill meets the Environmental Standards for the Site.

Care should be taken immediately adjacent to foundation walls to avoid over compaction of the soil and resulting wedging pressures, which may result in damage to the walls.

4.5 FOUNDATIONS

In the west bank abutment area, bedrock was encountered at shallow depths of approximately 1.8 mbeg to 2.9 mbeg (91.9 to 90.9 m elev.), with groundwater observed at about 1.8 mbeg to 2.3 mbeg (about 92.0 m to 91.5 m elev.). Structural loading for the proposed bridge's west-bank abutment could be supported on shallow concrete footings, placed either directly on the competent bedrock, or on at least 0.3 m of engineered fill placed on the competent bedrock. Based on the relatively high ground water levels, and the need for deep foundations at the east bank abutment and the pivot pier, it is however WSP's understanding that the designers of the new structure have elected use a similar deep foundation at the west bank abutment. Guidance and recommendations for shallow foundations at the west bank area could be provided if the design approach changes.

It is WSP's understanding that a new foundation pivot pier is to be located immediately west of the canal's west edge. Based on the bedrock depth of 28.6 ft (8.7 m) reported by BRMH's BH-11 that was located in the canal near its west edge, it appears that deep foundations into the bedrock will be required for this new pier.

Variable, generally soft/loose soils were present in the boreholes on the east bank; these soils extended to bedrock at depths of 12.8 mbeg to 13.7 mbeg (80.7 m to 80.6 m elev.), with groundwater present at about 2.1 mbeg (92.1 m elev.). To minimize any differential settlements relative to the west bank foundations, a shallow conventional foundation system (footings) would require removal of these soils full-depth down to the bedrock prior to placement of engineered fill. Considering the depth required, and the need for corresponding groundwater control, this approach is considered unfeasible.

In order to minimize foundation excavation depths and related groundwater challenges, and to minimize differential settlements between the various founding elements, it is recommended that the bridge foundations be supported on deep foundations advanced down to and into the underlying bedrock.

Considering the presence of a boulder and cobble layer as reported in SNC's borehole BH3 between 5.2 mbeg and 8.5 mbeg, the deep foundations used should be capable of advancing through such potential obstructions. For this reason, cased micropiles are recommended to support the new bridge foundations.

Micropiles will be able to advance through/past obstructions including any boulders and/or cobbles, will advance into the bedrock, and are ideal in areas with a sloped or undulating rock surface where other tradition deep foundation methods do not perform well or will experience excessive construction delays to clear the boulders/cobbles. Micropiles will also work in any areas where deeper overburden is encountered. Micropiles are therefore recommended as they are expected to suit all conditions to be encountered at this site.

Micropile foundation systems are typically proprietary, and we would recommend that the information presented herein is to assist Designers. We recommend that the final tender documents allow for this item to be a "Design/Build" type, as this type of foundation must be designed and installed by a firm specializing in such systems. Such firms must provide the load capacities (ULS and SLS) for their micropile systems.

Based on the results from the boreholes, suitable resistance (for loads in compression) within the bedrock is anticipated for the new micropile foundations. It is noted that the bedrock quality was typically lower in the upper portion of bedrock (see RQD values on the borehole logs), and therefore the design of the micropiles should take this into consideration and be advanced beyond this upper 1.5 m of bedrock to encounter at least fair to good quality bedrock.

Socketing of the micropiles into the bedrock may also be required for uplift resistance (loads in tension). Socket depth should be determined by the designer of the micropiles. The ultimate bond stress between grout and bedrock is typically taken as 10% of the unconfined compressive strength of the bedrock or the compressive strength of the grout material whichever is less, but not more than 3.1 MPa. Considering the granitic nature of the bedrock and the higher compressive strength values granitic bedrock typically provides, the compressive strength of the grout material should dictate the design (up to the maximum 3.1 MPa). The allowable bond stress between the rock and the grout is normally 50% or less of the ultimate bond stress, (ie, Safety Factor of at least 2.0). The upper 1.5 m of the bedrock should not be included as part of the bond length, because this zone is typically weathered/fractured, and does not typically allow development of the required ultimate bond stresses within that zone. Considerations should be made for tension and compression load testing, during construction, to confirm that design capacities are being achieved in the field.

In order to minimize the effects of adfreezing on the micropiles, it is recommended that the pile caps and other foundation elements be backfilled with a coarse grained non frost susceptible soil. One example would be an OPSS Granular 'B', Type II. Also, we recommend that the use of a bond break be incorporated into the design of the foundation system to prevent frost adhering to foundation elements.

The actual pile capacities can vary over a large range due to many factors including but not limited to penetrated and bearing strata, and construction practices used. The design parameters provided in this report should therefore be considered preliminary. It is recommended that a specialist micropiling contractor review the borehole logs and verify micropiling suitability and capacity based on experience under similar conditions.

The design and installation of the micropiles must take into consideration what appears to be an undulating / uneven / sloped nature of the bedrock topography (into which the micropiles must be advanced). Because of the variable and uneven bedrock depths anticipated across this project site, it is recommended that the construction tender require a per-unit price for depth of micropile installation, to accommodate such variations.

The geotechnical resistance of micropiles is sensitive to construction processes and quality control, such as grouting and borehole flushing procedures. Performance testing of sacrificial micropiles and proof testing of production micropiles (compression and tensile loadings) is recommended to demonstrate that micropiles are designed and installed properly, and that they will achieve the required structural capacities. The quantity of tests, loads and other information is provided in the Micropile Specification 31 63 19 included in overall design package. Testing is to be performed in accordance with FHWA-SA-97-070, ASTM D1143/D1143M-07e1 and ASTM D3689-07 procedures.

The piles should be installed under the supervision of a professional geotechnical engineer to ensure that the piles are constructed in accordance with the aforementioned recommendations. Detailed installation records should also be taken both by the Contractor and the supervising party and submitted to the Engineer of Record.

4.6 FROST PENETRATION DEPTH

The depth of frost penetration for the site is 1.5 m as per Ontario Provincial Standard Drawing (OPSD) 3090.101. All foundation elements should therefore have a permanent soil cover of at least 1.5 m (or its thermal equivalent if artificial insulation is used). If thermal insulation is used, product specifications and installation directions should be obtained from the manufacturer.

Note that a suitably prepared and approved bedrock subgrade at this site should be free of fractured / weathered rock and would not be considered frost susceptible. A final determination of this must be obtained at the time of construction, through appropriate geotechnical inspections and approval of the exposed subgrade materials. To ensure exposed bedrock is not frost susceptible, seal any exposed fractures, voids, joints, cracks, crevices etc. into which water could penetrate.

4.7 SEISMIC SITE CLASS

For design purposes, based on the criteria listed in Table 4.1 of the 2014 Canadian Highway Bridge Design Code (CHBDC) and the results of our boreholes, the Seismic Site Class of this project site can be conservatively classified as Site Class C on the west bank and as Site Class D on the east bank.

If verification (and possible optimization) of the Seismic Site Class is required, consideration can be given to performing a Multi-channel Analysis of Surface Waves (MASW) geophysical survey to confirm the site's shear wave velocity, Vs. at the site.

4.8 CORROSIVITY POTENTIAL

One (1) soil sample was submitted to SGS Canada Inc. (SGS) for laboratory analyses of redox potential, sulphide, moisture content, pH, chloride, sulphate, conductivity, and resistivity to assess the corrosive potential of the soil. The tested sample was a composite made up of combining soil from all samples obtained from BH19-03.

The

Table 4.1 summarizes the ANSI/AWWA rating for the tested soil sample with respect to the potential for corrosion towards buried grey or ductile cast iron pipe. A score of 10 points or more indicates potential for corrosion. Based on the results and associated rating, the tested soil sample has a low corrosivity potential. It should be noted that there are other factors which may influence the corrosion potential such as; the nature of effluent conveyed, the application of de-icing salts on the site and subsequent leaching into the subsoils; and stray currents.

Table 4.1 Results of ANSI/AWWA Soil Corrosivity Potential Rating and Sulphate Content

SAMPLE I.D.	SULPHATE CONTENT (µg/g or ppm)	RESISTIVITY (ohms-cm)	PH	REDOX POTENTIAL (mV)	SULPHIDE (%)	MOISTURE CONTENT (%)	TOTAL POINTS
BH19-03-Composite	20	8750/0	8.67/3	158/0	<0.02/Trace/2	22.7/Poor Drainage/2	7

Regarding the potential for sulphate attack on concrete, Table 12 of the Canadian Standards Association (CSA) document A23.1 09 “Concrete Materials and Methods of Concrete Construction” divides the degree of exposure into the following three classes:

Degree (Class) of Exposure	Water Soluble (SO_4) in Soil Samples (%)
Very Severe (S-1)	>2.0
Severe (S-2)	0.2 – 2.0
Moderate (S-3)	0.1 – 0.2

The sulphate level within the soil sample tested was 20 ppm (<0.1%). Samples having low water-soluble sulphate content (i.e. <0.1%) is considered to have negligible potential for sulphate attack and do not require special cements or mixtures. Class S-3 concrete may be used, as the potential for sulphate attack is less than the degree of exposure outlined for Class S-3 concrete.

4.9 APPROACHES (ABUTMENTS AND EMBANKMENTS)

It is recommended that free draining non-frost susceptible granular material such as Granular “B” Type II, in accordance with OPSS Form 1010 (and having maximum aggregate diameter of 100 mm), be provided as backfill to the abutments. The backfill should be placed in lifts not exceeding 200 mm before compaction and compacted to 98% of its SPMDD. The backfill should be completed as per OPSD 3101.150. Provisions for drainage of the backfill should be implemented.

With positive drainage behind the abutments, and using Granular B Type II, the following lateral earth pressure parameters are recommended for design purposes:

Compacted Granular “B” Type II:

- Internal Friction Angle (Ø) = 30°

- Active Earth Pressure Coefficient (K_a) = 0.33
- Coefficient Earth Pressure at Rest (K_o) = 0.50
- Assumed unit weight 21 kN/m3

For slope stability purposes, it is recommended that earth slopes constructed during this project be constructed of free-draining materials, and maintain a maximum, non-mechanically stabilized gradient of 3 horizontal to 1 vertical (3H:1V) or flatter. This gradient is expected to ensure mass global stability of the slopes. If steeper gradients are required, slope stabilization or retaining features should be incorporated into the slope's design. The embankment slopes, and any drainage system runoff areas constructed, should be protected from surficial degradation and erosion by incorporating appropriate erosion-control features including (but not limited to): installation of erosion mats, gabion baskets, sodding and mulching (or other appropriate vegetation), rip rap, and/or other acceptable stabilizing features.

It is WSP's understanding there may be a grade raise of up to 0.3 m in the reconstructed approaches. Based on the borehole results, the consolidation testing performed, and this maximum proposed grade raise, the corresponding estimated settlement of the approach embankments are expected to be negligible.

4.10 APPROACH PAVEMENT RECONSTRUCTION

The rebuilt approach structure should match the existing, adjacent pavement structure but as a minimum consist of at least 40 mm of HL3, HL4 or SP 12.5 mm over 50 mm of HL8 or SP 19.0 mm overlaying a compacted granular base consisting of 150 mm of Granular “A” over 450 mm of Granular “B”. A Traffic Category of Level B has been assumed appropriate for this road and the asphaltic cement grade should be PG 58-28 for all courses since the design ESAL is below 3.0 million. Frost tapers should be utilized (at 10H:1V or flatter) to join differential granular depths.

It is recommended that all granular fill material be placed in uniform lifts not exceeding 200 mm in thickness before compaction. It is suggested that all granular material used as fill should have an in-situ moisture content within 2% of their optimum moisture content. All granular materials should be compacted to 100% of their SPMDD. Granular materials should consist of Granular “A” and “B” conforming to the requirements of OPSS Form 1010 or equivalent.

4.11 WINTER CONSTRUCTION

The subsoils encountered across the site are frost-susceptible and freezing conditions could cause problems to the structures. As preventive measures, the following recommendations are presented:

1. During winter construction, exposed surfaces intended to support foundations including pile caps must be protected against freezing by means of loose straw, propane heaters, polystyrene insulation, insulated tarpaulins, or other suitable means that prevent freeze movements.
2. Because of the frost heave potential of the soils during winter, it is recommended that the trenches for exterior underground services be excavated with shallow transition slopes in order to minimize the abrupt change in density between the granular backfill, which is relatively non-frost susceptible, and the more frost-susceptible native soils.

4.12 DESIGN REVIEW AND INSPECTION

It is WSP's understanding that the detailed design of the micro piles are proposed as a design-build, WSP's geotechnical group must be allowed to review the foundation design and proposed final grading plans, prior to their finalization. In addition, we strongly recommend that our firm be retained to review the related earthworks specifications when they are available.

Geotechnical inspection and review of foundation excavations and compaction procedures should be carried out to ensure compliance with our recommendations. Full time geotechnical inspection during the installation of the micropile is heavily recommended due to the hidden nature of the micropile construction which can not be verified post construction. Grout testing and load testing of the micropiles is also recommended. Grout testing and recording the micropile installations can be typically carried out by most professional geotechnical engineers. Load testing of the micropiles is typically carried out by the Contractor under the supervision of a professional geotechnical engineer.

5 GENERAL COMMENTS AND LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Group Ltd. at the time of preparation. Unless otherwise agreed in writing by WSP Canada Group Ltd., it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

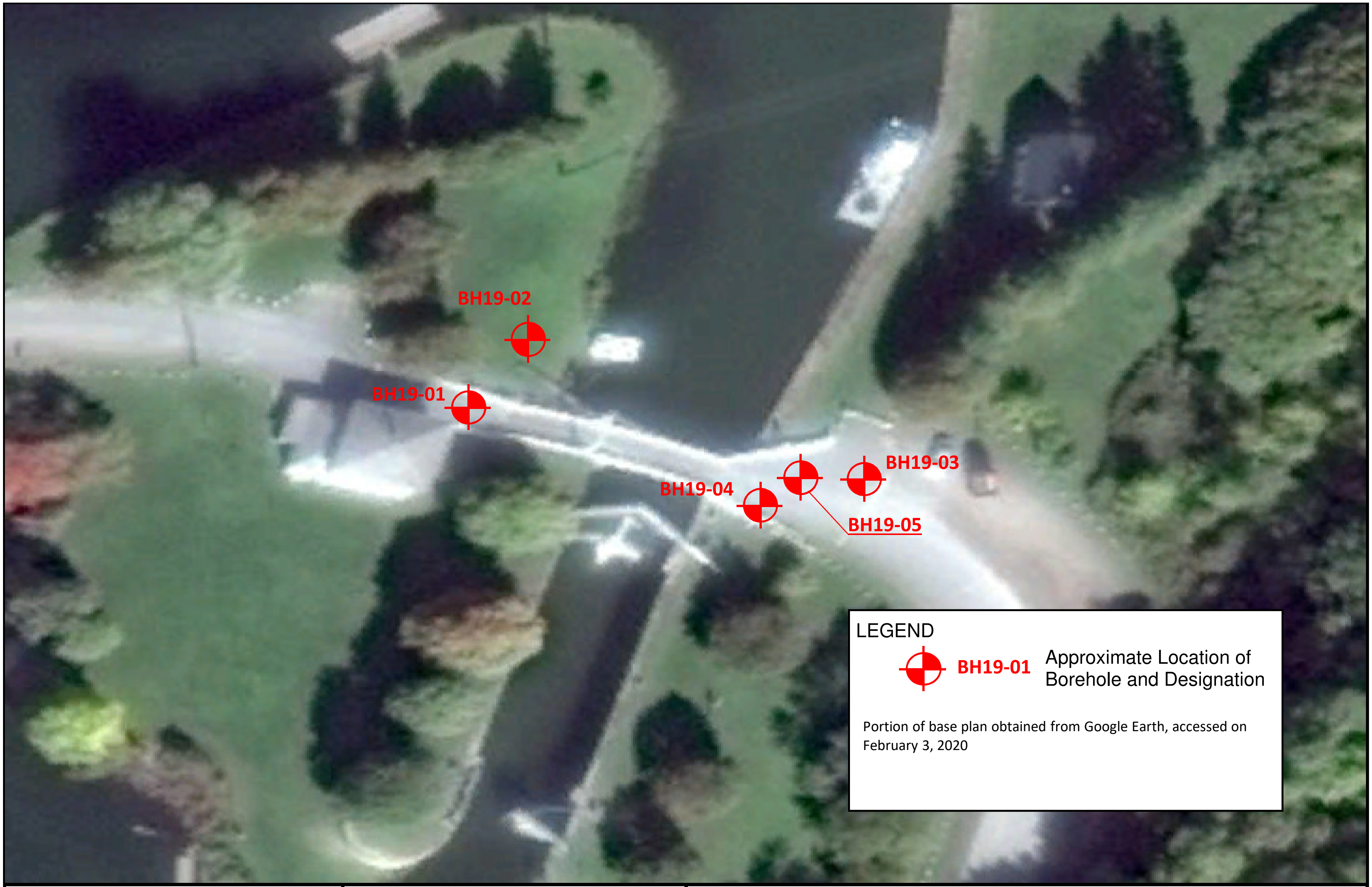
The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP Canada Group Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

6 CLOSURE

Thank you for the opportunity to be of service to you. Should you have any questions or require further clarification on any aspect of this report, please do not hesitate to contact this office.

FIGURES



Portion of base plan obtained from Google Earth, accessed on February 3, 2020

N

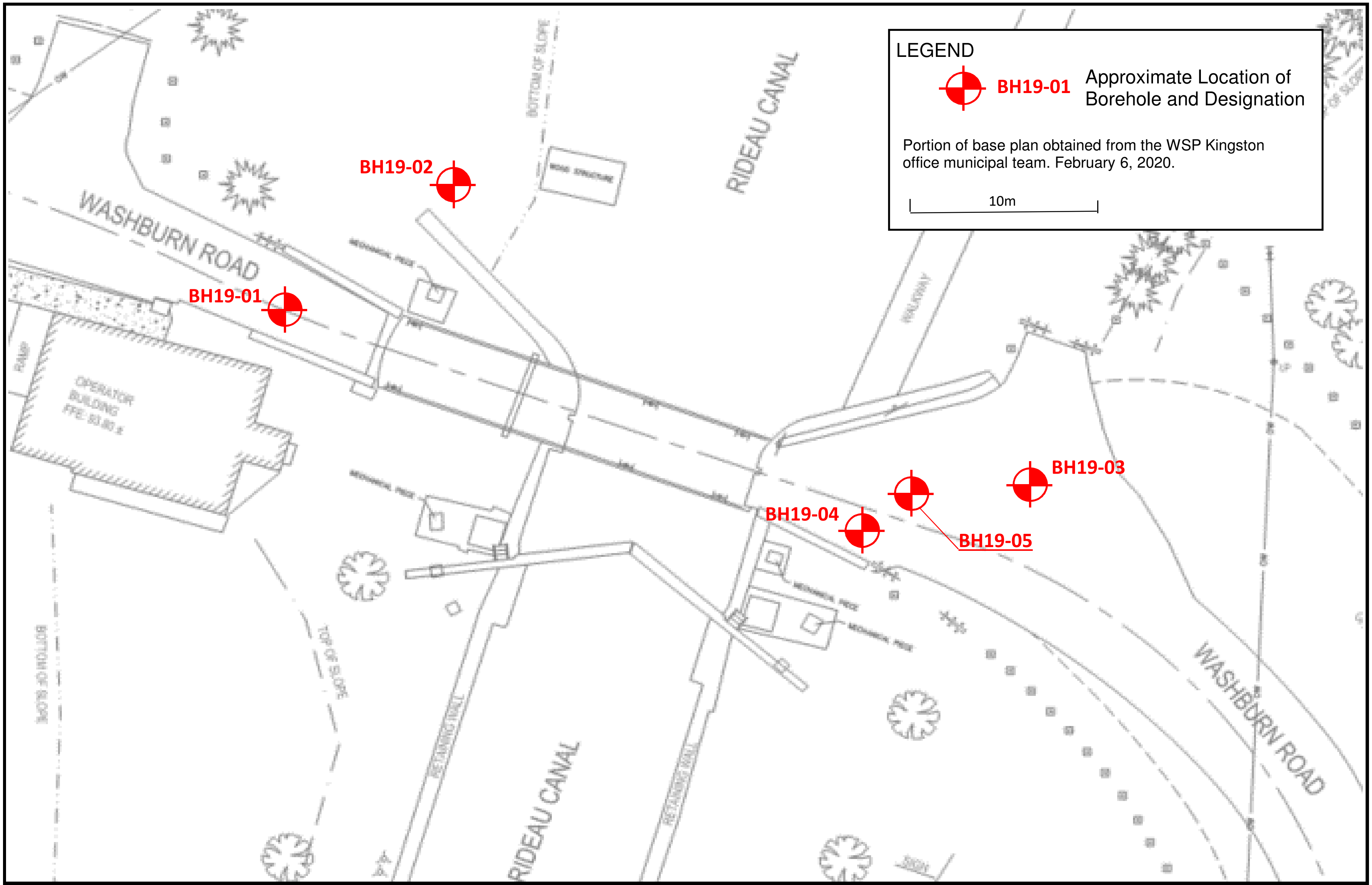
REF. NO.: 19M-01599-00 F1
DATE: FEBRUARY, 2020
PROJECT: 19M-01599-00

BOREHOLE LOCATION PLAN

GEOTECHNICAL INVESTIGATION
LOWER BREWERS SWING BRIDGE REPLACEMENT
KINGSTON, ON
FOR: PARKS CANADA

FIGURE

1



LEGEND

🟥 **BH19-01** Approximate Location of Borehole and Designation

Portion of base plan obtained from the WSP Kingston office municipal team. February 6, 2020.

N

REF. NO.:	19M-01599-00 F2
DATE:	FEBRUARY, 2020
PROJECT:	19M-01599-00

wsp

BOREHOLE LOCATION PLAN

GEOTECHNICAL INVESTIGATION
LOWER BREWERS SWING BRIDGE REPLACEMENT
KINGSTON, ON
FOR: PARKS CANADA

FIGURE

2

APPENDIX

A

BOREHOLE LOG EXPLANATION FORM AND BOREHOLE LOGS

BOREHOLE LOG EXPLANATION FORM

This explanatory section provides the background to assist in the use of the borehole logs. Each of the headings used on the borehole log, is briefly explained.

DEPTH

This column gives the depth of interpreted geologic contacts in metres below ground surface.

STRATIGRAPHIC DESCRIPTION

This column gives a description of the soil based on a tactile examination of the samples and/or laboratory test results. Each stratum is described according to the following classification and terminology.

Soil Classification*		Terminology	Proportion
Silt & Clay	< 0.075 mm	"trace" (e.g. trace sand)	<10%
Sand	0.075 to 4.75 mm	"some" (e.g. some sand)	10% - 20%
Gravel	4.75 to 75 mm	adjective (e.g. sandy)	20% - 35%
Cobbles	75 to 300 mm	"and" (e.g. and sand)	35% - 50%
Boulders	>300 mm	noun (e.g. sand)	>50%

* Extension of USCS Classification system unless otherwise noted.

The use of the geologic term "till" implies that both disseminated coarser grained (sand, gravel, cobbles or boulders) particles and finer grained (silt and clay) particles may occur within the described matrix.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

COHESIONLESS SOIL		COHESIVE SOIL	
Compactness	Standard Penetration Resistance "N", Blows / 0.3 m	Consistency	Standard Penetration Resistance "N", Blows / 0.3 m
Very Loose	0 to 4	Very Soft	0 to 2
Loose	4 to 10	Soft	2 to 4
Compact	10 to 30	Firm	4 to 8
Dense	30 to 50	Stiff	8 to 15
Very Dense	Over 50	Very Stiff	15 to 30
		Hard	Over 30

The moisture conditions of cohesionless and cohesive soils are defined as follows.

COHESIONLESS SOILS	COHESIVE SOILS		
Dry	DTPL	-	Drier Than Plastic Limit
Moist	APL	-	About Plastic Limit
Wet	WTPL	-	Wetter Than Plastic Limit
Saturated	MWTPL	-	Much Wetter Than Plastic Limit

STRATIGRAPHY

Symbols may be used to pictorially identify the interpreted stratigraphy of the soil and rock strata.

MONITOR DETAILS

This column shows the position and designation of standpipe and/or piezometer ground water monitors installed in the borehole. Also the water level may be shown for the date indicated.

●	Standpipe	■□■	Geotextile Material / Liner	░	Granular Backfill
▲	Piezometer	▧	Borehole Seal (Bentonite Grout)	▒	Granular (Filter) Pack
☰	Screened Interval	▨	Cement Seal	□□□	Native Soil Backfill / Cave / Slough
▧	Borehole Seal (Peltonite, Bentonite or Hole Plug)				

Where monitors are placed in separate boreholes, these are shown individually in the "Monitor Details" column. Otherwise, monitors are in the same borehole. For further data regarding seals, screens, etc., the reader is referred to the summary of monitor details table.

SAMPLE

These columns describe the sample type and number, the "N" value, the water content, the percentage recovery, and Rock Quality Designation (RQD), of each sample obtained from the borehole where applicable. The information is recorded at the approximate depth at which the sample was obtained. The legend for sample type is explained below.

SS = Split Spoon
ST = Thin Walled Shelby Tube
AS = Auger Flight Sample
CC = Continuous Core
GS = Grab Sample
CS = Channel Sample
WS = Wash Sample
RC = Rock Core

$$\% \text{ Recovery} = \frac{\text{Length of Core Recovered Per Run}}{\text{Total Length of Run}} \times 100$$

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD (%)
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

TEST DATA

The central section of the log provides graphs which are used to plot selected field and laboratory test results at the depth at which they were carried out. The plotting scales are shown at the head of the column.

Dynamic Penetration Resistance - The number of blows required to advance a 51 mm diameter, 60º steel cone fitted to the end of 45 mm OD drill rods, 0.3 m into the subsoil. The cone is driven with a 63.5 kg hammer over a fall of 750 mm.

Standard Penetration Resistance - Standard Penetration Test (SPT) "N" Value - The number of blows required to advance a 51 mm diameter standard split-spoon sampler 300 mm into the subsoil, driven by means of a 63.5 kg hammer falling freely a distance of 750 mm. In cases where the split spoon does not penetrate 300 mm, the number of blows over the distance of actual penetration in millimetres is shown as $\frac{xBlows}{mm}$

Water Content -	The ratio of the mass of water to the mass of oven-dry solids in the soil expressed as a percentage.
W_P -	Plastic Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.
W_L -	Liquid Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

REMARKS

The last column describes pertinent drilling details, field observations and/or provides an indication of other field or laboratory tests that were performed.

BOREHOLE NO. BH19-01

PAGE 1 of 1

PROJECT NAME: LOWER BREWERS SWING BRIDGE

PROJECT NO.: 19M-01599-00

CLIENT: PARKS CANADA

DATE COMPLETED: Nov 18, 2019

BOREHOLE TYPE: 82 mm DIRECT PUSH

SUPERVISOR: MN

GROUND ELEVATION: 95.5 m

REVIEWER: GB

DEPTH (m)	ELEV (mASL)	STRATIGRAPHIC DESCRIPTION	STRATIGRAPHY	MONITOR DETAILS	SAMPLE TYPE	N VALUE	% WATER	% RECOVERY	RQD (%)	CONE PENETRATION "N" VALUE 10 20 30 SHEAR STRENGTH 20 40 60 80 Intact (MaX) Cu Remoulded Cu	WATER CONTENT % 10 20 30 WP WL	REMARKS
0.0	95.5											
0.1	95.4	ASPHALT (100mm)			CC1a							
0.3	95.2	FILL:(150mm) Light brown gravelly sand FILL, trace silt, moist										
1.0		SANDY SILT: Light brown to grey SANDY SILT, trace clay, occasional gravel, APL			CC1b		22	70			●	GSA CC1A: Gravel: 33% Sand: 60% Silt & Clay: 7% Borehole open and dry upon completion of drilling.
1.5	94.0	Borehole terminated at 1.5 m below ground surface in SANDY SILT.										
2.0												
3.0												
4.0												
5.0												
6.0												
7.0												
8.0												
9.0												
10.0												
11.0												
12.0												
13.0												
14.0												
15.0												
16.0												
17.0												
18.0												
19.0												
20.0												

BOREHOLE NO. BH19-02

PAGE 1 of 1

PROJECT NAME: LOWER BREWERS SWING BRIDGE

PROJECT NO.: 19M-01599-00

CLIENT: PARKS CANADA

DATE COMPLETED: Nov 18, 2019

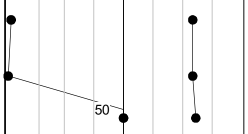
BOREHOLE TYPE: 82 mm DIRECT PUSH

SUPERVISOR: MN

GROUND ELEVATION: 93.7 m

REVIEWER: GB

DEPTH (m)	ELEV (mASL)	STRATIGRAPHIC DESCRIPTION	STRATIGRAPHY	MONITOR DETAILS	SAMPLE TYPE	SAMPLE N VALUE	SAMPLE % WATER	SAMPLE % RECOVERY	SAMPLE RQD (%)	CONE PENETRATION "N" VALUE 10 20 30 SHEAR STRENGTH 20 40 60 80 Intact (MaX) Cu Remoulded Cu	WATER CONTENT % 10 20 30 WP WL	REMARKS
0.0	93.7	TOPSOIL										
0.2	93.5	SANDY SILT: Grey SANDY SILT, trace clay, occasional gravel, DTPL, soft to very soft - Light brown, APL			SS1	2	23	58				
1.0					SS2	1	23	75				
1.5	92.2	SILT: Light brown SILT, some clay, trace sand, APL, very soft			SS3	50/ 75mm	24	100				
1.8	91.9	GRANITE BEDROCK: Red and black, fair to good quality										
2.0					RC1			100	78			
3.0												
4.0					RC2			100	58			
4.6	89.1	Borehole terminated at 4.6 m below ground surface in GRANITE BEDROCK.										
5.0												
6.0												
7.0												
8.0												
9.0												
10.0												
11.0												
12.0												
13.0												
14.0												
15.0												
16.0												
17.0												
18.0												
19.0												
20.0												

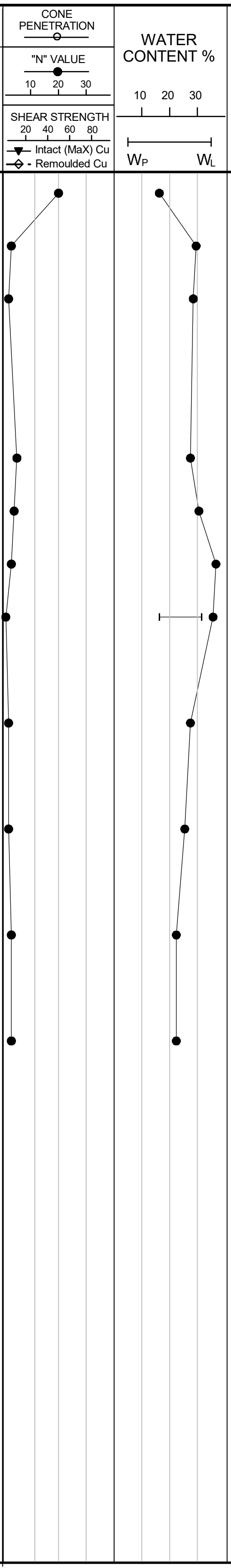


BOREHOLE NO. BH19-03

PROJECT NAME: LOWER BREWERS SWING BRIDGE
PROJECT NO.: 19M-01599-00
CLIENT: PARKS CANADA
DATE COMPLETED: Nov 19, 2019
BOREHOLE TYPE: 82 mm DIRECT PUSH
SUPERVISOR: MN
GROUND ELEVATION: 93.6 m
REVIEWER: GB

DEPTH (m)	ELEV (mASL)	STRATIGRAPHIC DESCRIPTION	MONITOR DETAILS
0.0	93.6		
0.1	93.6	ASPHALT (50mm)	
0.3	93.3	FILL: Light brown gravelly sand FILL, moist, compact	
1.0	92.6	SANDY SILT: Grey / light brown SANDY SILT, trace clay, occasional gravel, DTPL, very stiff to soft	
		CLAYEY SILT: Grey CLAYEY SILT, trace sand, APL, soft to firm - Stiff - Firm	
4.6	89.0	SANDY SILT: Light brown SANDY SILT, trace to some clay, APL, soft to firm	
5.3	88.3	CLAYEY SILT: Grey CLAYEY SILT, trace sand, APL, soft	
6.1	87.5	SILTY CLAY: Grey SILTY CLAY, WTPL, very soft to soft	
9.2	84.4	CLAYEY SILT: Grey CLAYEY SILT, trace sand, WTPL, soft	
10.8	82.8	SANDY SILT: Grey SANDY SILT, trace clay, saturated, very loose	
12.2	81.4	SILTY SAND: Grey SILTY SAND to some silt, saturated, very loose	
12.7	80.9	SAND: Grey SAND, some silt, some gravel, saturated, very loose	
12.9	80.7	GRANITE BEDROCK: Light grey to dark grey, very poor to fair quality	
17.7	75.9	Borehole terminated at 17.7 m below ground surface in GRANITE BEDROCK.	

SAMPLE TYPE	N VALUE	% WATER	% RECOVERY	RQD (%)	REMARKS
SS1	20	16	75		
SS2	3	29	83		
SS3	2	28	100		
ST1					ST1: Cc = 0.25
VT1					VT1: Cu = >74 kPa
SS4	5	27	100		
SS5	4	30	58		
SS6	3	36	50		
SS7	1	35	100		AL SS7: Liquid Limit: 31% Plastic Limit: 16% Plasticity Index: 15
SS8	2	27	100		
SS9	2	25	100		
SS10	3	22	100		
SS11	3	22	100		
RC1			72	22	
RC2			100	19	
RC3			87	39	
RC4			100	67	
RC5			100	65	
RC6			100	60	



BOREHOLE NO. BH19-04

PROJECT NAME: LOWER BREWERS SWING BRIDGE

PROJECT NO.: 19M-01599-00

CLIENT: PARKS CANADA

DATE COMPLETED: Nov 19, 2019

BOREHOLE TYPE: 82 mm DIRECT PUSH

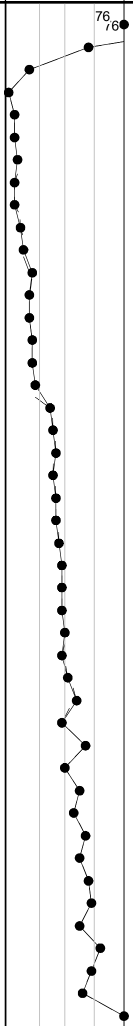
SUPERVISOR: MN

GROUND ELEVATION: 94.2 m

REVIEWER: GB

DEPTH (m)	ELEV (mASL)	STRATIGRAPHIC DESCRIPTION	STRATIGRAPHY	MONITOR DETAILS	SAMPLE TYPE	SAMPLE N VALUE	SAMPLE % WATER	SAMPLE % RECOVERY	SAMPLE RQD (%)	CONE PENETRATION / "N" VALUE 10 20 30 / SHEAR STRENGTH 20 40 60 80 Intact (MaX) Cu, Remoulded Cu	WATER CONTENT % 10 20 30 Wp WL	REMARKS
0.0	94.2	DCPT (Dynamic Cone Penetration Test)										
13.7	80.5	DCPT terminated at effective refusal to further advancement (>100 blows per 0.3 m)										

Depth scale: 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0



APPENDIX

B

LABORATORY DATA

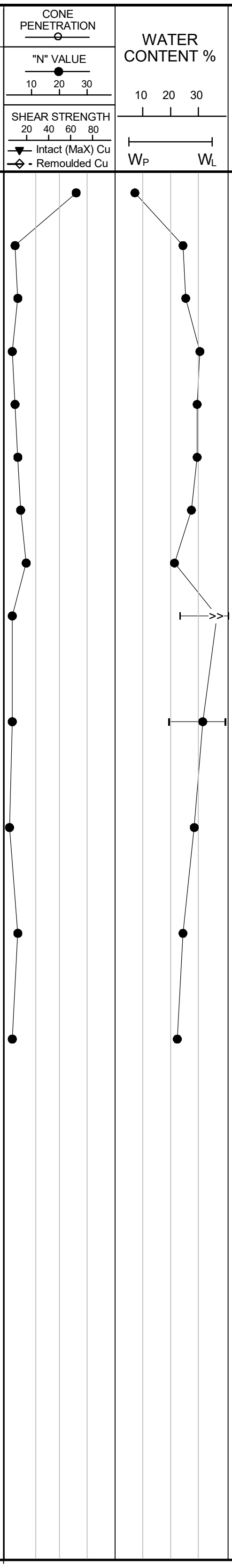
BOREHOLE NO. BH19-05

PAGE 1 of 1

PROJECT NAME: LOWER BREWERS SWING BRIDGE
PROJECT NO.: 19M-01599-00
CLIENT: PARKS CANADA
DATE COMPLETED: Nov 20, 2019
BOREHOLE TYPE: 82 mm DIRECT PUSH
SUPERVISOR: MN
GROUND ELEVATION: 94.2 m
REVIEWER: GB

DEPTH (m)	ELEV (mASL)	STRATIGRAPHIC DESCRIPTION	STRATIGRAPHY	MONITOR DETAILS
0.0	94.2			
0.1	94.1	ASPHALT (100mm)		
		FILL: Light brown gravelly sand FILL, moist (frozen), loose		
0.8	93.4	FILL: Light brown sandy silt FILL, trace gravel, moist, loose		
1.3	92.9	CLAYEY SILT: Light brown CLAYEY SILT, trace sand, DTPL, firm to soft - Grey		
3.4	90.8	SILTY CLAY: Grey SILTY CLAY, trace sand, APL, firm - Light brown to grey		
5.6	88.6	SANDY SILT: Light brown SANDY SILT, some clay, APL		
5.9	88.3	SILTY CLAY: Grey SILTY CLAY TO CLAY, some silt, trace sand, APL, soft		
9.1	85.1	CLAYEY SILT: Grey CLAYEY SILT, some sand, saturated, soft to firm		
11.0	83.2	SILTY SAND: Grey SILTY SAND, saturated, very loose		
13.6	80.6	Borehole terminated upon refusal at 13.6 m below ground surface on presumed BEDROCK.		

SAMPLE TYPE	N VALUE	% WATER	% RECOVERY	RQD (%)
SS1	26	7	71	
SS2	4	24	79	
SS3	5	25	88	
SS4	3	30	79	
SS5	4	29	100	
SS6	5	29	100	
SS7	6	27	100	
SS8	8	21	100	
SS9	3	36	100	
SS10	3	31	100	
SS11	2	28	100	
SS12	5	24	100	
SS13	3	22	100	



REMARKS

Groundwater at 2.1 m below ground surface in open borehole upon completion of drilling.

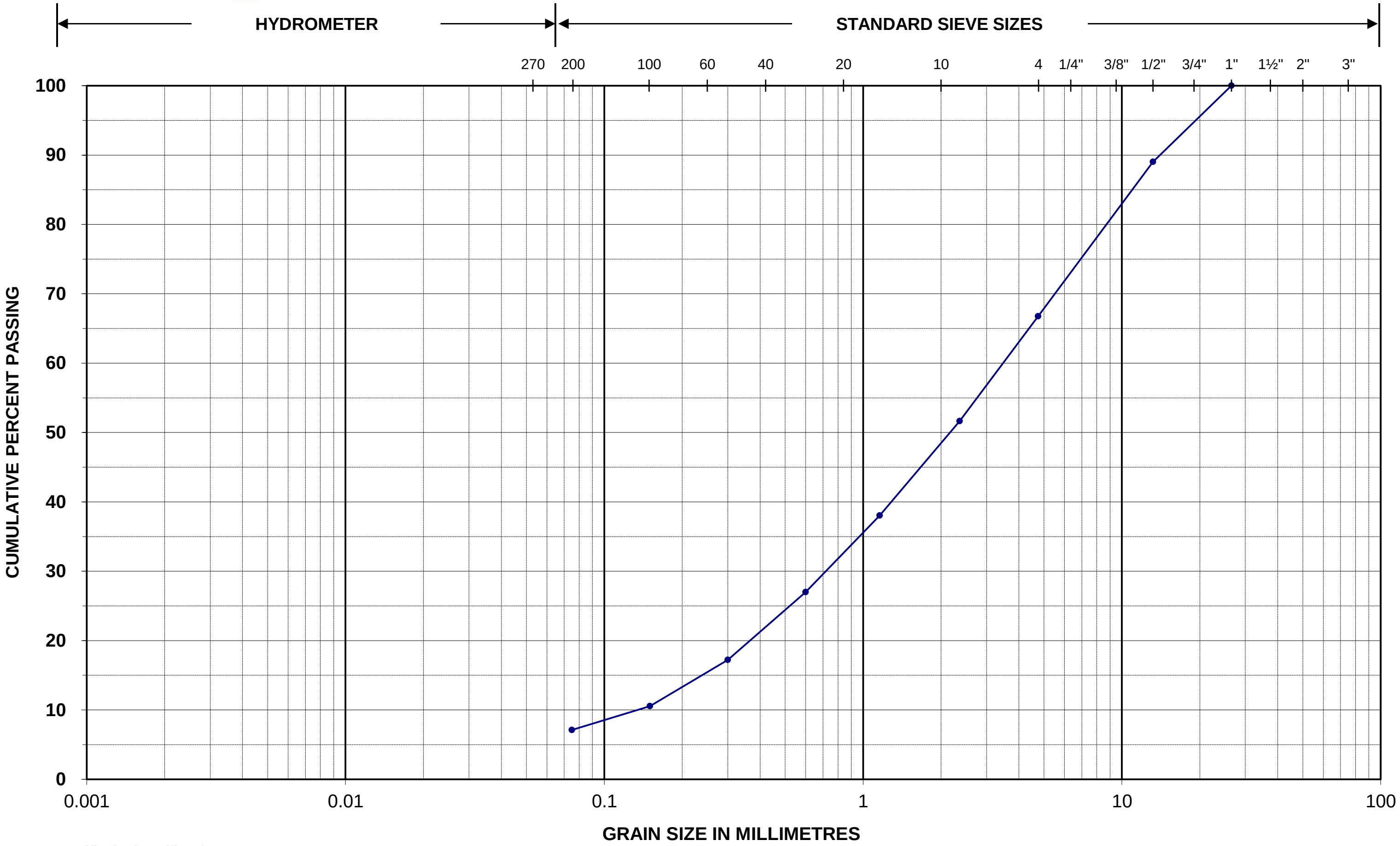
GSA SS8:
Sand: 10%
Silt & Clay: 90%

AL SS9:
Liquid Limit: 53%
Plastic Limit: 23%
Plasticity Index: 30

GSA SS9:
Sand: 3%
Silt & Clay: 97%

AL SS10:
Liquid Limit: 39%
Plastic Limit: 19%
Plasticity Index: 20

PARTICLE SIZE DISTRIBUTION



Unified Classification System

SILT AND CLAY	SAND	GRAVEL

Project Name:	Lower Brewers Swing Bridge	**Project No.:**	19M-01599-00
Location ID.:	BH19-01	**Sample No./Depth:**	CS1A

Sieve Size	% Passing Coarse	**Sieve Size**	% Passing Fine
37.5 mm	100.0	**1.16 mm**	38.0
26.5 mm	100.0	**0.60 mm**	27.0
13.2 mm	89.0	**0.30 mm**	17.2
4.75 mm	66.8	**0.15 mm**	10.6
2.36 mm	51.6	**0.075 mm**	7.1

ATTERBERG LIMITS

ASTM D4318

Date:	29-Nov-19	Job No.:	19M-01599-00
Project Name:	Lower Brewers Swing Bridge	Tech.:	NLO
Borehole/Sample No.:	BH19-03 / SS7		

Liquid Limit Test

Number of Shocks	33	25	15
Tin No.	SK91	SK74	26
Tin + Wet soil	21.5	28.7	29.4
Tin + Dry soil	19.8	26.9	27.4
Wt. of Water	1.7	1.8	2.0
Wt. of Tin	14.0	21.2	21.5
Wt. of Dry Soil	5.8	5.7	5.9
Water Content	30	31	33

Plastic Limit Test

Tin No.	SK50	SK23
Tin + Wet soil	38.9	30.2
Tin + Dry soil	36.4	27.9
Wt. of Water	2.5	2.3
Wt. of Tin	21.4	13.9
Wt. of Dry Soil	15.0	14.1
Water Content	16	16

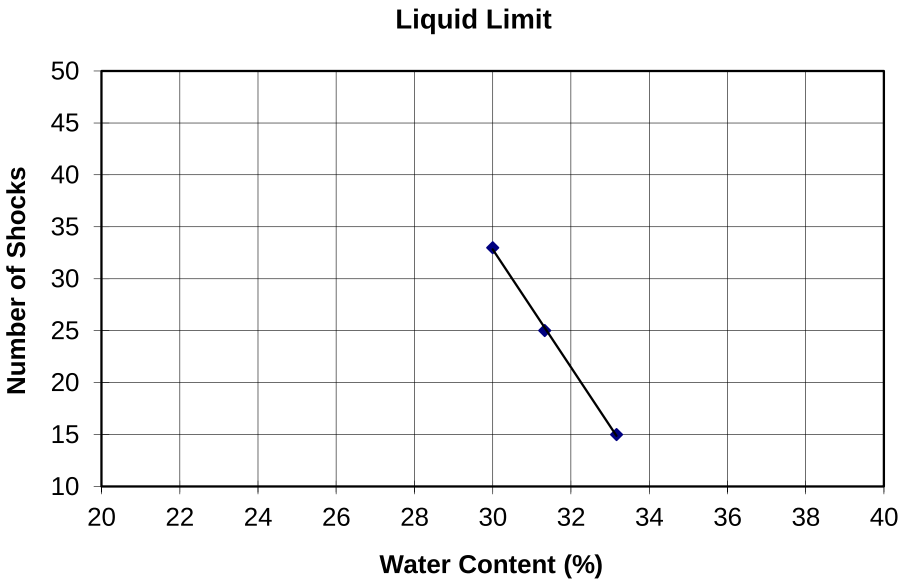
Natural Water Content

P2
93.2
73.3
19.9
15.9
57.4
34.69

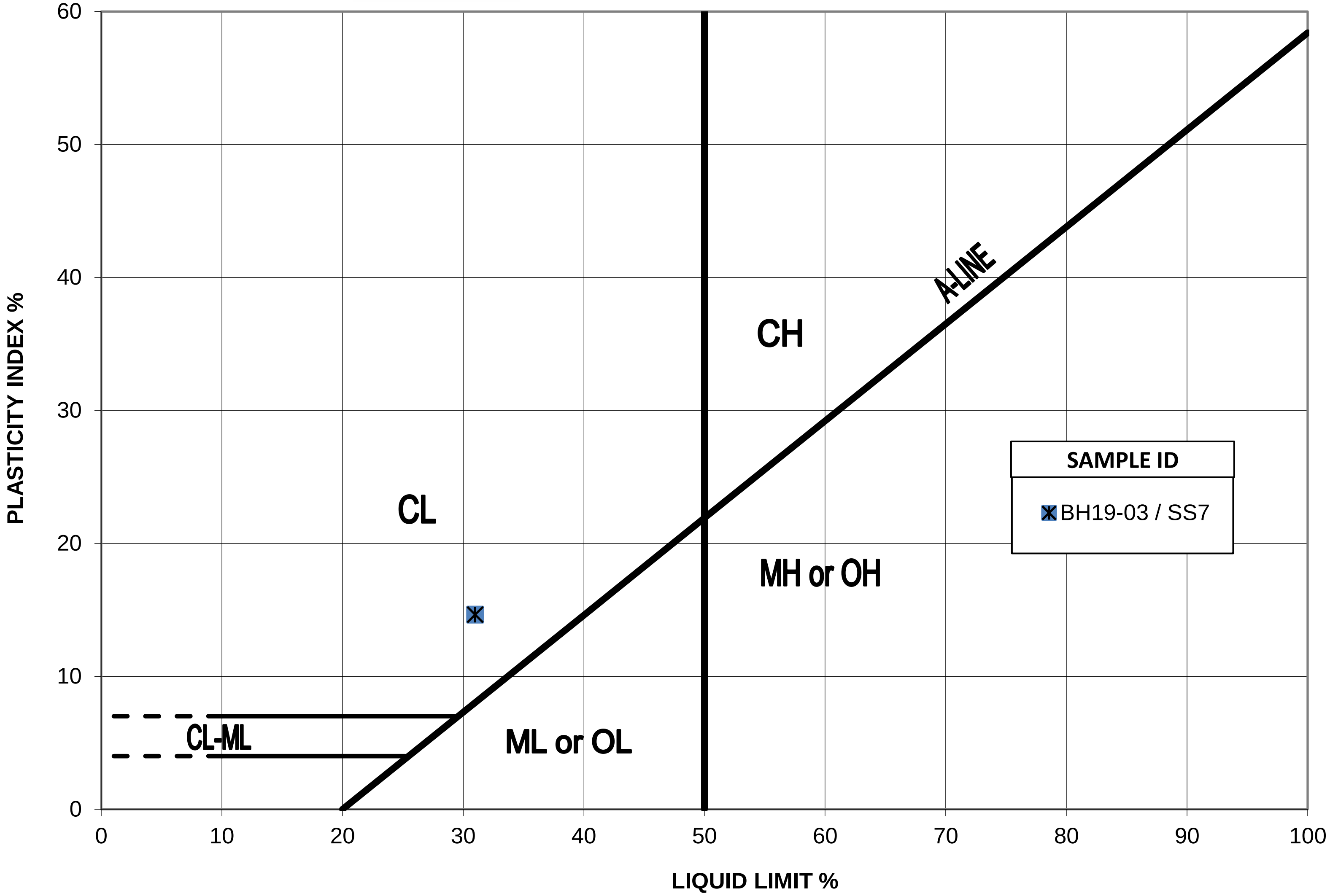
Liquid Limit, (W_L)	31
Plastic Limit, (WP)	16
Plasticity Index (I_P=W_L-W_P)	15
Natural Water Content, W	35
Liquidity Index (I_L=W-W_P/W_L-W_P)	1

Control Results

Liquid Limit, (W_L)	30
Plastic Limit, (WP)	19
Plasticity Index (I_P=W_L-W_P)	11



Atterberg Limits Plasticity Chart
19M-01599-00



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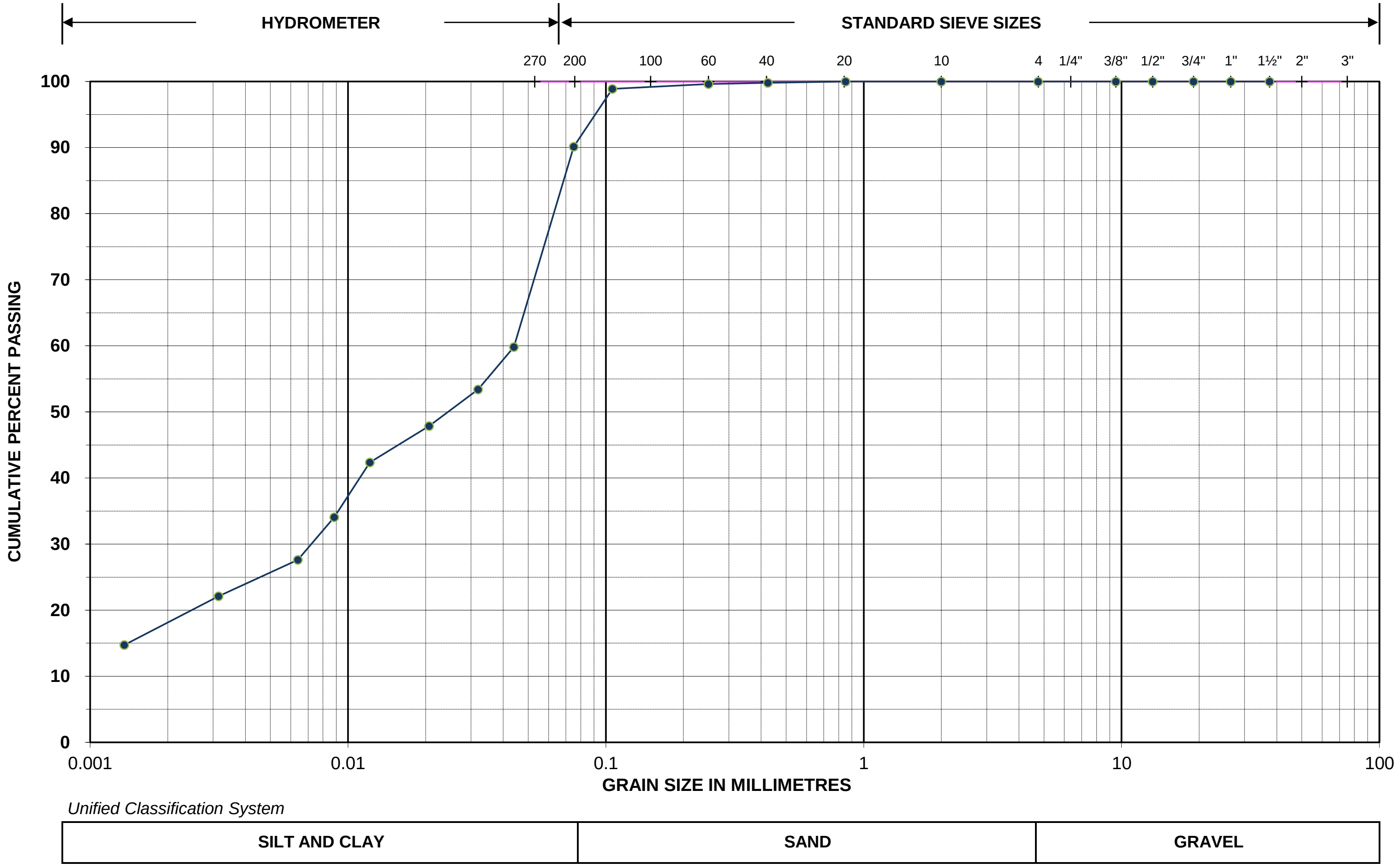
PARTICLE SIZE DISTRIBUTION ASTM D422



Project Name: Lower Brewers Swing Bridge

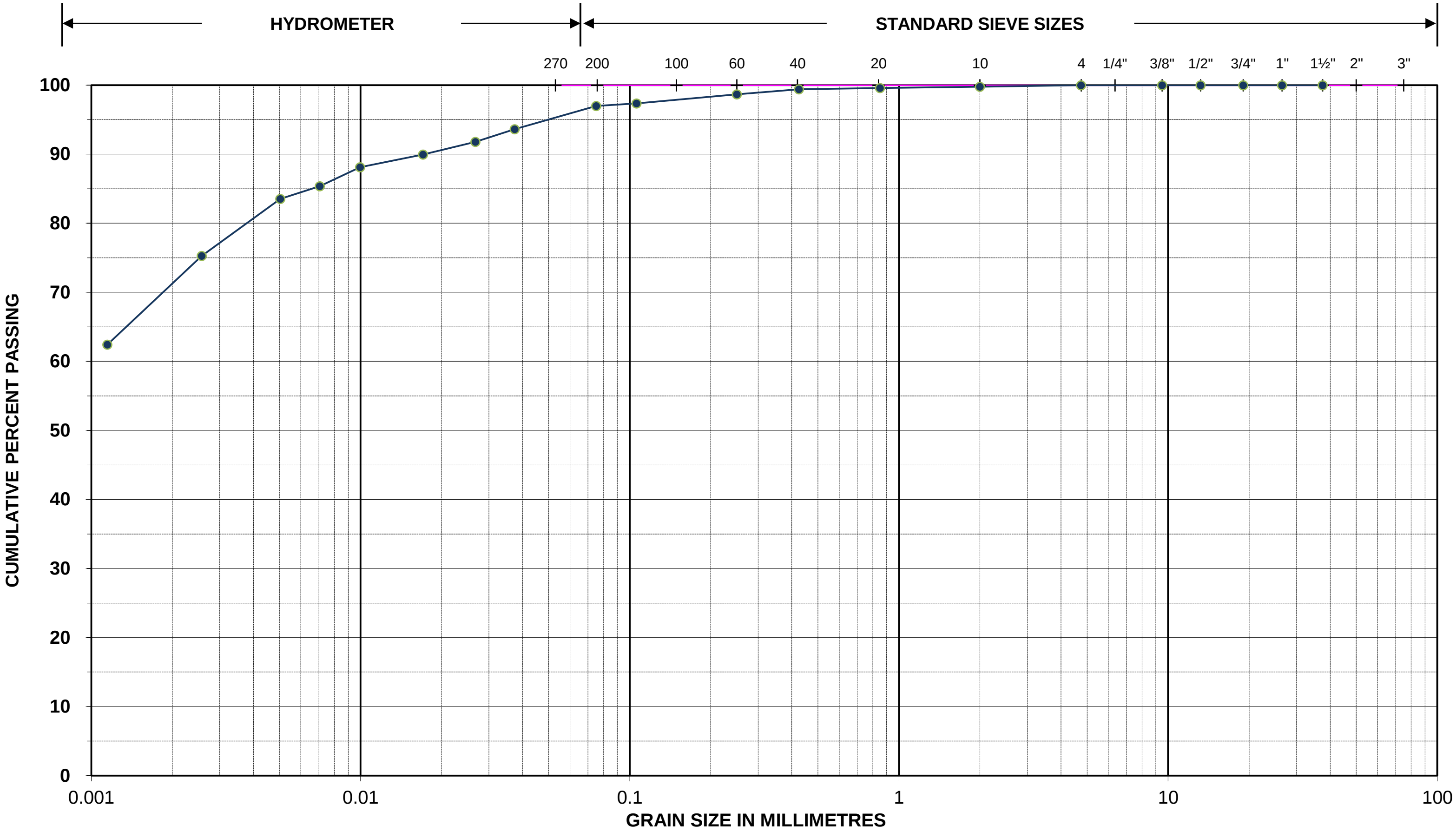
Project No.: 19M-01599-00

Location ID.: BH19-05

Sample No./Depth: SS8

Sieve Size	% Passing Coarse	Sieve Size	% Passing Fine	Hydrometer (mm)	% Passing
37.5 mm	100.0	**2.00 mm**	100.0	**0.044**	59.8
26.5 mm	100.0	**0.850 mm**	100.0	**0.021**	47.9
19.0 mm	100.0	**0.425 mm**	99.8	**0.009**	34.1
13.2 mm	100.0	**0.250 mm**	99.6	**0.003**	22.1
9.50 mm	100.0	**0.106 mm**	98.9	**0.001**	14.7
4.75 mm	100.0	**0.075 mm**	90.1		

PARTICLE SIZE DISTRIBUTION ASTM D422



Unified Classification System

SILT AND CLAY	**SAND**	**GRAVEL**

Project Name: Lower Brewer Swing Bridge

Project No.: 19M-01599-00

Location ID.: BH19-05

Sample No./Depth: SS9

Sieve Size	% Passing Coarse	**Sieve Size**	% Passing Fine	Hydrometer (mm)	% Passing
37.5 mm	100.0	**2.00 mm**	99.8	**0.037**	93.6
26.5 mm	100.0	**0.850 mm**	99.6	**0.017**	90.0
19.0 mm	100.0	**0.425 mm**	99.4	**0.007**	85.4
13.2 mm	100.0	**0.250 mm**	98.7	**0.003**	75.3
9.50 mm	100.0	**0.106 mm**	97.4	**0.001**	62.4
4.75 mm	100.0	**0.075 mm**	97.0		

ATTERBERG LIMITS ASTM D4318

Date:	29-Nov-19	Job No.:	19M-01599-00
Project Name:	Lower Brewers Swing Bridge	Tech.:	NLO
Borehole/Sample No.:	BH19-05 / SS9		

Liquid Limit Test

Number of Shocks	35	26	15
Tin No.	SK19	34	13-10A
Tin + Wet soil	21.9	28.9	34.8
Tin + Dry soil	19.3	26.2	32.5
Wt. of Water	2.6	2.6	2.3
Wt. of Tin	14.3	21.3	28.4
Wt. of Dry Soil	5.1	4.9	4.1
Water Content	51	53	55

Plastic Limit Test

Tin No.	KC10	Bits
Tin + Wet soil	33.3	42.9
Tin + Dry soil	30.8	40.1
Wt. of Water	2.5	2.8
Wt. of Tin	20.0	28.4
Wt. of Dry Soil	10.9	11.8
Water Content	23	24

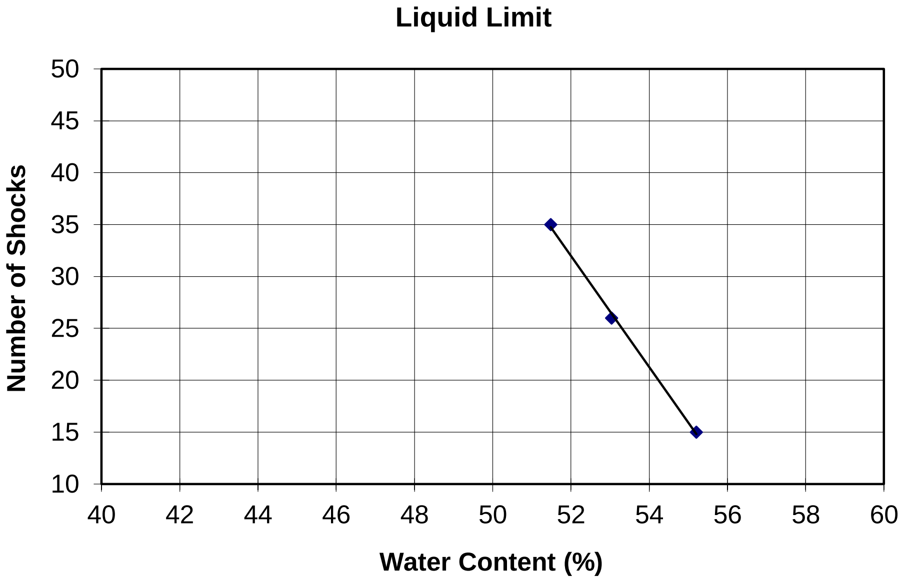
Natural Water Content

6G
107.6
83.5
24.1
16.7
66.8
36.1

Liquid Limit, (W_L)	53
Plastic Limit, (WP)	23
Plasticity Index (I_P=W_L-W_P)	30
Natural Water Content, W	36
Liquidity Index (I_L=W-W_P/W_L-W_P)	0

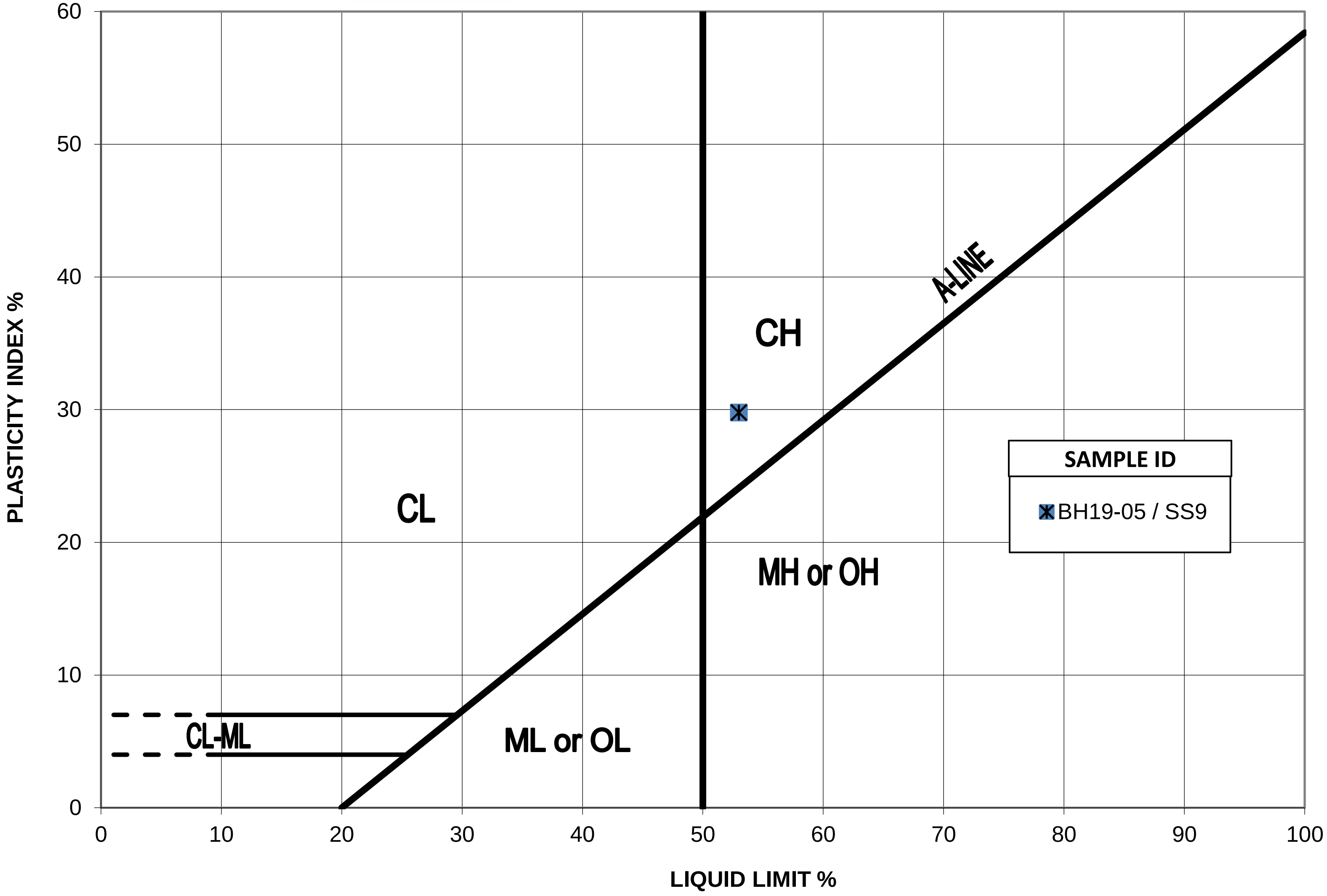
Control Results

Liquid Limit, (W_L)	30
Plastic Limit, (WP)	19
Plasticity Index (I_P=W_L-W_P)	11



Atterberg Limits Plasticity Chart
19M-01599-00

CERTIFIED BY CCIL Canadian Council of Independent Laboratories For specific tests as listed on www.ccil.com



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ATTERBERG LIMITS

ASTM D4318

Date:	29-Nov-19	Job No.:	19M-01599-00
Project Name:	Lower Brewers Swing Bridge	Tech.:	NLO
Borehole/Sample No.:	BH19-05 / SS10		

Liquid Limit Test

Number of Shocks	35	25	15
Tin No.	16	JLK	CJ3
Tin + Wet soil	34.9	26.5	27.5
Tin + Dry soil	33.1	24.6	25.2
Wt. of Water	1.8	2.0	2.3
Wt. of Tin	28.3	19.6	19.8
Wt. of Dry Soil	4.8	4.9	5.4
Water Content	37	39	42

Plastic Limit Test

Tin No.	B3	RK2
Tin + Wet soil	41.3	43.2
Tin + Dry soil	39.3	40.8
Wt. of Water	2.0	2.4
Wt. of Tin	28.5	28.0
Wt. of Dry Soil	10.8	12.7
Water Content	18	19

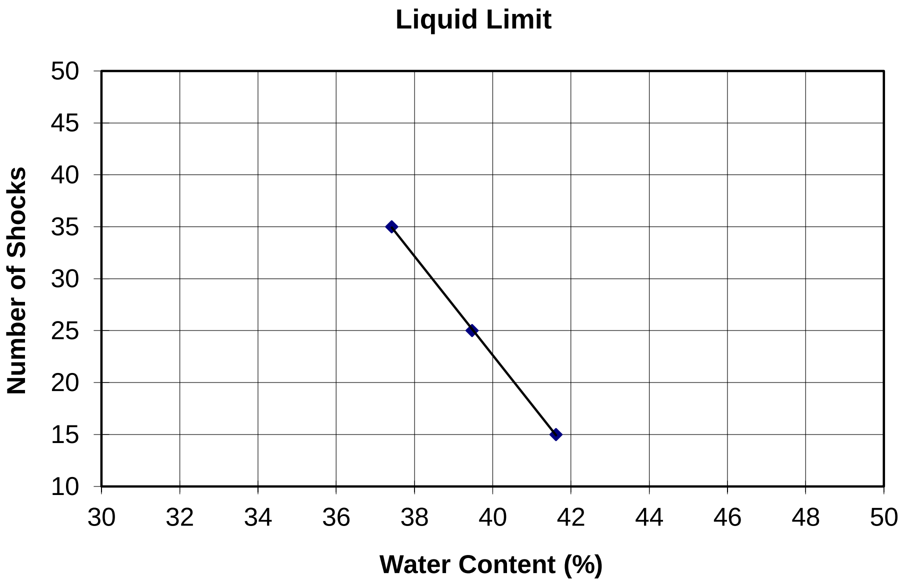
Natural Water Content

KR12
95.0
76.4
18.6
15.7
60.7
30.6

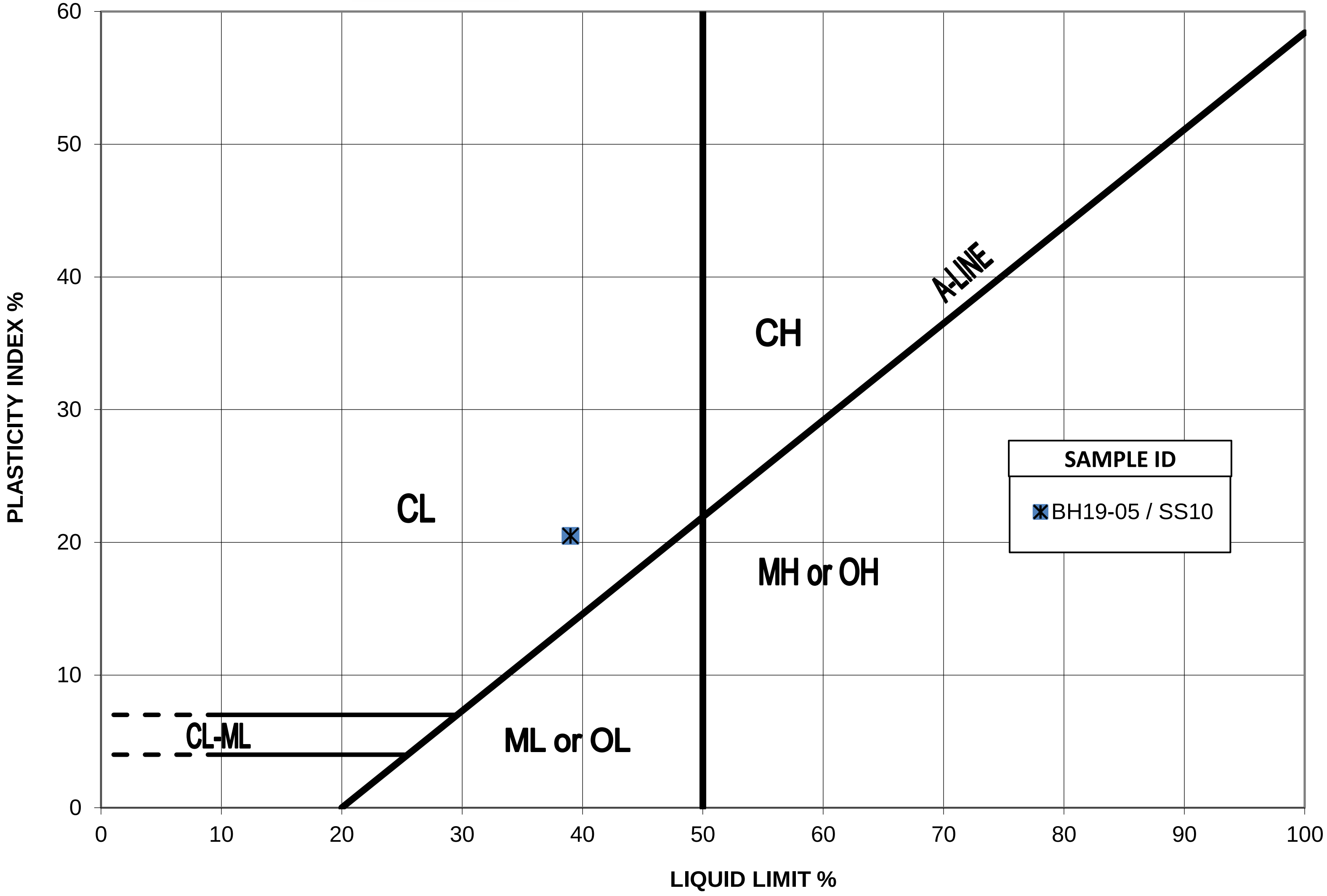
Liquid Limit, (W_L)	39
Plastic Limit, (WP)	19
Plasticity Index (I_P=W_L-W_P)	20
Natural Water Content, W	31
Liquidity Index (I_L=W-W_P/W_L-W_P)	1

Control Results

Liquid Limit, (W_L)	30
Plastic Limit, (WP)	19
Plasticity Index (I_P=W_L-W_P)	11



Atterberg Limits Plasticity Chart
19M-01599-00



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252 GALAXY BLVD
TORONTO, ON M9W 5R8
T: +1 416 798-0065 F: +1 416 798-0518

ONE DIMENSIONAL CONSOLIDATION TEST
(D2435/D2435M - 11)

Client:	PARKS CANADA	**Job #:**	19M-01599-00
Project:	Lower Brewers Swing Bridge	**Location:**	N/A
BH #:	19-03	**Start Date:**	26-Jan-2020
Samp#:	ST1	**End Date:**	7-Feb-2020
Depth:	N/A	**Pre|App by:**	HR

Apparatus Data		**Moisture Content Data**	**Initial**	**Final**
Ring Height (mm)	19.93	Wt. of Ring + Soil + Water (gm)	232.3	229.71
Ring Int. Diameter (mm)	63.46	Wt. of Tare (gm)		66.29
Ring Internal Area (mm^2)	3162.93	Wt. of Tare + Dry Soil (gm)		166.27
Ring Weight (gm)	107.6	Moisture Content (%)	24.72	22.13

Sample Data	**Initial**	**Final**
Sample Height (mm)	19.93	17.787
Sample Volume (mm^3)	63037.26	56259.09
Void Ratio	0.704	0.520
Saturation (%)	94.95	114.92
Wet Density (kg/m^3)	1978.20	2170.49
Dry Density (kg/m^3)	1586.05	1777.13

Index Tests	
Specific Gravity	2.702
Liquid Limit (%)	
Plastic Limit (%)	

Soil Description

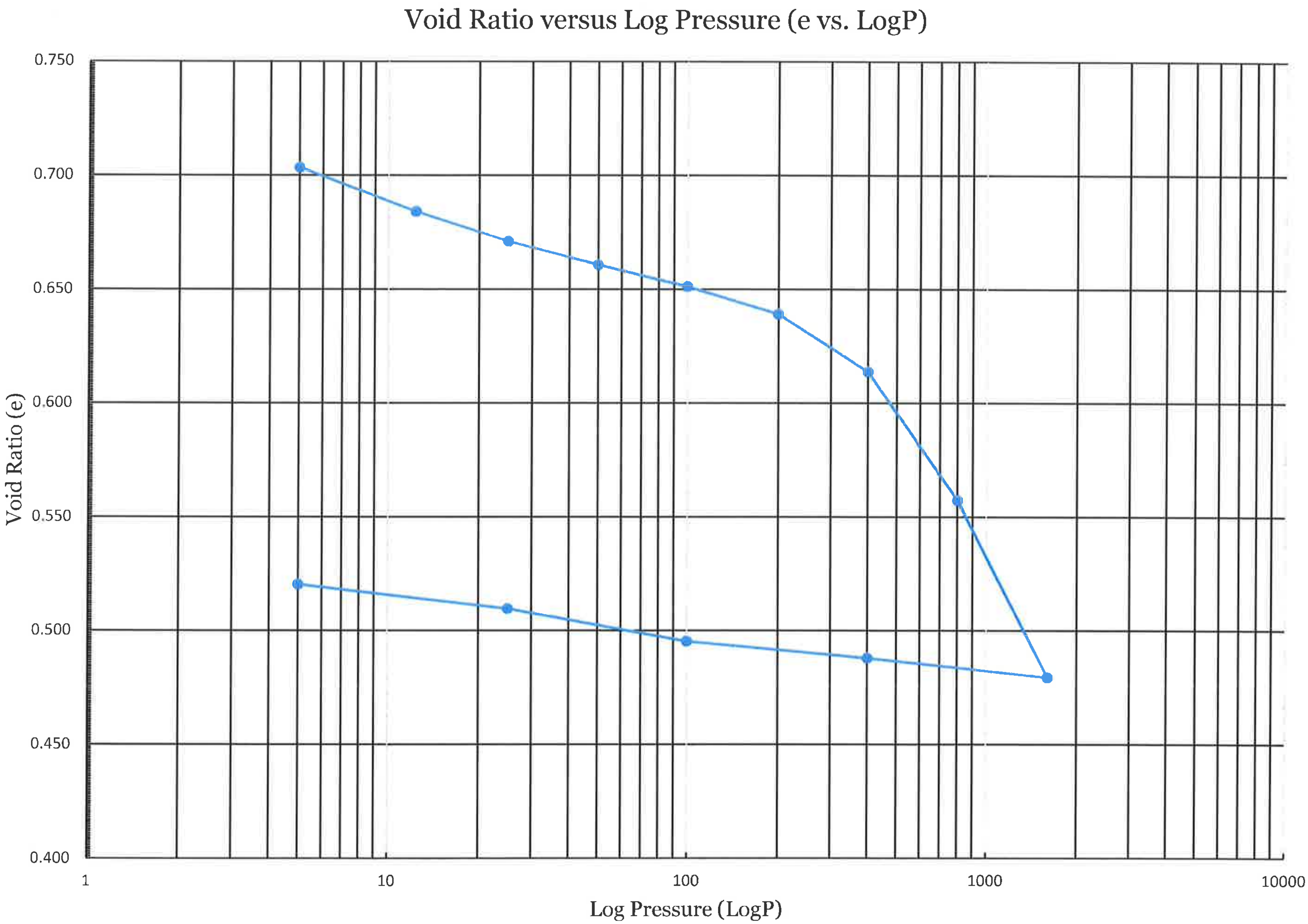
Silty Clay: stiff, sandy, brownish, low plastic (CL)

Test Procedure

Sample was placed in a consolidometer ring by trimming using a cutting shoe and wire saw. The ring was then placed in consolidometer and flooded with water immediately after putting a seating load of 5 kPa. The test was then started quickly, after flooding, with the first load of 12 kPa. The sample began to consolidate at the application of 12 kPa. Each loading stage was applied for time interval of 24 hours. The coefficient of consolidation was determined using Square Root of Time method.

Pressure (kPa)	ΔH_C (mm)	H_C (mm)	Axial Strain (%)	Void Ratio	m_v (m^2/KN)	t_{90} (min)	HD (mm)	$HD_{,50}$ (mm)	C_v (mm^2/s)	C_v (m^2/year)	K (m/s)
5	0.0010	19.9290	0.01	0.704							
12	0.2270	19.7030	1.14	0.684	1.55E-03	1.00	19.7030	9.8515	1.37E+00	4.33E+01	2.08E-08
25	0.3810	19.5490	1.91	0.671	6.11E-04	1.21	19.5490	9.7745	1.12E+00	3.52E+01	6.69E-09
50	0.5000	19.4300	2.51	0.661	2.39E-04	2.25	19.4300	9.715	5.93E-01	1.87E+01	1.39E-09
100	0.6120	19.3180	3.07	0.651	1.12E-04	2.56	19.3180	9.659	5.15E-01	1.62E+01	5.68E-10
200	0.7540	19.1760	3.78	0.639	7.13E-05	2.89	19.1760	9.588	4.50E-01	1.42E+01	3.14E-10
400	1.0500	18.8800	5.27	0.614	7.42E-05	3.10	18.8800	9.44	4.06E-01	1.28E+01	2.96E-10
800	1.7110	18.2190	8.59	0.557	8.29E-05	3.61	18.2190	9.1095	3.25E-01	1.02E+01	2.64E-10
1600	2.6200	17.3100	13.15	0.480	5.70E-05	5.29	17.3100	8.655	2.00E-01	6.31E+00	1.12E-10
400	2.5215	17.4085	12.65	0.488							
100	2.4360	17.4940	12.22	0.495							
25	2.2680	17.6620	11.38	0.510							
5	2.1430	17.7870	10.75	0.520							

Completed By: Harun Rashid


Void Ratio versus Log Pressure (e vs. LogP)
Void Ratio (e)
0.750
0.700
0.650
0.600
0.550
0.500
0.450
0.400
1
10
100
1000
10000
Log Pressure (LogP)

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN
ASTM D 7012

CLIENT: Parks Canada

PROJECT: Lower Brewers Swing Bridge

PROJECT No.: 19M-01599-00

LAB No.: OL993-2

SAMPLE No.: BH19-02

DEPTH: 8'1"-8'9" (2.46-2.67m)

SAMPLING DATE: -

TESTING APPARATUS USED: **Loading device No.:** 1 **Caliper No.:** 1

	Average	
Diameter:	47.2	(mm)
Length:	113.2	(mm)

Mass: 536.4 (g)

Volume: 197912.0 (mm³)

Density: 2710 (kg/m³)

Moisture conditions: as received

Loading rate (0.5 to 1.0 MPa / sec): 1.3 (MPa/sec)

Test duration (2-15 minutes) 1:13 (minutes)

Maximum applied load: 177.56 (kN)

Compressive strength **101.5** (MPa)

REMARKS:

TESTED BY: N.Krebs **DATE:** November 19, 2021

VERIFIED BY: N.Krebs **DATE:** November 19, 2021

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN
ASTM D 7012

CLIENT:	Parks Canada	**LAB No.:**	OL993-1
PROJECT:	Lower Brewers Swing Bridge	**SAMPLE No.:**	BH19-03
PROJECT No.:	19M-01599-00	**DEPTH:**	57'6"-58' (17.53-17.68m)
		SAMPLING DATE:	-

TESTING APPARATUS USED: **Loading device No.:** 1 **Caliper No.:** 1

	Average	
Diameter:	47.4	(mm)
Length:	111.0	(mm)

Mass: 516.2 (g) Volume: 196046.9 (mm³)

Density: 2633 (kg/m^3)

Moisture conditions: as received

Loading rate (0.5 to 1.0 MPa / sec): 1.9 (MPa/sec)

Test duration (2-15 minutes) 0:57 (minutes)

Maximum applied load: 194.2 (kN)

Compressive strength **110.0** (MPa)

REMARKS:

TESTED BY:	N.Krebs	**DATE:**	November 19, 2021
VERIFIED BY:	N.Krebs	**DATE:**	November 19, 2021

APPENDIX

C

CHEMICAL LABORATORY DATA

FINAL REPORT

CA14845-NOV19 R1

19M-01599-00 Kingston, ON

Prepared for

WSP Canada Inc.

FINAL REPORT

First Page

CLIENT DETAILS

Client	WSP Canada Inc.
Address	294 Rink St. Peterborough, ON K9J 2K2. Canada
Contact	Garnet Brenchley
Telephone	705.761-0128
Facsimile	
Email	garnet.brenchley@wsp.com
Project	19M-01599-00 Kingston, ON
Order Number	
Samples	Soil (1)

LABORATORY DETAILS

Project Specialist	Brad Moore Hon. B.Sc
Laboratory	SGS Canada Inc.
Address	185 Concession St., Lakefield ON, K0L 2H0
Telephone	705-652-2143
Facsimile	705-652-6365
Email	brad.moore@sgs.com
SGS Reference	CA14845-NOV19
Received	11/21/2019
Approved	11/29/2019
Report Number	CA14845-NOV19 R1
Date Reported	11/29/2019

COMMENTS

Temperature of Sample upon Receipt: 14 degrees C
Cooling Agent Present:Yes
Custody Seal Present:No

Chain of Custody Number:NA

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Brad Moore Hon. B.Sc

TABLE OF CONTENTS

PACKAGE: **- Corrosivity Index** (SOIL)

Sample Number 5
Sample Name BH19-03-Composite
Sample Matrix Soil
Sample Date 20/11/2019

Parameter	Units	RL		Result
Corrosivity Index				
Corrosivity Index	none	1		4
Soil Redox Potential	mV	-		158
Sulphide	%	0.02		< 0.02
pH	pH Units	0.05		8.67
Resistivity (calculated)	ohms.cm	-9999		8750

PACKAGE: **- General Chemistry** (SOIL)

Sample Number 5
Sample Name BH19-03-Composite
Sample Matrix Soil
Sample Date 20/11/2019

Parameter	Units	RL		Result
General Chemistry				
Conductivity	uS/cm	2		114

PACKAGE: **- Metals and Inorganics** (SOIL)

Sample Number 5
Sample Name BH19-03-Composite
Sample Matrix Soil
Sample Date 20/11/2019

Parameter	Units	RL		Result
Metals and Inorganics				
Moisture Content	%	0.1		22.7

PACKAGE: **- Metals and Inorganics** (SOIL)

Sample Number 5
Sample Name BH19-03-Composite
Sample Matrix Soil
Sample Date 20/11/2019

Parameter	Units	RL		Result
Metals and Inorganics (continued)				
Sulphate	µg/g	0.4		20

PACKAGE: **- Other (ORP)** (SOIL)

Sample Number 5
Sample Name BH19-03-Composite
Sample Matrix Soil
Sample Date 20/11/2019

Parameter	Units	RL		Result
Other (ORP)				
Chloride	µg/g	0.4		8.2

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0444-NOV19	µg/g	0.4	<0.4	9	20	93	80	120	96	75	125
Sulphate	DIO0444-NOV19	µg/g	0.4	<0.4	5	20	98	80	120	98	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	ECS0038-NOV19	%	0.02	<0.02	2	20	112	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0375-NOV19	uS/cm	2	< 0.002	0	10	98	90	110	NA		

FINAL REPORT

QC SUMMARY

pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0375-NOV19	pH Units	0.05	NA	1		100			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

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

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates 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.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS	Insufficient sample for analysis.
RL	Reporting Limit.
↑	Reporting limit raised.
↓	Reporting limit lowered.
NA	The sample was not analysed for this analyte
ND	Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated. This document is issued, on the Client's behalf, by the Company under its General Conditions of Service available on request and accessible at http://www.sgs.com/terms_and_conditions.htm. The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents.

This report must not be reproduced, except in full. This report supersedes all previous versions.

-- End of Analytical Report --

SGS

SGS Environment,
Health and Safety

Request for Laboratory Services and CHAIN OF CUSTODY

- Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Toll Free: 877-747-7658 Fax: 705-652-6365
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361 Web: www.ca.sgs.com

No:
Page 1 of 1

Laboratory Information Section - Lab use only

Received By: Mack Dopplis
Received Date: NOV 2 1 2019 (mm\dd\yy)
Received Time: 16 : 40 am / pm (circle)

Received By (signature): MKay
Custody Seal Present: Y [✓ N] (circle)
Custody Seal Intact: Y [✓ N]

Cooling Agent Present: [✓ Y] N Type: Icepack
Temperature Upon Receipt (°C) 14x3
LAB LIMS #: CA-14825 NOV19

REPORT INFORMATION

Company: WSP Canada Inc.
Contact: Garnet Brenchley
Address: 294 Rink Street Suite 103
Peterborough, ON K9J 2K2
Phone: 705-270-0168
Fax:
Email: Garnet.Brenchley@wsp.com

INVOICE INFORMATION

☒ (same as Report Information)
Company:
Contact:
Address:
Phone:
Email:

PROJECT INFORMATION

Quotation #: P.O. #:
Project #: 19M-01599-00 Site Location/ID: Kingston, ON

TURNAROUND TIME (TAT) REQUIRED

☐ Regular TAT (5-7days)
TAT's are quoted in business days (exclude statutory holidays & weekends).
Samples received after 3pm or on weekends : TAT begins the next business day

RUSH TAT (Additional Charges May Apply) ☐ 1 Day ☐ 2 Days ☒ 3-4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date: Rush Confirmation ID:

DRINKING WATER SAMPLES (POTABLE WATER FOR HUMAN CONSUMPTION) MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

REGULATIONS

Regulation 153 (2011):
☒ Table 1 ☒ Res/Park Soil Texture:
☐ Table 2 ☐ Ind/Com ☐ Coarse
☐ Table 3 ☐ Agri/Other ☐ Medium
☐ Table ______ ☒ Fine

Other Regulations:
☐ Reg 347/558 (3 Day min TAT)
☐ PWQO ☐ MMER
☐ CCME ☐ Other:
☐ MISA

Sewer By-Law:
☐ Sanitary
☐ Storm
Municipality:

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	Corrosivity Pkg.											COMMENTS: Field Filtered (F) Preserved (P)
1	BH19-03 - Composite	Nov.20/19		2	S	x											
2																	
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

Observations/Comments/Special Instructions

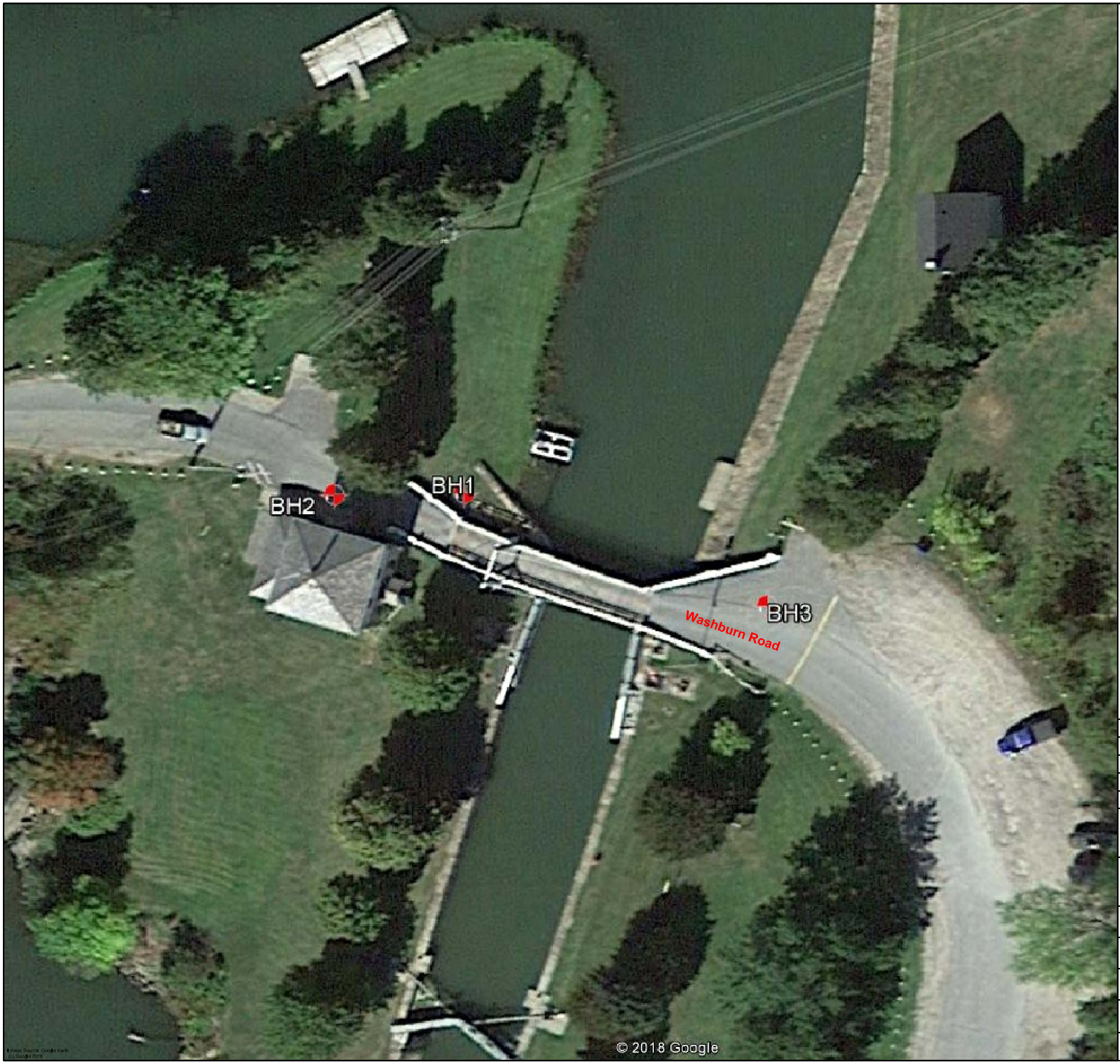
Sampled By (NAME): Mike Nieukirk Signature: Date: 11/21/19 (mm/dd/yy) Pink Copy - Client
Relinquished by (NAME): Whitney Hilderbrandt Signature: Date: 11/21/19 (mm/dd/yy) Yellow & White Copy - SGS

Revision #: 1.1
Date of Issue: 25 July, 2016

APPENDIX

D

BOREHOLE PLAN AND LOGS (FROM SNC LAVALIN GEM ONTARIO INC. REPORT DATED NOVEMBER 15, 2018)



LEGEND

BH1 ⌖ - Approximate borehole location

NOTES

1. All site features are approximate.

2. Drawing should be viewed in conjunction with report No.: 657932.

NO.	DESCRIPTION	DATE

CLIENT:
SNC-Lavalin Infrastructure
Engineering - Eastern Canada

PROJECT:
Lower Brewers Swing Bridge
Rehabilitation

LOCATION:
Washburn Rd, 250m west of HW 15 , Ontario

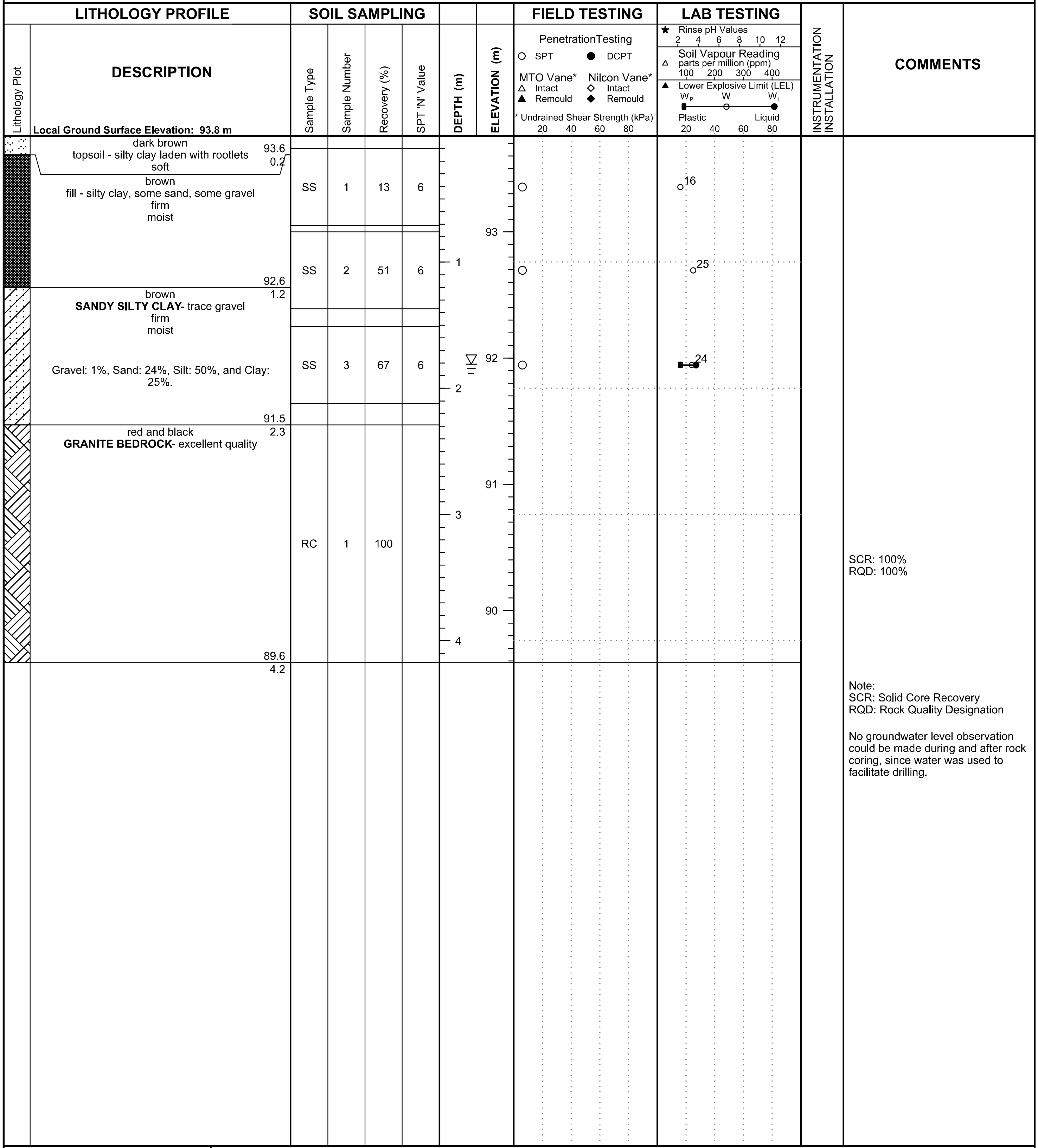
TITLE:
Borehole Location Plan

SCALE:
NTS

DATE:	FILE:	DIV.	DRAWING:
October 2018	657932	00	1

RECORD OF BOREHOLE No. BH1 Co-Ord. 18 N 4915994 394460

Project Number: **657932**
Drilling Location: **Lower Brewers Bridge**
Logged by: **MM**
Project Client: **SNC Lavalin**
Drilling Method: **100 mm Solid Stem Augers**
Compiled by: **MM**
Project Name: **Lower Brewers and Brass Point Swing Bridges**
Drilling Machine: **Truck Mounted Drill**
Reviewed by: RG
Project Location: **Kingston, ON**
Date Started: **Aug 27, 18** Date Completed: **Aug 27, 18**
Revision No.: **0**



DESCRIPTION (Local Ground Surface Elevation: 93.8 m)	Elevation / Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value	Lab Testing	COMMENTS
dark brown topsoil - silty clay laden with rootlets soft	93.6 0.2						
brown fill - silty clay, some sand, some gravel firm moist		SS	1	13	6	16	
	92.6 1.2	SS	2	51	6	25	
brown **SANDY SILTY CLAY**- trace gravel firm moist Gravel: 1%, Sand: 24%, Silt: 50%, and Clay: 25%.	91.5 2.3	SS	3	67	6	24	
red and black **GRANITE BEDROCK**- excellent quality	89.6 4.2	RC	1	100			SCR: 100% RQD: 100%

Note:
SCR: Solid Core Recovery
RQD: Rock Quality Designation

No groundwater level observation could be made during and after rock coring, since water was used to facilitate drilling.

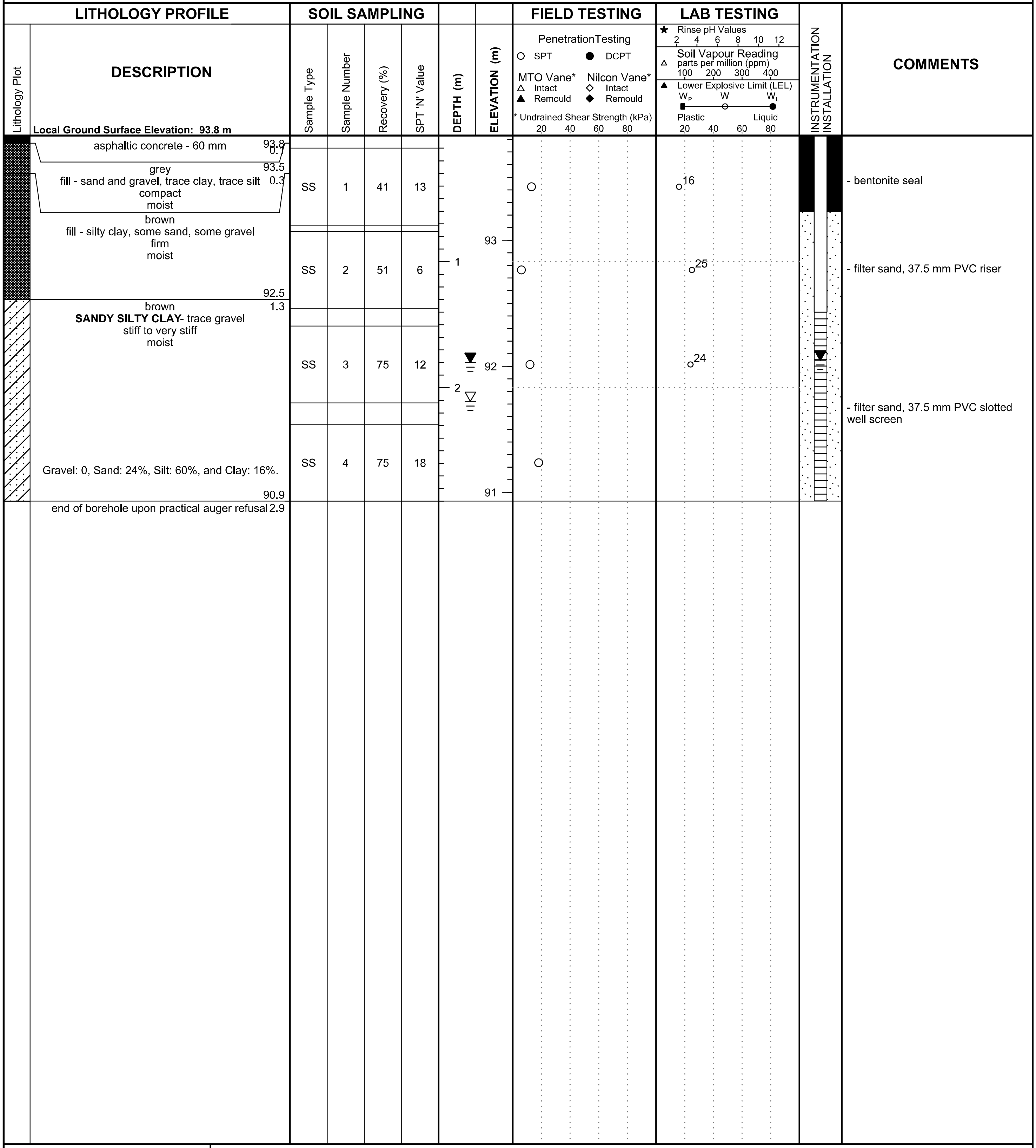
▽ Groundwater depth on completion of drilling: 1.8 m.

SNC·LAVALIN
1164 Clyde Court
Kingston, ON K7P 0G5
Tel: 613-389-1781
Fax: 613-389-4204

Scale: 1 : 42

RECORD OF BOREHOLE No. BH2 Co-Ord. 18 N 4915994 394447

Project Number: 657932
Project Client: SNC Lavalin
Project Name: Lower Brewers and Brass Point Swing Bridges
Project Location: Kingston, ON
Drilling Location: Lower Brewers Bridge
Drilling Method: 100 mm Solid Stem Augers
Drilling Machine: Truck Mounted Drill
Date Started: Aug 27, 18 Date Completed: Aug 27, 18
Logged by: MM
Compiled by: MM
Reviewed by: RG
Revision No.: 0



Local Ground Surface Elevation: 93.8 m

DESCRIPTION	Elev. / Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value	COMMENTS
asphaltic concrete - 60 mm	93.8 0.1					
grey fill - sand and gravel, trace clay, trace silt compact moist	93.5 0.3	SS	1	41	13	- bentonite seal
brown fill - silty clay, some sand, some gravel firm moist	92.5 1.3	SS	2	51	6	- filter sand, 37.5 mm PVC riser
brown **SANDY SILTY CLAY**- trace gravel stiff to very stiff moist		SS	3	75	12	- filter sand, 37.5 mm PVC slotted well screen
Gravel: 0, Sand: 24%, Silt: 60%, and Clay: 16%.	90.9 2.9	SS	4	75	18	
end of borehole upon practical auger refusal						

SNC · LAVALIN
1164 Clyde Court
Kingston, ON K7P 0G5
Tel: 613-389-1781
Fax: 613-389-4204

▽ Groundwater depth on completion of drilling: 2.1 m.
▼ Groundwater depth observed on September 5 2018 at a depth of: 1.8 m.

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 42

RECORD OF BOREHOLE No. BH3 Co-Ord. 18 N 4915983 394491

Project Number: **657932**

Project Client: **SNC Lavalin**

Project Name: **Lower Brewers and Brass Point Swing Bridges**

Project Location: **Kingston, ON**

Drilling Location: **Lower Brewers Bridge**

Drilling Method: **100 mm Solid Stem Augers**

Drilling Machine: **Truck Mounted Drill**

Date Started: **Aug 27, 18** Date Completed: **Aug 27, 18**

Logged by: **MM**

Compiled by: **MM**

Reviewed by: **RG**

Revision No.: **0**

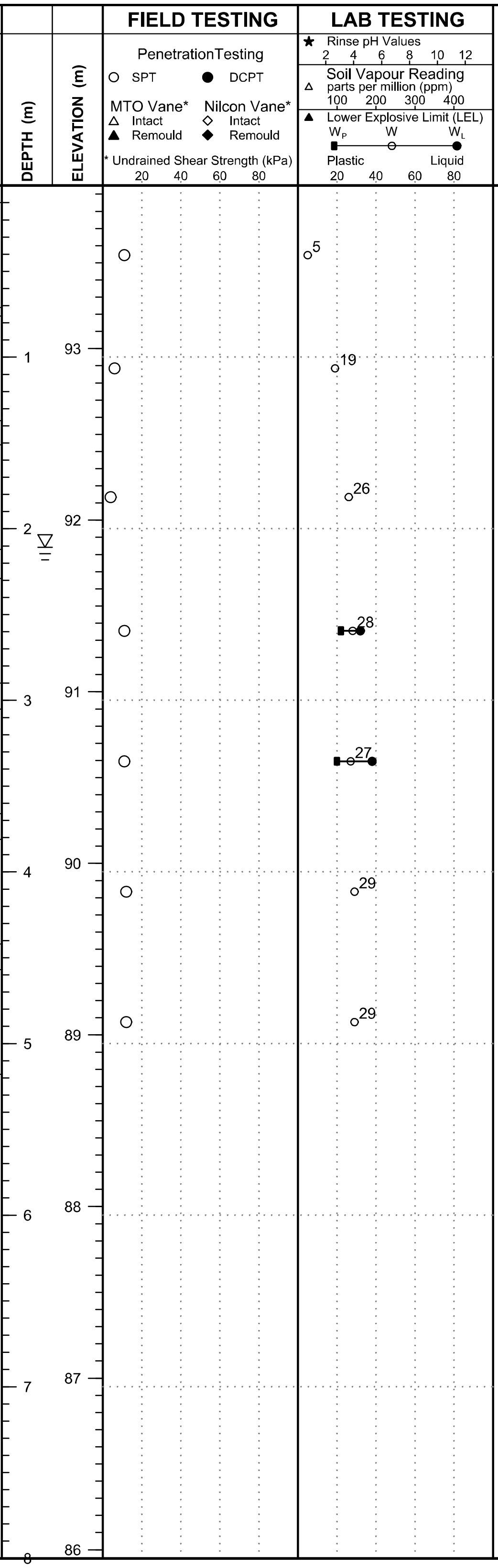
LITHOLOGY PROFILE | **SOIL SAMPLING** | **FIELD TESTING** | **LAB TESTING** | **INSTRUMENTATION INSTALLATION** | **COMMENTS**

Lithology Plot

DESCRIPTION

Local Ground Surface Elevation: 94.0 m

Description	Elevation / Depth (m)	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value
asphaltic concrete - 50 mm	93.9 0.1				
grey fill - sand and gravel, trace silt, trace clay compact moist	93.7 0.3	SS	1	41	11
grey **SANDY SILTY CLAY**- trace gravel firm to stiff moist		SS	2	51	6
		SS	3	54	4
Gravel: 0, Sand: 24%, Silt: 60%, and Clay: 16%.		SS	4	75	11
		SS	5	84	11
		SS	6	67	12
Less sand content Gravel: 0, Sand: 7%, Silt: 40%, and Clay: 53%.		SS	7	75	12
red/black **BOULDERS AND COBBLES** granite Borehole advanced through Boulders and Cobbles by Rock Coring	88.8 5.2				



$\underline{\nabla}$ Groundwater depth on completion of drilling: 2.1 m.

SNC·LAVALIN
1164 Clyde Court
Kingston, ON K7P 0G5
Tel: 613-389-1781
Fax: 613-389-4204

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 42

Continued on Next Page

RECORD OF BOREHOLE No. BH3 Co-Ord. 18 N 4915983 394491

Project Number: 657932 Drilling Location: Lower Brewers Bridge Logged by: MM

LITHOLOGY PROFILE | SOIL SAMPLING | FIELD TESTING | LAB TESTING | INSTRUMENTATION INSTALLATION | COMMENTS

PenetrationTesting: ○ SPT ● DCPT

MTO Vane*: △ Intact ▲ Remould Nilcon Vane*: ◇ Intact ◆ Remould

* Undrained Shear Strength (kPa) 20 40 60 80

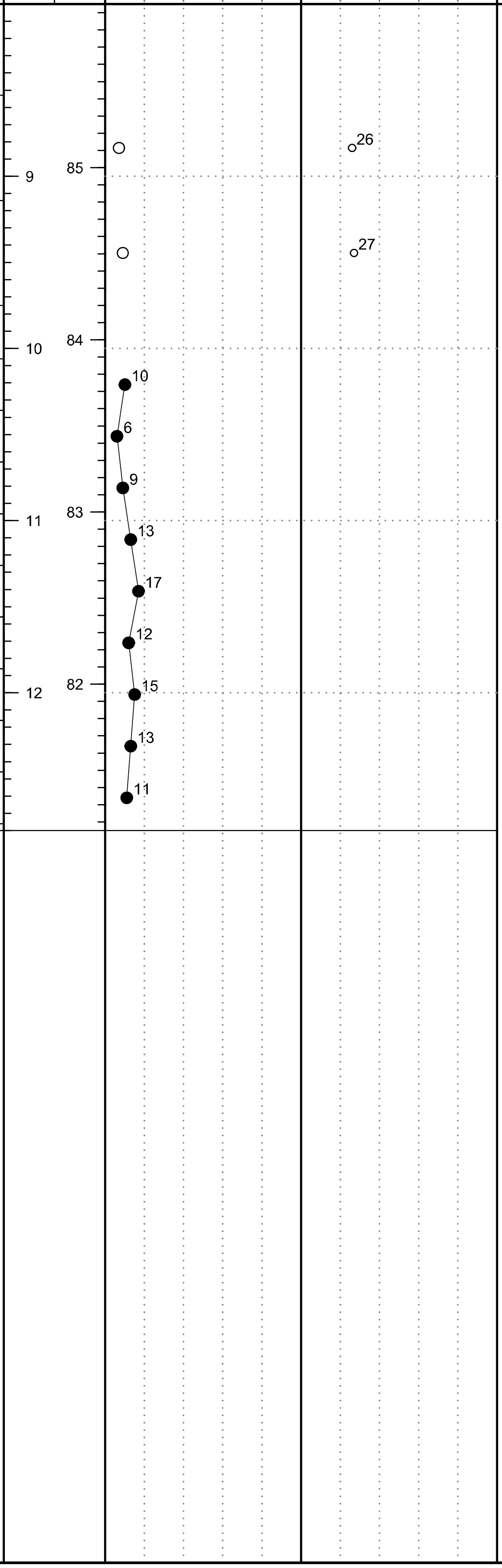
★ Rinse pH Values 2 4 6 8 10 12

△ Soil Vapour Reading parts per million (ppm) 100 200 300 400

▲ Lower Explosive Limit (LEL)

W_P W W_L Plastic Liquid 20 40 60 80

Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT 'N' Value
	85.4 8.5				
	grey **SILTY CLAY**- trace sand stiff moist	SS	8	59	7
	Gravel: 0, Sand: 1%, Silt: 62%, and Clay: 37%. 84.2	SS	9	75	9
	Overburden soil 9.8				
		DC	1		
		DC	2		
		DC	3		
		DC	4		
		DC	5		
		DC	6		
		DC	7		
		DC	8		
		DC	9		
	81.2	DC	10		
	end of borehole 12.8				



SNC·LAVALIN

Borehole details as presented, do not constitute a thorough understanding of all potential conditions present and requires interpretative assistance from a qualified Geotechnical Engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commisioned and the accompanying'Notes to Record of Boreholes'.

Scale: 1 : 42

APPENDIX

E

BOREHOLE PLAN, LOGS, AND PROFILE (FROM BUTTS, ROSS, MAGWOOD & HALL LTD. REPORT DATED JUNE 28, 1968)

BOREHOLE ANALYSIS
BOREHOLE NO. 7
DRILLING DATE Apr 1 23/6
TESTING DATE Apr 1 9/

ep e of Tran per
u a w re s

PENETRATION DATA	HAMMER	DROP
CASING		
CONE		
SAMPLER	140 lbs.	30 in.

va e a

Penetration

end

empl

/m

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BUTTS, ROSS & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS — OTTAWA, ONTARIO

BOREHOLE ANALYSIS
BOREHOLE NO. 7
DRILLING DATE April 23/68
TESTING DATE

CLIENT Department of Transport.
LOCATION Rideau Canal, Lower Brewers.

REMARKS

BOREHOLE ELEVATION

PENETRATION DATA	HAMMER	DROP
CASING		
CONE		
SAMPLER		

DEPTHS MEASURED FROM GROUND LEVEL

Depth	Blows/ft	Cone Penetration Blows/Foot		Description and Remarks	Sample Type No.	Sample Blows/ft	M.C.	L.L.	P.L.	P.I.	U.C.	Vane und.	Vane rem.	Vane sen.	U.W.		Water Table Date & Time
32'5"			▨														
				Cored 5'(32'5"to 37'5")													
				Recovered 4'5"													
				Coarse grained red,													
				black and green granite													
37'5"				traces of phlogopite.													

Symbols

M.C. = Moisture content
L.L. = Liquid limit
P.L. = Plastic limit
P.I. = Plasticity index
U/C = Unconfined compressive strength tons sq/ft
U.W. = Unit weight
und. = Undisturbed shear strength Tons/sq ft
rem. = Remoulded " " "
sen. = Sensitivity - $\frac{und}{rem}$

Plate No.
13

BUTTS, ROSS & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS — OTTAWA, ONTARIO

BOREHOLE ANALYSIS
BOREHOLE NO. 11
DRILLING DATE April 25,68
TESTING DATE April 29,68

CLIENT Department of Transport.
LOCATION Rideau Canal, Lower Brewers.

REMARKS

BOREHOLE ELEVATION 304.25 at a waterboard reading of 7'

PENETRATION DATA	HAMMER	DROP
CASING		
CONE		
SAMPLER	140 lbs.	30 ins.

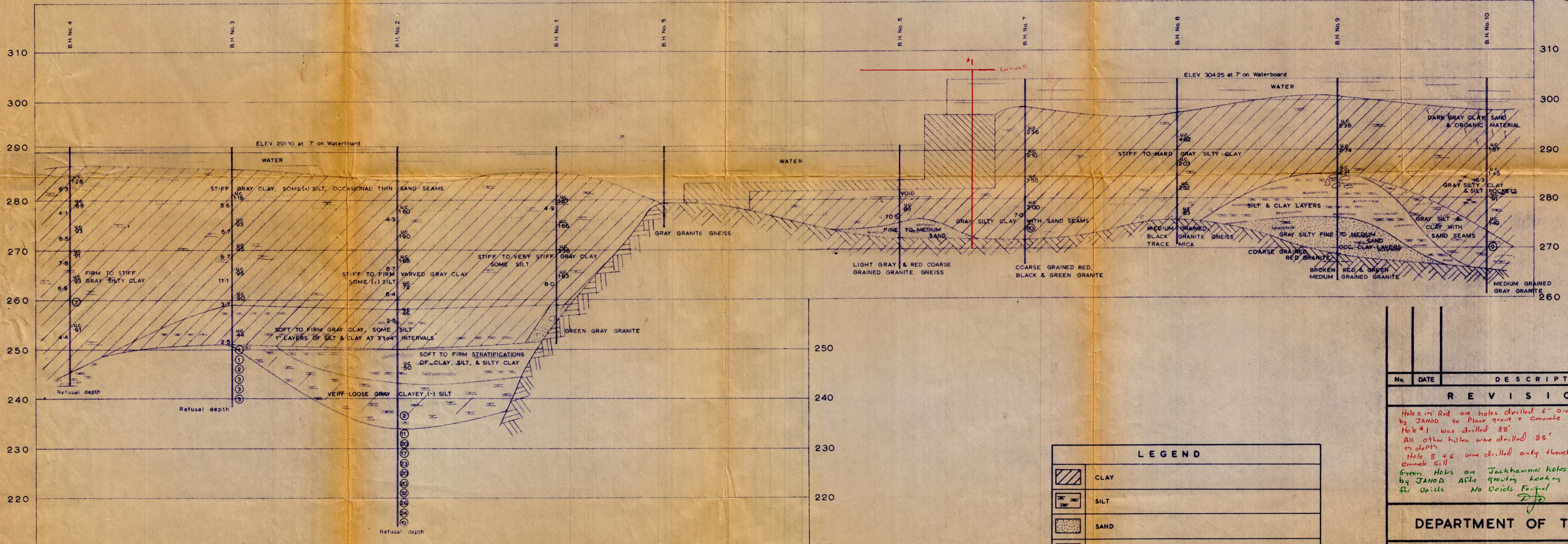
DEPTHS MEASURED FROM GROUND LEVEL

Depth	Blows/ft	Cone Penetration	Description and Remarks	Sample		M.C.	L.L.	P.L.	P.I.	U.C.	Vane			U.W.		Water Table
		Blows/Foot		Type No.	Blows/ft						und.	rem.	sen.			Date & Time
			Water													
4'10"			Dark gray clay, some (+)silt, some pea gravel													
7'			Very stiff gray clay some silt.	SS1	19											
			Stiff to very stiff gray clay, some(+) silt.	T02						1.90						
			Stiff to firm pockets of badly fissured clay AND fine sand and silt.	T03						1.07						
			Stiff gray silty cla 1"layer of silt at 24'	T04						1.37						
28'7"			Cored 4'9½"(28'7"to 33'4½") Recovered 4'3" Medium to coarse grained gray and red granite gneiss.													
33'4½"																

Symbols

M.C. = Moisture content
L.L. = Liquid limit
P.L. = Plastic limit
P.I. = Plasticity index
U/C = Unconfined compressive strength tons sq/ft
U.W. = Unit weight
und. = Undisturbed shear strength Tons/sq ft
rem. = Remoulded " " "
sen. = Sensitivity - $\frac{\text{und}}{\text{rem}}$

Plate No. 19



NOTE:

THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLES. BETWEEN BOREHOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

LEGEND

▨	CLAY
≡	SILT
░	SAND
⁘	GRAVEL
⸾	ORGANIC MATERIAL
▧	PROVEN BEDROCK
u.c. 2·52	UNCONFINED COMPRESSIVE STRENGTH TONS PER SQ. FT.
6·4	VANE SENSITIVITY OF CLAY
⑰	BLOWS PER FOOT, STANDARD PENETRATION RESISTANCE 140 lb. HAMMER - 30in. DROP

No.	DATE	DESCRIPTION	BY

REVISIONS

Holes in Red are holes drilled 6" Dia by JANOD to Place grout + Concrete.
Hole #1 was drilled 38'
All other holes were drilled 25' in depth
Hole 5 + 6 were drilled only through concrete sill.
Green Holes are Jackhammer holes by JANOD After grouting Looking for Voids No Voids Found

DEPARTMENT OF TRANSPORT

LOWER AND UPPER WHARFS
LOWER BREWERS LOCKSTATION

ARCHITECT

BUTTS, ROSS, MAGWOOD & HALL LTD.
CONSULTING CIVIL ENGINEERS
1489D MERIVALE RD. OTTAWA TEL. 224 1414

DRAWN BY: W. O.		DATE: JUNE 1968	
CHECKED BY: E. O. B.		SCALE: HOR: 1" = 30' VER: 1" = 10'	
APPROVED BY: E. O. B.		CONT. No. 8-259-L	DWG. No. 22

TRANSTEX 13 500-1-68