Geotechnical Investigation

John Paul's Lane Bridge
Eskasoni, NS

File No: 193135

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1.0 INTRODUCTION

Acting on the request and authorization of Strait Engineering Limited (SEL), Harbourside Geotechnical Consultants (Harbourside) have completed a geotechnical investigation to aid with the design and construction of the John Paul's Lane Bridge replacement in Eskasoni, Nova Scotia.

The existing bridge is nearing the end of its serviceable lifespan and acts as the sole connection between a residential area and the community of Eskasoni. A replacement structure is required to carry John Paul's Lane over a narrow stretch of water on the north shore of the eastern bay of Bras d'Or Lake. The purpose of this investigation was to determine the subsurface conditions at this site and to develop recommendations to aid with foundation design and construction.

The scope of work completed for this project includes the following:

- Completion of a geotechnical field investigation comprised of two boreholes;
- A laboratory testing program; and
- Preparation of this report detailing the findings of the field investigation and laboratory analyses, as well as discussion and recommendations to aid with site earthworks and foundation design.

2.0 SITE DESCRIPTION

The existing bridge consists of a single-lane, timber bridge deck, approximately 12.3 m in length, 5.4 m in width, supported on steel beams which carries John Paul's Lane over a narrow bay of Bras d'Or Lake in Eskasoni, located approximately 170 m south of the intersection with Route 216. The surrounding landscape is relatively flat, with John Paul's Lane climbing a gentle hill as it extends south onto the populated peninsula. The existing bridge appears to rest on a narrow causeway, the side slopes are vegetated with grass above water level. Lands to the north and south of the bridge are a mix of residential and forested/treed. The tidally influenced water level of the channel below the bridge was approximately 0.7 m at its deepest point at the time of measurement. A temporary bailey bridge was in place and in use immediately west of the existing bridge at the time of the investigation. The project location is shown on Sketch G01, Borehole Location Plan in Appendix C.

Previous experience in the area and geological mapping indicate that the principal overburden strata consists of silty glacial till. Mapping of the bedrock geology indicates that the underlying bedrock at the site is comprised of alluvial conglomerate and sandstone of the Grantmire Formation.

3.0 INVESTIGATIVE PROCEDURES

3.1 GENERAL

The field investigation, consisting of two boreholes, was conducted between November 6 to 8, 2019. Samples of soil were recovered from the boreholes, classified in the field and taken to our laboratory for final classification and testing. A detailed summary of the soil conditions encountered, as well as the sampling and testing carried out, is present in the Borehole Records in Appendix A. Appendix A also includes a document entitled "Symbols and Terms used on Borehole and Test Pit Records", which clarifies terms used through this report and symbols used on the borehole and test pit records.

3.2 BOREHOLES

To support the design and construction of the replacement bridge, a total of two boreholes were advanced. Borehole BH01 was advanced through the centre of the north abutment and borehole BH02 was advanced through the centre of the south abutment. Conditions at each test location were observed and logged by Harbourside personnel. The boreholes were drilled to depths of 30.4 m and 30.6 m, respectively. A standpipe was installed in borehole BH01 and the water level was measured on November 8, 2019. The measured groundwater level is indicated on the Borehole Records in Appendix A.

Boreholes were advanced using HW-sized casing. Soil sampling was carried out at regular intervals using conventional 50-mm diameter split spoon samplers while performing standard penetration testing as described in *ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*. The standard penetration test (SPT) "N-value" is the number of blows required to advance a 50-mm outer-diameter split-spoon sampler a distance of 300 mm into the soil using a standardized drop height and weight. N-values generally provide an indication of soil consistency or compactness and may also be used to aid in estimation of other soil parameters. A pocket penetrometer was also used to aid in the estimation of soil strength parameters for cohesive soils. The test indicates the unconfined compressive strength of cohesive soils by pushing a 6.4 mm diameter piston into the soil sample and measuring the penetration resistance. A record of the sampling carried out is included on the borehole records.

3.3 LABORATORY TESTING

All samples of soil recovered from the test locations were taken to our geotechnical laboratory for final classification and testing. Laboratory testing on select samples included:

- Water content determinations (*ASTM D2216 Standard Test Methods for Laboratory Determination of Water Content of Soil and Rock by Mass*),
- Particle-size analyses (*ASTM D6913 Standard Test Method for Particle-Size Distribution of Soils Using Sieve Analysis*),
- Atterberg Limits (*ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils)*, and

A summary of the testing performed is presented on the borehole records and in separate figures in Appendix B. Soil descriptions used throughout this report are in general accordance with the Unified Soil Classification System (*ASTM D2487 Standard Practice for Classification of Soils for Engineering purposes / ASTM D2488 Standard Practice for Description and Identification of Soils*).

3.4 SURVEYING

The location and ground surface elevation of all boreholes were surveyed by Harbourside personnel with construction-grade GPS equipment. Borehole coordinates are provided in UTM Zone 20 NAD83 CSRS (2010). Elevations are referenced to the Canadian Geodetic Vertical Datum of 2013 (CGVD2013).

4.0 SUBSURFACE CONDITIONS

The subsurface conditions encountered generally consisted of the following sequence:

- Asphalt
- Fill
- Silty Sand
- Upper Clayey Till
- Silty Sand Till
- Lower Clayey Sand Till

The subsurface conditions observed in the boreholes are summarized in Table 1 and the following paragraphs, and are described in additional detail on the borehole records in Appendix A.

Table 1 Summary of Subsurface Conditions

Location	Layer Thickness						Groundwater	Total Depth (m)
	Asphalt (m)	Fill (m)	Silty Sand (m)	Lean Clay with Sand Till (m)	Silty Sand Till (m)	Clayey Sand Till (m)	Depth [elev.][(a)] (m)	
BH01	0.15	2.85	1.04	5.81[(b)]	4.17	>16.36	1.80 [-0.08]	30.38
BH02	0.15	3.81	1.83	-	7.16	>17.66	-	30.61

(a) Elevations referenced to CGVD2013.
(b) Includes 1.50 m of clayey sand till in upper portion of layer

4.1.1 Asphalt

Both test locations were advanced through a layer of asphalt at the road surface. The thickness of this layer was 0.15 m at both locations.

4.1.2 Fill

Fill was encountered below the asphalt layer in both test locations. This layer ranged in thickness from 2.85 to 3.81 m.

The result of a single particle-size analysis on a sample collected from this layer indicated 38 percent gravel, 50 percent sand, and 12 percent silt and clay-sized particles. The water content of four samples ranged from 8 to 18 percent with an average of 12 percent. Standard Penetration Test (SPT) N-values obtained within the fill layer ranged from 9 to 21.

Based on the sampling and lab testing carried out, the fill can be described as brown to grey poorly graded sand with silt and gravel.

4.1.3 Silty Sand

A layer of dark brown to grey silty sand with frequent organics and a few shell fragments (original bottom sediments) was encountered below the fill layer in both boreholes. The layer ranged in thickness from 1.04 m at borehole BH01 to 1.83 m at borehole BH02. Wood was encountered at a depth of 5.00 m in borehole BH02.

The water contents of two samples from this layer were 51 and 410 percent. Standard Penetration Test (SPT) N-values obtained within this layer ranged from 1 to 5.

Based on the sampling and lab testing carried out, the layer can be described as very loose to loose dark brown to grey silty sand.

4.1.4 Upper Clayey Glacial Till

Glacial till consisting of reddish-brown clay with sand was encountered below the loose silty sand layer in BH01. This layer was 5.81 m in thickness, which includes 1.50 m of reddish-brown clayey sand in the upper portion. A pocket of reddish-brown silty sand with gravel was encountered within this layer at a depth of approximately 7.10 m.

The result of two particle-size analysis on samples collected from the upper clayey till indicated 0 to 9 percent gravel, 16 to 52 percent sand, and 39 to 84 percent silt and clay-sized particles. The natural water content of four samples from this layer ranged from 15 to 36 percent.

Three pocket penetrometer tests on the lean clay with sand indicated undrained shear strengths ranging from 135 kPa to equal or greater than 225 kPa. Standard Penetration Test (SPT) N-values obtained within the clay layer ranged from 18 to 36.

Based on the sampling and testing completed, this layer may be described as glacial till consisting of stiff to hard reddish-brown lean clay with sand.

4.1.5 Silty Sand Till

Glacial till, consisting of silty sand was encountered below the lean clay with sand till in BH01 and below the very loose silty sand layer in BH02. This layer ranged in thickness from 4.17 m in BH01 to 7.16 m in BH02. In BH01, a pocket of clean brown sand was encountered at a depth of 12.40 m.

The results of three particle size analyses on the silty sand till are presented in Table 2. The natural water contents of three samples from this layer ranged from 14 to 15 percent.

Table 2 Particle Size Analyses – Silty Sand Till

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification[(a)]	Material Composition by Weight (%)		
				Gravel	Sand	Fines[(b)]
BH01	SS14	10.03 to 10.64	Silty SAND	8	76	16
BH02	SS11	6.95 to 7.56	Well-graded SAND with Silt	12	78	10
BH02	SS16	11.50 to 12.10	Silty SAND	12	74	14

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For Particle size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.

Standard Penetration Test (SPT) N-values obtained within the silty sand layer ranged from 12 to 60. Based on the sampling and testing completed, this layer may be described as glacial till consisting of compact to very dense reddish-brown well-graded sand with silt to silty sand.

4.1.6 Clayey Sand Till

Glacial till, consisting of brown clayey sand was encountered below the silty sand till at both test locations. Both boreholes were terminated within this layer, with a maximum penetration of 17.66 m in BH02. Trace gravel and frequent cobbles were encountered throughout the layer in both boreholes. In BH01, occasional pockets of grey lean clay with sand and black lean clay with sand were observed below a depth of 19.90 m.

The results of three particle size analyses conducted on samples from within the clayey sand till layer are presented in Table 3.

Table 3 Particle Size Analyses – Clayey Sand Till

Location	Sample No.	Sample Depth (m)	ASTM Soil Classification(a)	Material Composition by Weight (%)		
				Gravel	Sand	Fines(b)
BH01	SS18	15.39 to 16.00	Clayey SAND	2	52	46
BH02	GB22	20.50 to 20.85	Lean CLAY with Sand(c)	2	19	79
BH02	SS20	18.40 to 18.90	Clayey SAND	12	58	30

(a) See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
(b) For Particle size analyses performed by sieve, the percent of silt- and clay-sized particles are reported collectively as the percent fines.
(c) Particle size analysis conducted on sample taken from pocket of grey and black clay encountered within the clayey sand till layer.

The natural water contents of nine samples from the clayey sand till ranged from 8 to 17 percent. Atterberg limit testing conducted on three samples from the clayey sand till indicated that this soil has a plastic limit of 12 to 15 and a liquid limit of 22 to 28.

Eleven pocket penetrometer tests on the clayey sand indicated undrained shear strengths ranging from 125 kPa to equal or greater than 225 kPa, with the majority of readings 200 kPa or higher. A single pocket penetrometer test on a pocket of lean clay with sand indicated an undrained shear strength equal or greater than 225 kPa. Standard Penetration Test (SPT) N-values obtained within the clayey sand layer ranged from 35 to 62. Refusal occurred eleven times within this layer, together with recovered samples from coring suggested frequent cobbles.

Based on the sampling and testing completed, this layer may be described as glacial till consisting of hard brown clayey sand.

4.1.7 Bedrock

Bedrock was not encountered in either test location during the investigation.

4.1.8 Groundwater

A standpipe piezometer was installed in borehole BH01. The groundwater surface was measured on November 8, 2019 at a depth of 1.80 m below the ground surface (elevation -0.08 m). This groundwater level closely matched the elevation of the tidally influenced water of Bras d'Or Lake, below the bridge

Groundwater levels at the site are likely controlled by the tide, but may also fluctuate with precipitation events and in response to climatic and seasonal weather trends.

5.0 DISCUSSION AND RECOMMENDATIONS

Based on the span of the structure, we assume a single span integral abutment, rigid frame, or modular panel bridge will be constructed for the replacement structure. To allow the bridge to be an integral abutment design and to protect the foundations against scour we are providing recommendations for driven pile foundations.

The glacial till encountered would also be suitable to support shallow foundations, but the design would need to meet the scour requirements from Section 1.9.5.2 of the Canadian Highway Bridge Design Code. Additionally, considerable water control measures, such as cofferdams, would be required to construct the foundations on native till or to replace the loose silty sand and existing fills with structural fill, which was encountered approximately 2 to 4 meters below water level. Based on discussions with the designer it is anticipated that spread footings will not be used.

At this time, it is unknown if road grades are to be significantly raised. If grades are to be raised at the bridge site, consideration will need to be given to potential settlement of the loose silty sand encountered below the existing fills.

5.1 SITE PREPARATION

Base preparation for the pile caps will require removal of the existing foundation and fills to the required underside of pile cap elevation. Depending on design grades, exposure of the loose silty sand layer may occur, and if so, should be sub excavated and replaced with structural fill to provide a stable working surface.

If the approaches are increased in height or widened, the grass/rootmat and any topsoil should be removed to expose the existing fill materials before placement of additional material. The periods between grubbing and filling should be minimized to limit disturbance and erosion of the exposed soils. Prepared surfaces should be protected to minimize the amount of degradation.

5.2 STRUCTURAL FILL

Structural fill should be used if over-excavation and replacement is required beneath pile caps and spread footings for retaining walls. Imported structural fill should consist of sand and gravel or well-graded quarried rock with a maximum particle size of 200 mm and a fines content less than 10 percent. Fill against structure, gravel type 1, and gravel type 2 as specified by NSTIR's Standard Specification for Highway Construction and Maintenance (NSTIR's Standard Specification) are examples of suitable materials. As a minimum, structural fill should extend below the area of the pile cap or shallow foundation plus the horizontal distance beyond the outside edge of the footprint to include a structural splay of 1H:1V (the zone of influence).

Structural fill should be compacted to 100 percent of the standard Proctor maximum dry density (SPMDD) as determined by *ASTM D698 Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort.* Materials where Proctor densities are not applicable, such as coarse rock fills, should be compacted to a relative density of at least 80 percent. All structural fill should be placed at a water content that allows compaction to the specified density.

Lift thicknesses for structural fill should be compatible with the compaction equipment used to ensure that the required density is achieved through the entire lift. Placement of structural fill should be monitored by experienced geotechnical personnel to ensure that the required density is achieved.

5.3 APPROACH FILL

As noted above, it is currently unknown as to whether grades will be increased for the new bridge alignment. A layer of loose silty sand (original bottom sediment) was encountered below the fills at the bridge foundation locations, and it is suspected, but not confirmed, that this layer may extend further longitudinally under the roadway. Preliminary estimates of settlement, based on the thickness encountered in the boreholes, indicate that increasing grades by 1 to 2 m would result in settlements of 50 to 80 mm respectively.

Approach fills should be comprised of sand and gravel, or well-graded rock fill. Approach fill should be compacted to at least 95 percent of the SPMDD and the upper 1.5 m below subgrade should be compacted to a minimum of 100 percent of the SPMDD (in accordance with NSTIR's Standard Specifications). To ensure compaction through the entire depth of the lift, fill should be placed in lifts compatible with the compaction equipment used.

If fine-grained materials (e.g. common borrow derived from local glacial till) are used as approach fills, they will be prone to disturbance by water and traffic. Prepared surfaces should be protected to minimize the amount of degradation.

5.4 SLOPE STABILITY

Stability of the approach fill will mostly be controlled by the properties of the fill material used. All permanent fill slopes constructed of common fill should be no steeper than 2 horizontal to 1 vertical (2H:1V). Where steeper slopes are required, permanent slopes as steep as 1.5H:1V may be constructed provided that a minimum width of angular, well-graded rock fill is placed on the slope. The width of the rock fill required will depend on several factors including the embankment height.

Temporary cut slopes in excavations should be no steeper than 1.5H:1V. Shallower slopes may be required in excavations below the water level.

5.5 FOUNDATIONS

Based on our geotechnical investigation and our understanding of the proposed design, we are providing recommendations for driven piles which will act in a combination of end bearing and shaft friction.

5.5.1 General

The design depth of frost penetration should be taken as 1.2 m. The bottom of footings in frost susceptible soils should be located below this depth to prevent heave under frost action. Where this depth is not maintained, an equivalent combination of soil and insulation, or other measures such as excavation and replacement with non-frost susceptible soil, may be used to protect the structure from frost.

Base preparation for the pile cap will require removal of the existing foundation and fills to the required underside of pile cap elevation. Depending on design grades, exposure of the loose silty sand layer may occur, and if so, should be over-excavated and replaced with structural fill to provide a stable working surface.

5.5.2 Driven Pile Foundations

Driven Steel friction piles are a suitable option to support the new structure. Friction piles may be designed using the ULS geotechnical axial compressive resistance provided in Table 4, below. We would be pleased to review alternative pile sections upon your request. In accordance with the Canadian Highway Bridge Design Code (CAN/CSA S6-14, 2014) Clause 6.9.1 the values presented in Table 3 include a resistance factor of 0.4. The resistance provided is based on a penetration length into the underlying glacial till, as described in the table.

Table 4 Factored Axial Compressive Resistance at ULS for Driven Piles

Pile Embedment into Glacial Till (m)	Factored Axial Compressive Resistance at ULS (kN)					
	HP 360 x 152	HP 360 x 132	HP 310 x 132	HP 310 x 110	406 x 12.7 mm OEPP	406 x 12.7 mm CEPP
10	540	530	455	450	200	405
15	785	775	665	655	400	615
20	1020	1010	865	850	605	820

The resistance of pile groups may be calculated as the sum of the individual pile capacities provided that the centre-to-centre spacing of the piles is at least three pile diameters. The expected settlement of piles driven to the noted embedment will be less than 10 mm.

Piles should be driven to the specified embedment with a hammer having a minimum rated energy of 400 Joules/cm^2 of steel cross-sectional area. The contractor should provide full details on the method of installation and equipment to the geotechnical engineer prior to starting the work.

The resistance will be achieved mainly through shaft resistance in the glacial till. It is anticipated that in order for the piles to achieve their full geotechnical axial resistance they will need to "set-up" over time. Pile set-up is realized due to a temporary loss in shaft friction from increases in pore-water pressures as a result of driving. As excess pore-water pressures dissipate over time, increases in axial resistance relative to initial driving are obtained. In order to minimize the generation of excess pore-water pressures piles should be installed by driving (impact) methods only, installation by vibratory methods or drilling should not be permitted.

Dynamic pile monitoring (e.g. Pile Driving Analyzer System) should be carried out on the initial pile installations to verify that overstressing does not occur, that the hammer is operating within normal efficiencies, and that the estimated resistance provided for design is achieved at the embedment criteria.

Dynamic pile monitoring should be performed on at least one pile at end of initial drive (EOID) and at the beginning of re-strike (BOR) at each foundation element and on a minimum of 10 percent of all piles. Full-time inspection by qualified geotechnical personnel is recommended during all pile installation. In order to assess the time related effects (amount of pile set-up with time), testing at restrike conditions should be conducted at 24 and 72 hours after the end of initial driving. Estimates of the long term ULS geotechnical axial compressive resistance can be made from testing at these intervals, should the piles not achieve the required resistance at the time of initial driving.

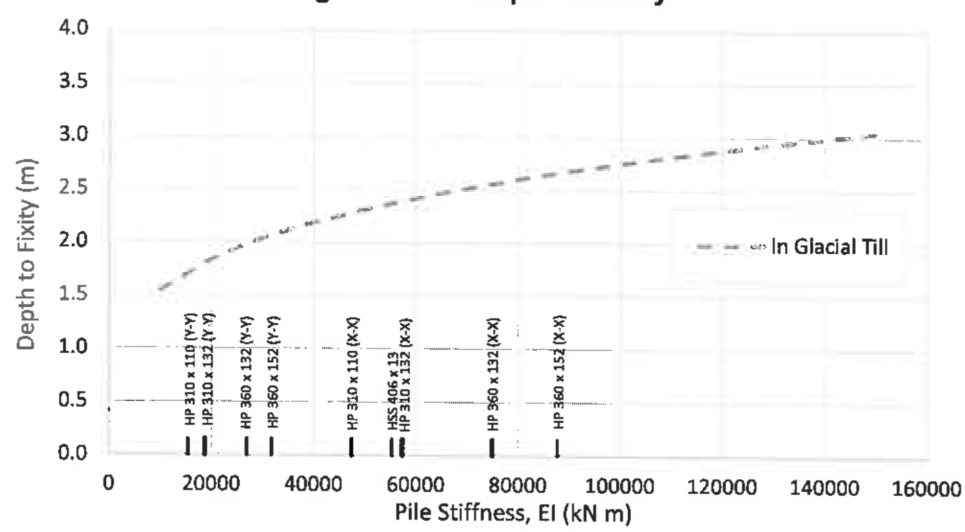
Current industry practice for re-strikes is 24 to 72 hours after EOID. Notwithstanding current practice, it should be noted that the time period recommended by the Canadian Foundation Engineering Manual (4th Edition, 2006) is two weeks irrespective of pile-soil conditions.

5.5.3 Lateral Pile Behavior

For consideration of lateral loads, the depth to fixity depends on the stiffness of the pile and the material in which the pile is embedded. The depth to fixity for a range of pile stiffnesses (EI) for piles where the depth to fixity occurs within the native glacial till is provided in Figure 1, below. Virtually no lateral support will be provided from the loose silty sand layer, and therefore should be discounted from the analyses.

Non-linear "p-y" springs can also be used to represent the reaction along the soil-pile interface. P-y springs relate lateral ground pressure to lateral deformation. These springs, which vary along the length of the pile, depend on the soil type, soil stiffness, soil strength, pile geometry, pile stiffness, and group effects. P-y springs can be provided, upon request, when preliminary pile details and layout are known.

Figure 1 Depth to Fixity



5.5.4 Down-Drag Loads

When piles are installed through soil subject to settlements (such as the loose silty sand encountered in this investigation upon new loading) the resulting downward movement of the soil around the piles, as well as in any soil above the settling layer, induces down-drag forces on the piles and any attached structures (e.g. the pile caps and abutments). The down-drag forces will only occur on the piles to the bottom of the compressible (i.e. loose silty sand) layer.

If grades are significantly raised for the roadway, we would anticipate settlement of the loose silty sand layer and drag loads due to negative skin friction on the piles. Drag loads increase the structural loads in the pile and thus have to be considered in structural design of the piles. In this assessment, it is important to note that drag load and transient live load do not combine and that separate loading cases must be considered:

- Permanent load plus drag load, but no transient live load; and
- Permanent load and transient live load, but no drag load.

The magnitude of down-drag loads may be calculated based on the vertical effective stress and the combined shaft resistance factor, β. Values of the total unit weight, submerged unit weight, and β for use in down-drag analyses are presented in Table 5.

Table 5 Combined Shaft Resistance Factor, β for use in Down-Drag Analyses

Material	Total Unit Weight (kN/m^3)	Submerged Unit Weight (kN/m^3)	Combined Shaft Resistance Factor, β
New Abutment Fills	22.0	12.0	0.55
Existing Fills	21.0	11.0	0.45
Loose Silty Sand	16.5	6.5	0.27

5.6 ABUTMENT BACKFILL

The abutments for the new structure should be backfilled with fill against structure as specified by NSTIR's Standard Specifications. Backfill should be placed in lifts and compacted, as a minimum, to 95 percent of the standard Proctor maximum dry density or to the manufacturer's specifications. Care should be taken not to damage abutments when performing backfilling and compaction operations. To limit compaction-induced stresses, compaction within 1.5 m of retaining structures should be performed with a walk-behind vibratory plate tamper or other lightweight compaction equipment in lieu of a vibratory drum roller.

All drainage materials, including backfill and drainage blankets, must be designed to limit loss of soil according to filter criteria.

5.7 RETAINING WALLS

It is anticipated that wingwalls for a bridge type structure will be cantilevered from the abutment and that footings will not be required to support these components of the bridge. Backfill placed against retaining walls should conform to NSTIR Standard Specifications for fill against structure. A drainage system with a positive outlet should be included to prevent water from backing up against the retaining structure. The extent of the granular backfill should be in accordance with the wall design requirements. All backfill should be placed in lifts and compacted to 95 percent of maximum standard Proctor dry density. Compaction immediately adjacent to the wall should be accomplished with relatively thin soil lifts and light compaction equipment to prevent over-stressing of the wall. The parameters presented in Table 6 may be used for design of retaining walls.

The earth pressure coefficients used for design should be selected based on the appropriate finished back-slope angle.

Table 6 Unfactored Geotechnical Parameters (Retaining Walls)

Parameter	Value	
	Glacial Till, and Compacted Common Fill(a)	Compacted NSTIR Fill Against Structure, Gravel Type 1 or Gravel Type 2 (a) (b)
Effective Angle of Internal Friction, degrees	32	36
Effective Cohesion, kPa	0	0
Undrained Shear Strength, kPa	100	-
Total Unit Weight, kN/m^3	22.0	22.0

Parameter	Value	
	Glacial Till, and Compacted Common Fill[(a)]	Compacted NSTIR Fill Against Structure, Gravel Type 1 or Gravel Type 2 [(a) (b)]
Submerged Unit Weight[(c)], kN/m³	11.7	12.2
Coefficient of Active Earth Pressure[(d)]	0.31	0.26
Coefficient of Passive Earth Pressure[(d)]	3.25	3.85
Coefficient of At-Rest Earth Pressure[(d)]	0.47	0.41
Friction Factor, Soil/Concrete Interface[(e)]	0.40	0.50
Friction Factor, Soil/ Precast Concrete Interface	0.25	0.45

(a) Compacted material shall be placed in lifts and suitably compacted as described above.
(b) As NSTIR's Standard Specification for Highway Construction and Maintenance (2011).
(c) For uplift design the groundwater table should be assumed at the ground surface and submerged unit weights should be used.
(d) Coefficients of earth pressure presented in table assume a frictionless wall with a vertical back face and a horizontal back slope.
(e) For mass concrete or masonry, lower values will be required for formed or pre-cast concrete. A geotechnical resistance factor of 0.8 should be applied in sliding analysis.

Structures that can tolerate little or no movement should be designed for at-rest lateral earth pressures. The factored bearing resistance at ultimate limit states (ULS) includes a resistance factor of 0.5. The serviceability limit states (SLS) bearing resistance is provided for allowable settlements of 25 and 35 mm.

5.8 TEMPORARY COFFER DAMS

The following unfactored values may be used for the design of temporary cofferdams if required. Steel sheet piles should be advanced sufficiently into the glacial till stratum to prevent base heave during dewatering.

Table 7 Unfactored Geotechnical Parameters (Temporary Cofferdams)

Parameter	Fill	Loose Silty Sand	Glacial Till
Effective Angle of Internal Friction, degrees	30	26	32
Effective Cohesion, kPa	0	-	0
Undrained Shear Strength (kPa)	-	5	100
Total Unit Weight, kN/m³	20	16	21
Submerged Unit Weight, kN/m³	10	6	11
Coefficient of Active Earth Pressure[(a)]	0.33	0.39	0.30
Coefficient of Passive Earth Pressure[(a)]	3.00	2.60	3.33
Coefficient of At-Rest Earth Pressure[(a)]	0.50	0.56	0.47
Friction Factor, Soil/Steel Interface[(a)]	0.25	0.20	0.25

(a) Coefficients of earth pressure presented in table assume a frictionless wall with a vertical back face and a horizontal back slope. Values can be provided for different conditions upon request.

5.9 WINTER WEATHER CONDITIONS

Where practical, earthwork during freezing temperatures should be avoided. In the event of winter construction, special measures will be required to ensure that fills and foundations are not placed on frozen ground and that the soils are protected from freezing after placement. Even following careful procedures and precautions experience has shown that earthworks in these types of soils often become impractical at temperatures below approximately -5°C.

5.10 SEISMIC SITE CLASSIFICATION

Based on the findings at the boreholes, the site classification for seismic site response in accordance with Clause 4.4.3.2 of the Canadian Highway Bridge Design Code (CAN/CSA-S6-14, 2014) is Seismic Site Class D (stiff soil).

6.0 CLOSURE

This report has been prepared to assist in the design and construction of the John Paul's Lane Bridge. This report has been prepared for the sole benefit of Strait Engineering Limited and their agents. Any use which a third party makes of this report is the responsibility of such third party.

The recommendations made in this report are in accordance with our present understanding of your project. If any details are included in the final design of the proposed structure that differ from the assumptions outlined in this report, the geotechnical engineer should be consulted.

This report is based on the site conditions encountered by Harbourside Geotechnical Consultants at the time of the work at the specific sampling locations and can only be extrapolated to a limited extent around these locations. Should any conditions differ from those detailed on the borehole records, the engineer should be notified to allow reassessment of any design assumptions.

If you have any questions or require any additional information, please do not hesitate to contact the undersigned at your convenience.

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APPENDIX A

Symbols and Terms Used on Borehole and Test Pit Records

Borehole Records BH01 and BH02

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:

USCS SOIL CLASSIFICATION SYMBOLS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN 75 µm SIEVE SIZE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON 4.75 mm SIEVE	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES. LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL – SAND – SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL – SAND – CLAY MIXTURES
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING THE 4.75 mm SIEVE	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS, SAND – SILT MIXTURES
				SC	CLAYEY SANDS, SAND – CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN 75 µm SIEZE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

OTHER COMMONLY USED SYMBOLS

GLACIAL TILL		UNSTRATIFIED GLACIAL DEPOSIT RANGING FROM CLAY TO BOULDERS
BEDROCK		IGNEOUS BEDROCK
		METAMORPHIC BEDROCK
		SEDIMENTARY BEDROCK
MATERIALS PLACED BY HUMANS		FILL: SUBSURFACE MATERIALS IDENTIFIED AS PLACED BY HUMANS
		ASPHALT
		CONCRETE

SAMPLE TYPE

SS	Split Spoon (obtained by performing SPT)
ST	Shelby Tube (Thin-Walled Tube)
GB	Grab Sample
PS	Piston Sample
WS	Wash Sample
HQ, NQ, AQ, BQ, etc.	Rock Core Samples Obtained Using Standard Size Diamond Bits

SPT N-VALUE (N-INDEX)

The standard penetration test (SPT) provides a qualitative evaluation of compactness and a qualitative comparison of subsoil stratification. The SPT is performed in in the bottom of a borehole where a split-barrel sampler having an outside diameter of 50.8 mm is impacted using a hammer weighing 623 N falling 0.76 m for each hammer blow. The SPT N-value is the blow count representation of the penetration resistance of the soil. In accordance with ASTM D1586, the N-value, reported in blows per 300 mm, equals the sum of the number of blows (N) required to drive the sampler over the depth interval of 150 to 450 mm. However, when a 600 mm sampler is used the number of blows (N) required to drive the sampler over the interval of 300 to 600 mm may be reported if this value is lower. For samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in mm (e.g. 50/120). Although some methods make use of N-values corrected for various factors (for equipment used, overburden stress, length of drill rod, etc.) no corrections have been applied to the N-values presented on the logs.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests (DCPT) are performed using a standard 60-degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the SPT test. The DCPT value is the number of blows of the hammer required to drive the cone 300 mm. The DCPT provides a qualitative evaluation of compactness and allows for a qualitative comparison of subsurface stratification.

RECOVERY

For soil samples, recovery is recorded as the total length of the soil sample recovered. For rock core, recovery is expressed as a percentage of the total length drilled on a per run basis.

OTHER TESTS

S	Sieve Analysis	CD	Consolidated-Drained Triaxial	C	Consolidation
H	Hydrometer Analysis	CU	Consolidated-Undrained Triaxial	Q_u	Unconfined Compression
γ	Unit Weight	UU	Unconsolidated Undrained Triaxial	I_p	Point Load Index, Ip(50)
G_s	Specific Gravity of Soil Particles	DS	Direct Shear	k	Laboratory Permeability

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	Vegetation, roots, and moss with organic matter and topsoil typically forming a mattress at the ground surface.
Topsoil	Mixture of soil and humus capable of supporting vegetative growth.
Peat	A soil composed of vegetable tissue in various stages of decomposition usually with an organic odor, a dark-brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous.
Till	Non-stratified glacial deposit which may range from clay to boulders
Fill	Artificial (man-made) deposits transported and placed on the natural surface of soil or rock.

Terminology describing soil structure:

Homogeneous	The lack of visible bedding and the same appearance and colour throughout
Desiccated	Having visible signs of weathering by oxidation of clay minerals, shrinking cracks, etc.
Fissured	Having cracks and hence a blocky structure
Stratified	Composed of regular alternating successions of different soil types
Varved	Comprised of regular alternating successions of silt and clay which were transported into freshwater lakes by melt water
Layer	> 75 mm
Seam	2 mm to 75 mm
Parting	< 2 mm
Pocket	Small erratic deposit, usually less than 300 mm
Lens	Lenticular deposit

Terminology describing soil types:
Soils are described in accordance with the Unified Soil Classification System (USCS) as described in ASTM D2487 and ASTM D2488. This system classifies soil into categories representing the results of laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index. Using this system, soils are assigned a group name (e.g. silty sand) and symbol (e.g. SM). The various groupings of this classification system have been devised to correlate in a general way with the engineering behavior of soils. Laboratory tests are performed on the portion of the sample passing the 75 mm sieve.

When laboratory test results indicate that that the soil is close to another classification group, the borderline condition can be indicated with two symbols separated by a slash (e.g. CL/CH).

Terminology describing cobbles, boulders, and non-matrix materials:
Materials outside of the USCS (e.g. particles larger than 75 mm, organic matter, construction debris) are described based on the proportion of these materials by weight using the following terminology:

Trace, or occasional	< 10%
Some	10% to 20%
Frequent	> 20%

Terminology describing the compactness condition of cohesionless soils:
A qualitative term describing the compactness condition of a cohesionless soil is interpreted from the SPT N-value (also known as the N-index). The relationship between the SPT N-value and the compactness condition is shown in the following table.

Compactness Condition	SPT N-Value (blows per 0.3 m)
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Over 50

Terminology describing the compactness condition of cohesive soils:
Cohesive soils can be classified in relation to undrained strength. Undrained strength can be determined by a number of tests including: unconfined compression tests, field and laboratory vane tests, laboratory fall-cone tests, shear-box tests, and triaxial tests. The consistency and undrained shear strength may also be approximately related the SPT N-Value. The relationship between the consistency and the undrained shear strength, as well as a rough correlation with SPT N-Value as shown in the following table.

Consistency	Undrained Shear Strength (kPa)	SPT N-Value (blows per 0.3 m)
Very Soft	< 12	< 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

ROCK DESCRIPTION

Rock is a natural aggregate of minerals that cannot be readily broken by hand and that will not disintegrate on a first wetting and drying cycle. A rockmass comprises blocks of intact rock that are separated by discontinuities such as cleavage, bedding planes, joints, shears and faults.

Terminology Describing Geological Classification of Rock:

Rock is classified with respect to its geological origin or lithology as follows:

Igneous Rocks	Rocks such as granite, diorite, and basalt, which are formed by the solidification of molten material.
Sedimentary Rocks	Rocks such as sandstone, limestone and shale, which are formed by the lithification of sedimentary soils.
Metamorphic Rocks	Rocks such as quartzite, schist, and gneiss, which have been altered by the application of intense heat and/or pressure.

Terminology Describing the Strength of Intact Rock:

Strength is the maximum stress level that can be carried by a specimen. Rocks may be classified based on their intact strength as shown in the following table.

Term	Unconfined Compressive Strength (MPa)
Extremely Weak	0.25 to 1
Very Weak	1 to 5
Weak	5 to 25
Medium Strong	25 to 50
Strong	50 to 100
Very Strong	100 to 250
Extremely Strong	> 250

Terminology Describing Discontinuity Spacing

The structural integrity of a rockmass will be affected by the presence of discontinuities. The spacing of discontinuities can vary from extremely wide to extremely close as indicated in the table below.

Term	Spacing Width (m)
Extremely Close	< 0.02
Very Close	0.02 to 0.06
Close	0.06 to 0.20
Moderately Close	0.20 to 0.6
Wide	0.6 to 2.0
Very Wide	2.0 to 6.0
Extremely Wide	> 6.0

Rock Quality Designation (RQD)

RQD is an indirect measure of the number of fractures within a rockmass. The method provides a quick and objective technique to estimate rockmass quality during diamond drill core logging. All pieces of intact and sound rock greater than 100 mm long are summed and divided by the total length of the core run in accordance with ASTM D6032.

RQD Classification	RQD (%)
Very Poor Quality	0 to 25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

Terminology to Describe Rock Weathering

The state of weathering significantly alters the geotechnical behaviour of rocks and rockmasses. Weathering of the rockmass may be classified as shown in the following table.

Term	Description
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.
Slightly Weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker than its fresh condition.
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones
Highly Weathered	More than a half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.

HARBOURSIDE
Geotechnical Consultants

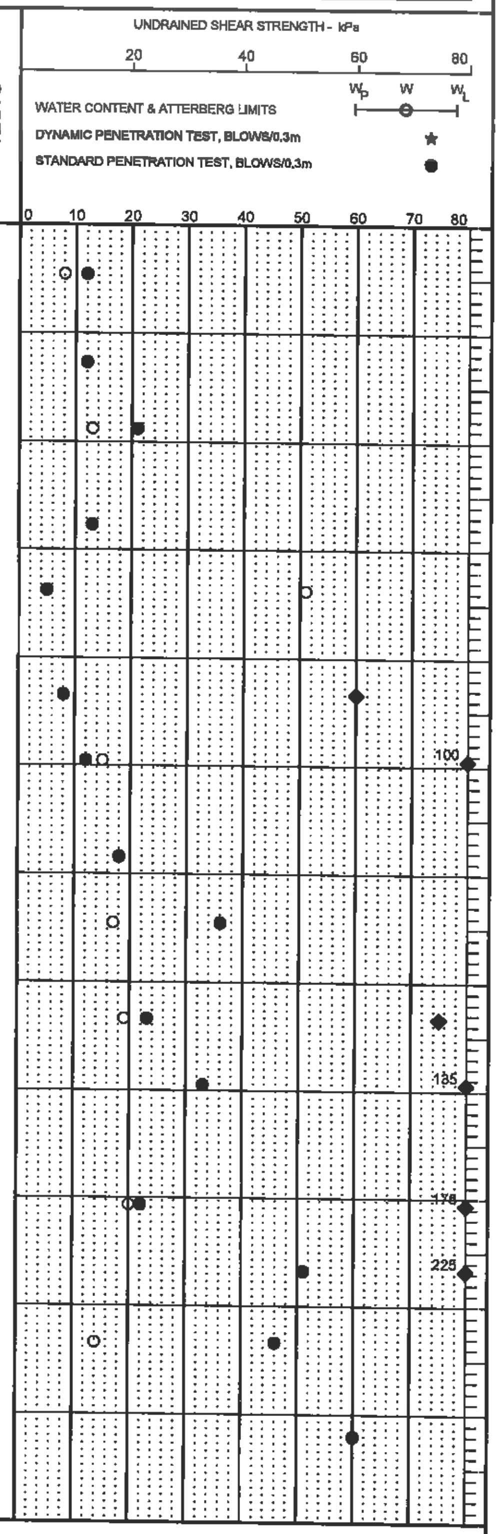
BOREHOLE RECORD

BH01

N: 5089447.39 E: 683716.59

CLIENT	STRAIT ENGINEERING LIMITED	PROJECT No.	193135
LOCATION	JOHN PAUL'S LANE BRIDGE, ESKASONI, NOVA SCOTIA	DATUM	CGVD2013
DATES: BORING	06/11/2019 TO 07/11/2019 WATER LEVEL 08/11/2019	BH SIZE	HW/HQ

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)	OTHER TESTS
	1.72								
	1.57	ASPHALT							
		FILL: brown to grey poorly graded sand with silt and gravel			SS	1	250	7-6-6-9 (12)	
1					SS	2	100	8-7-5-10 (12)	
2				▼	SS	3	300	12-12-9-12 (21)	S
					SS	4	50	9-6-7-8 (13)	
3	-1.28	Loose dark brown to grey silty sand with frequent organics - with trace shell fragments			SS	5	300	1-1-4-5 (5)	
4	-2.32	Stiff reddish-brown clayey sand TILL - with trace gravel			SS	6	150	12-6-4-4 (8)	
5					SS	7	450	6-4-8-8 (12)	S
6	-3.82	Very stiff to hard reddish-brown lean clay with sand TILL			SS	8	300	6-8-10-14 (18)	
					SS	9	375	15-18-18-21 (36)	S
7		- pocket of reddish-brown silty sand with gravel at 7.1 m			SS	10	375	29-14-9-16 (23)	
8					SS	11	200	19-17-16-24 (33)	
9					SS	12	550	7-8-14-24 (22)	
					SS	13	400	16-21-30-36 (51)	
10	-8.13	Dense to very dense reddish-brown silty sand TILL - trace to with gravel			SS	14	425	16-24-22-25 (46)	S
11					SS	15	250	8-22-38-31 (60)	



HARBOURSIDE GEOTECHNICAL CONSULTANTS, BOREHOLE RECORD 21/11/19

(Continued Next Page)

BOREHOLE RECORD

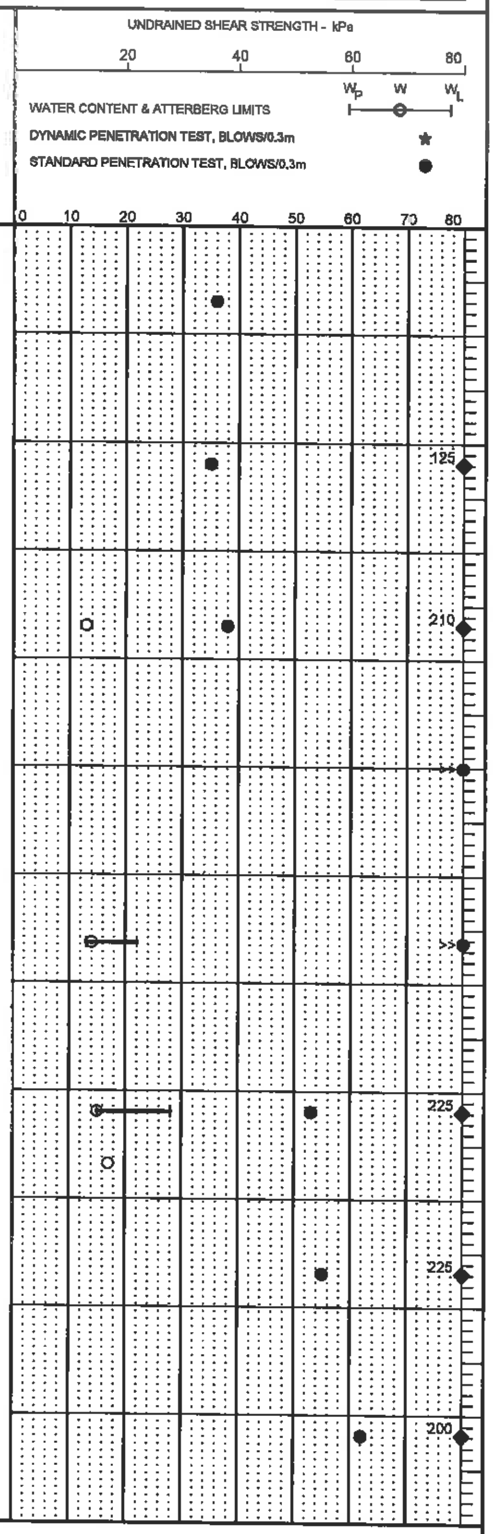
BH01

HARBOURSIDE Geotechnical Consultants

N: 5089447.39 E: 683716.59

CLIENT: STRAIT ENGINEERING LIMITED
PROJECT No.: 193135
LOCATION: JOHN PAUL'S LANE BRIDGE, ESKASONI, NOVA SCOTIA
DATUM: CGVD2013
DATES: BORING 06/11/2019 TO 07/11/2019 WATER LEVEL 08/11/2019
BH SIZE: HW/HQ

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)	OTHER TESTS
13		Dense to very dense reddish-brown silty sand TILL - trace to with gravel *(continued)* - with brown sand pocket at 12.4 m			SS	16	200	3-10-26-37 (36)	
14	-12.30	Hard brown clayey sand TILL - with trace gravel - with frequent cobbles			SS	17	350	12-16-19-20 (35)	
15–16					SS	18	450	13-17-21-34 (38)	S
17					SS	19	200	50-50 / 75 mm	
18–19					SS	20	375	21-45-113-50 / 25 mm	
20		- with occasional pockets of grey lean clay with sand and black lean clay with sand below 19.9 m			SS	21	350	13-24-29-48 (53)	
21					GB	22	350	-	S
22					SS	23	525	12-20-35-50 (55)	
23					SS	24	375	23-28-34-70 (62)	



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BOREHOLE RECORD

BH01

N: 5089447.39 E: 683716.59

CLIENT: STRAIT ENGINEERING LIMITED
PROJECT No.: 193135
LOCATION: JOHN PAUL'S LANE BRIDGE, ESKASONI, NOVA SCOTIA
DATUM: CGVD2013
DATES: BORING: 06/11/2019 TO 07/11/2019
WATER LEVEL: 08/11/2019
BH SIZE: HW/HQ

UNDRAINED SHEAR STRENGTH - kPa (20, 40, 60, 80)
WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC PENETRATION TEST, BLOWS/0.3m ★
STANDARD PENETRATION TEST, BLOWS/0.3m ●

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	GRAPHIC LOG	WATER LEVEL	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)	OTHER TESTS	SPT (blows/0.3m)
		Hard brown clayey sand TILL - with trace gravel - with frequent cobbles *(continued)*			HQ	25	810	-		
25					HQ	26	1065	-		
26					SS	27	500	38-61-82-50 / 50 mm		>>
27					SS	28	175	18-53-50 / 50 mm		>>
28					HQ	29	1090	-		
29–30	-28.66				HQ	30	1500	-		
31–35		End of borehole - standpipe installed ** The maximum undrained shear strength that can be determined with the pocket penetrometer used is 225 kPa. Readings exceeding this value have been reported as 225 kPa.								

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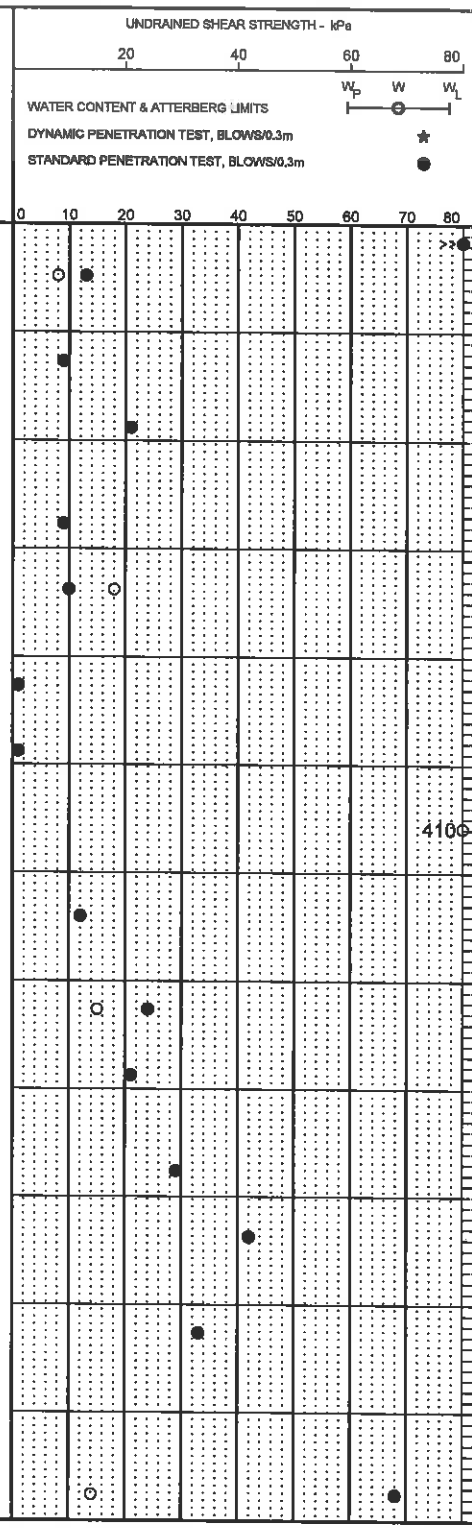
BOREHOLE RECORD

BH02

N: 5089433.41 E: 683716.25

CLIENT: STRAIT ENGINEERING LIMITED
PROJECT No. 193135
LOCATION: JOHN PAUL'S LANE BRIDGE, ESKASONI, NOVA SCOTIA
DATUM: CGVD2013
DATES: BORING 07/11/2019 TO 08/11/2019
WATER LEVEL: N/A
BH SIZE: HW/HQ

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)	OTHER TESTS
	1.66						
	1.51	ASPHALT	SS	1	25	50 / 25 mm	
		FILL: brown to grey poorly graded sand with silt and gravel	SS	2	200	6-6-7-7 (13)	
			SS	3	50	7-5-4-5 (9)	
			SS	4	100	8-8-13-13 (21)	
			SS	5	50	9-5-4-6 (9)	
			SS	6	175	9-8-2-22 (10)	
	-2.30	Very loose dark brown to grey silty sand with frequent organics - with trace shell fragments	SS	7	0	7-2-1-0 (1)	
		- with wood at 5.0 m	SS	8	150	0-0-1-15 (1)	
	-4.13	Compact to very dense reddish-brown well-graded sand with silt to silty sand TILL - with trace gravel	ST	9	50	PUSH	
			SS	10	175	9-6-6-12 (12)	
			SS	11	275	17-12-12-12 (24)	S
			SS	12	250	12-9-12-16 (21)	
			SS	13	325	12-15-14-18 (29)	
			SS	14	325	14-20-22-30 (42)	
			SS	15	225	13-14-19-24 (33)	
			SS	16	400	25-29-39-32	S



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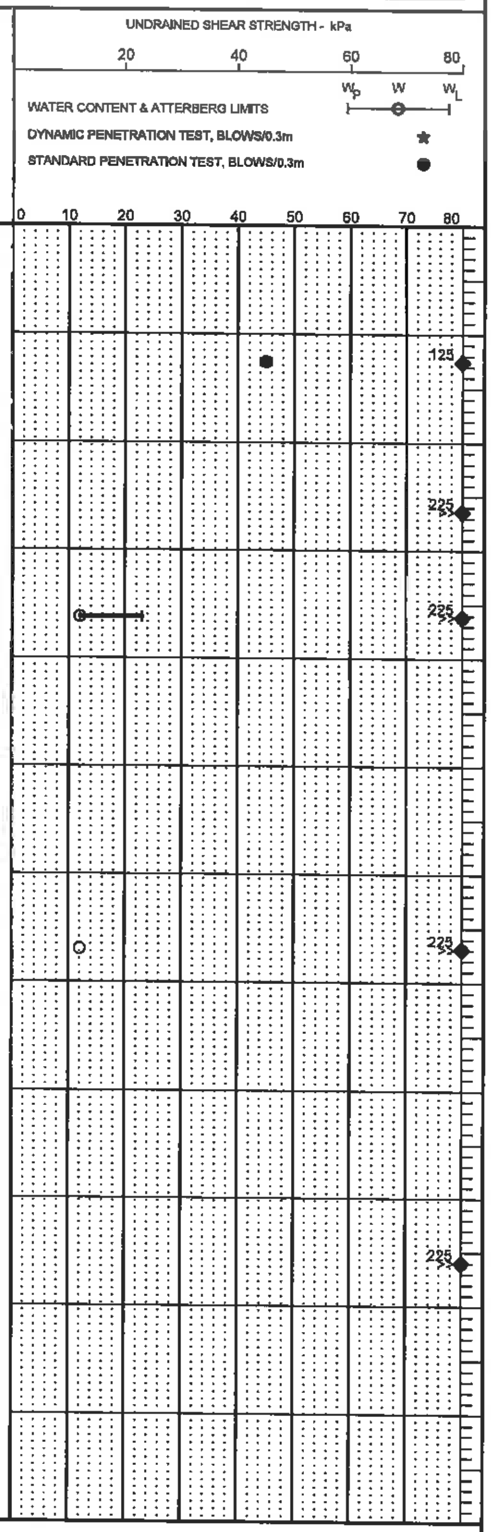
BOREHOLE RECORD

BH02

N: 5089433.41 E: 683716.25

CLIENT: STRAIT ENGINEERING LIMITED
PROJECT No.: 193135
LOCATION: JOHN PAUL'S LANE BRIDGE, ESKASONI, NOVA SCOTIA
DATUM: CGVD2013
DATES: BORING 07/11/2019 TO 08/11/2019
WATER LEVEL: N/A
BH SIZE: HW/HQ

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)	OTHER TESTS
		Compact to very dense reddish-brown well-graded sand with silt to silty sand TILL - with trace gravel *(continued)*				(68)	
13	-11.29	Hard brown clayey sand TILL - with trace gravel - with frequent cobbles	SS	17	550	14-16-29-31 (45)	
14–15			SS	18	300	25-58-50 / 75 mm	
15–16			SS	19	350	62-74-50 / 100 mm	
18–19			SS	20	425	12-35-64-50 / 100 mm	S
21–22			SS	21	175	34-68-50 / 75 mm	



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HARBOURSIDE Geotechnical Consultants

BOREHOLE RECORD

BH02

N: 5089433.41 E: 683716.25

CLIENT	STRAIT ENGINEERING LIMITED
LOCATION	JOHN PAUL'S LANE BRIDGE, ESKASONI, NOVA SCOTIA
DATES: BORING	07/11/2019 TO 08/11/2019
WATER LEVEL	N/A
PROJECT No.	193135
DATUM	CGVD2013
BH SIZE	HW/HQ

UNDRAINED SHEAR STRENGTH - kPa: 20, 40, 60, 80

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m

STANDARD PENETRATION TEST, BLOWS/0.3m

DEPTH (m)	ELEVATION (m)	SOIL/BEDROCK DESCRIPTION	TYPE	NUMBER	REC. SOIL (mm)	BLOWS / 150 mm (N VALUE)	OTHER TESTS
		Hard brown clayey sand TILL - with trace gravel - with frequent cobbles *(continued)*	GB	22	350	-	
			SS	23	350	12-15-50 / 100 mm	
			SS	24	250	24-50 / 100 mm	
			HQ	25	1245	-	
			HQ	26	1500	-	
	-28.95		SS	27	200	40-50 / 50 mm	
		End of borehole ** The maximum undrained shear strength that can be determined with the pocket penetrometer used is 225 kPa. Readings exceeding this value have been reported as 225 kPa.					

Depth scale: 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35

Plot scale: 0, 10, 20, 30, 40, 50, 60, 70, 80

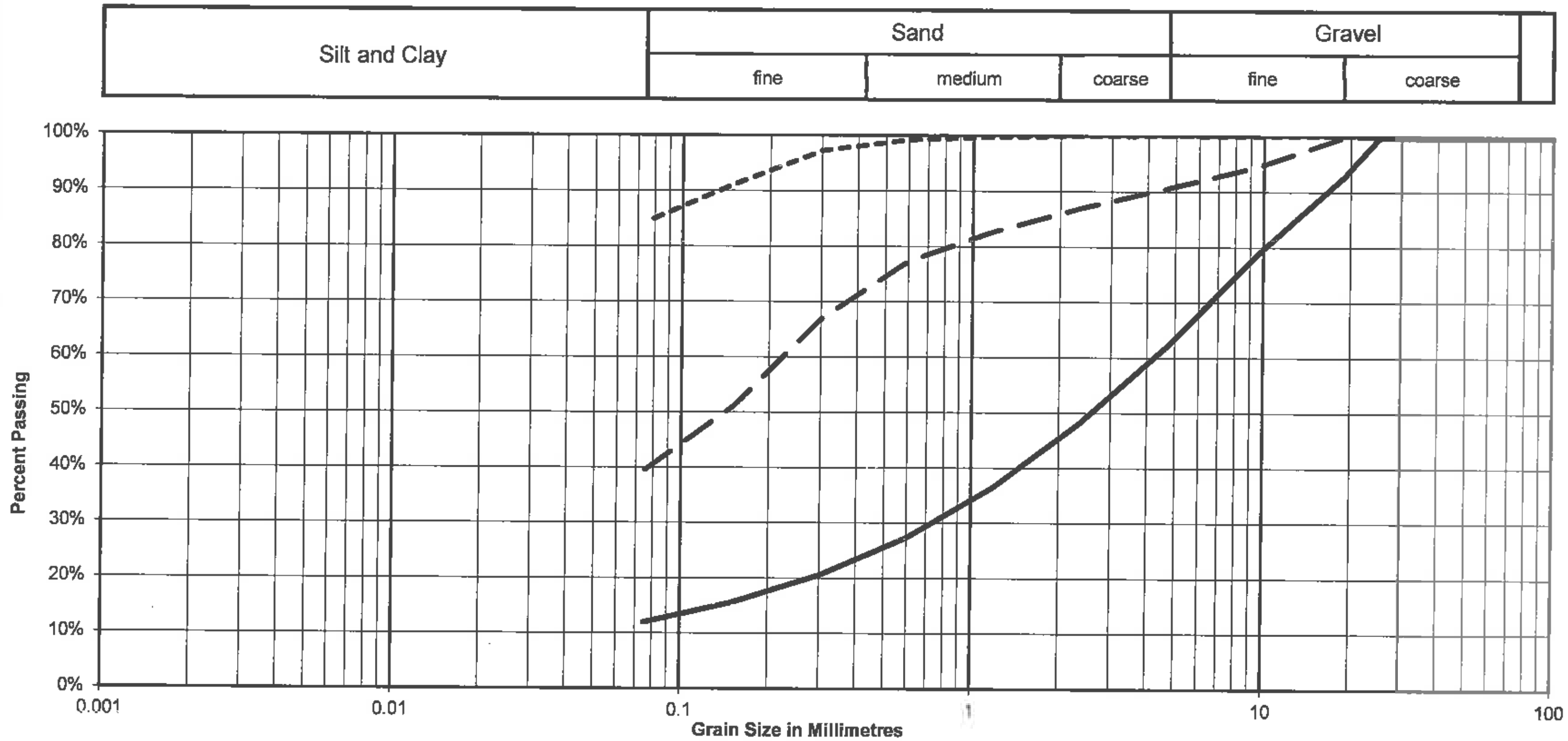
Plot annotations: 225 (GB 22), >> (SS 23), 225 (SS 24), >> (SS 27)

APPENDIX B

Laboratory Testing Results

Checked: DJW

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH01	SS3	0.97 to 1.57	38%	50%	12%	Poorly graded SAND with silt and gravel
— — -	BH01	SS7	4.65 to 5.26	9%	52%	39%	Clayey SAND
- - - -	BH01	SS9	6.15 to 6.76	0%	16%	84%	Lean CLAY with sand

HGC

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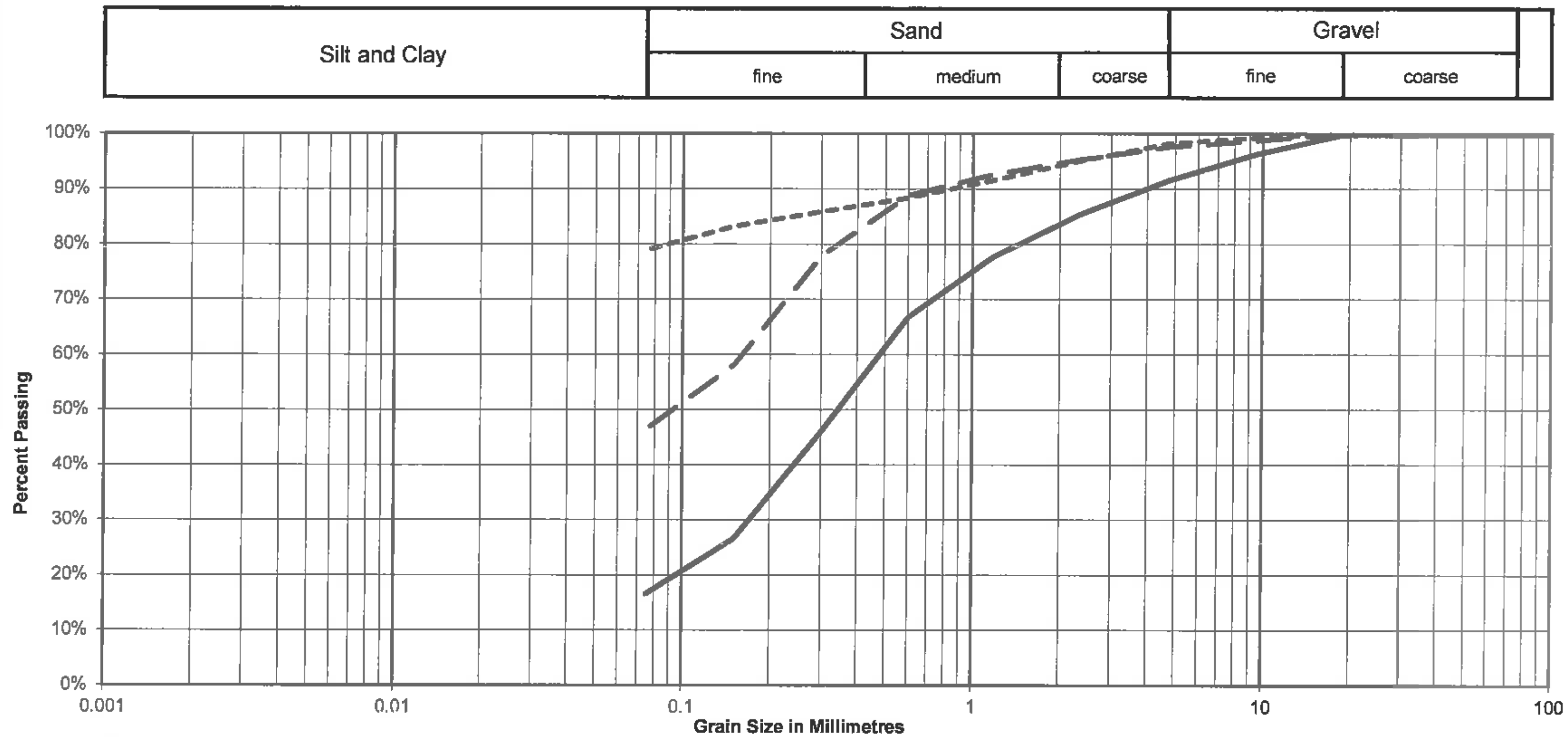
Dartmouth, NS B2X 2C3
http://harboursideengineering.ca

CLIENT Strait Engineering Limited
PROJECT John Paul's Lane Bridge
LOCATION Eskasoni, NS

PROJECT No.: 193135

Checked: DJW

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
———	BH01	SS14	10.03 to 10.64	8%	76%	16%	Silty SAND
— — -	BH01	SS18	15.39 to 16.00	2%	52%	46%	Clayey SAND
- - - -	BH01	GB22	20.50 to 20.85	2%	19%	79%	Lean CLAY with sand

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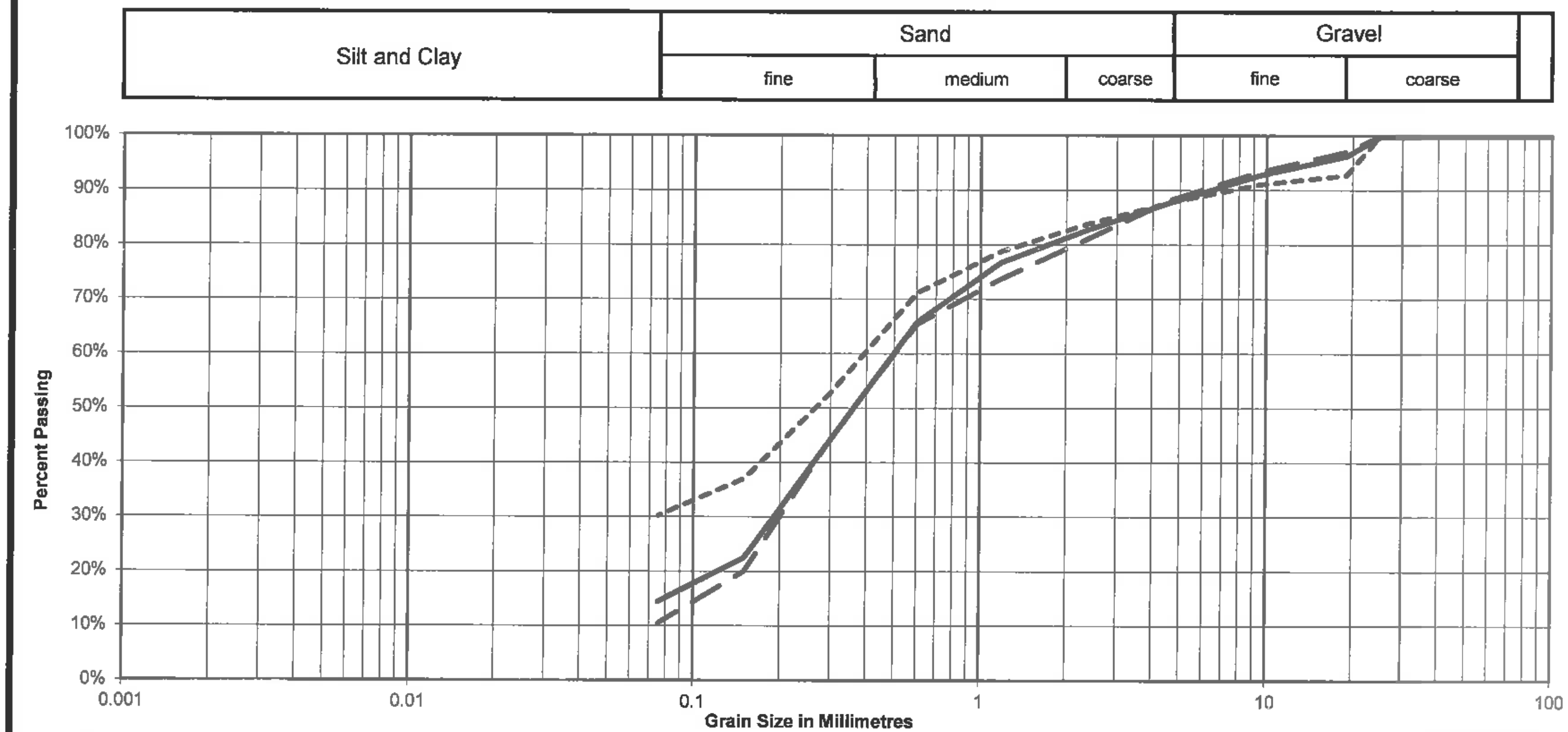
CLIENT	Strait Engineering Limited
PROJECT	John Paul's Lane Bridge
LOCATION	Eskasoni, NS

PROJECT No.: 193135

Checked: DJW

GRAIN SIZE DISTRIBUTION



CURVE	BOREHOLE / TESTPIT	SAMPLE	DEPTH (m)	SOIL FRACTION			SOIL DESCRIPITION
				GRAVEL	SAND	SILT/CLAY	
——	BH02	SS11	6.95 to 7.56	12%	78%	10%	Well-graded SAND with silt
— — -	BH02	SS16	11.50 to 12.10	12%	74%	14%	Silty SAND
- - - -	BH02	SS20	18.40 to 18.90	12%	58%	30%	Clayey SAND

HGC

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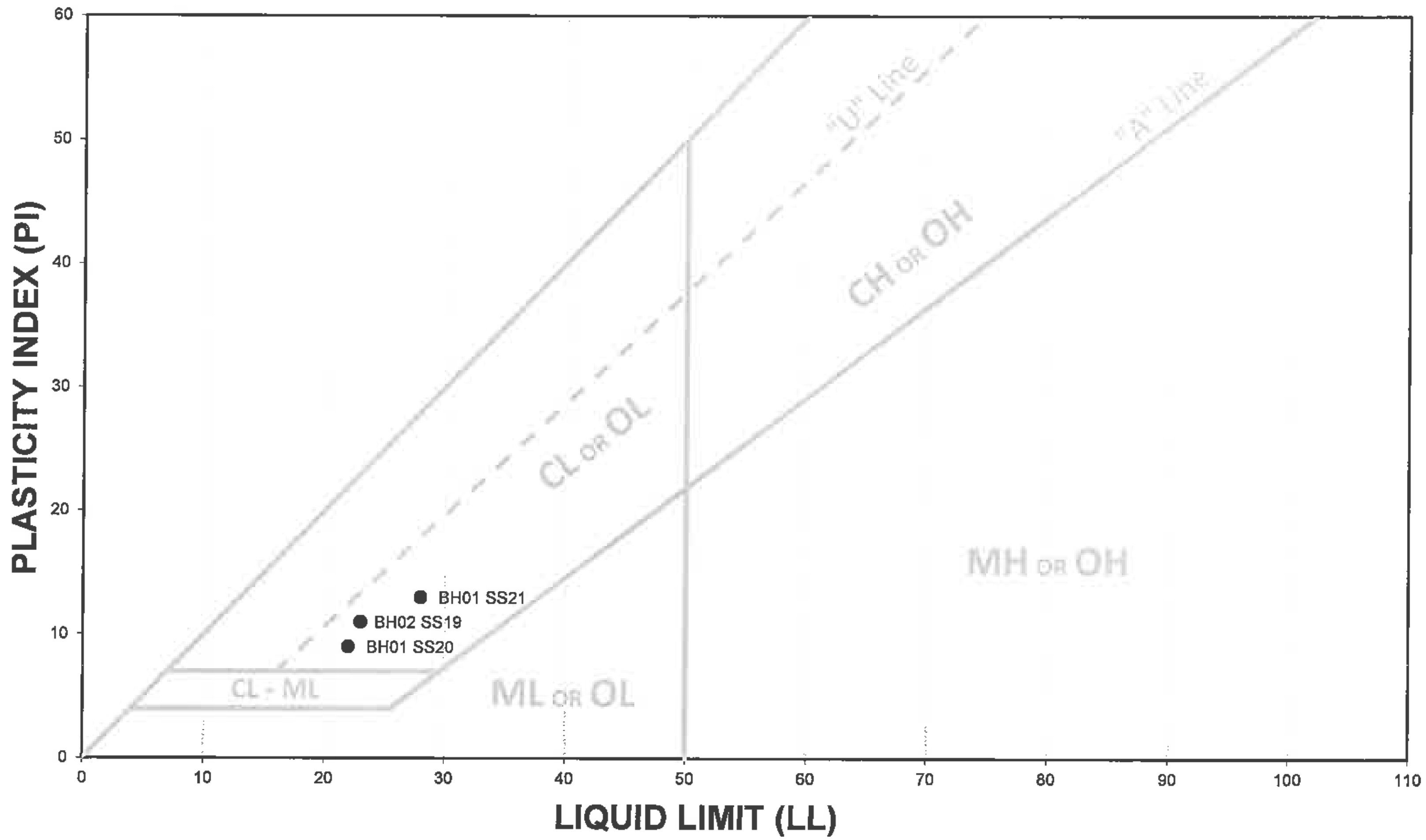
CLIENT Strait Engineering Limited
PROJECT John Paul's Lane Bridge
LOCATION Eskasoni, NS

PROJECT No.: 193135

Checked: DJW

ATTERBERG LIMITS



HGC

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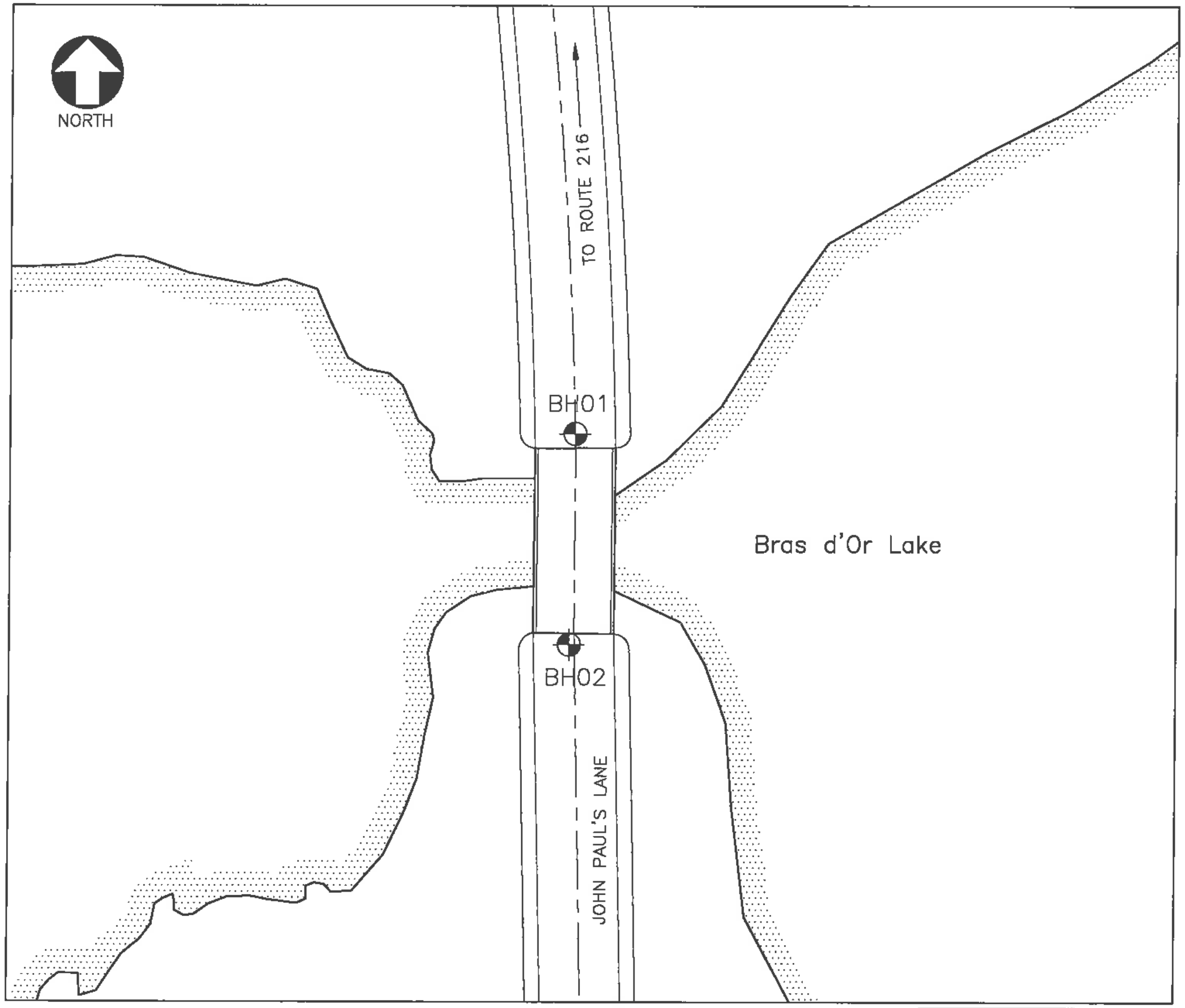
Dartmouth, NS B2X 2C3
http://harboursideengineering.ca

CLIENT	Strait Engineering Limited
PROJECT	John Paul's Lane Bridge
LOCATION	Eskasoni, NS

PROJECT No.: 193135

APPENDIX C

Sketch G01 – Borehole Location Plan



SITE PLAN

N.T.S.

KEY PLAN

BOREHOLE COORDINATES			
BOREHOLE #	NORTHING	EASTING	ELEVATION
BH01	5089447.39	683716.59	1.717m
BH02	5089433.41	683716.25	1.661m

NOTE:
ALL COORDINATES AT (UTM)
NAD 83 CSRS (2010)

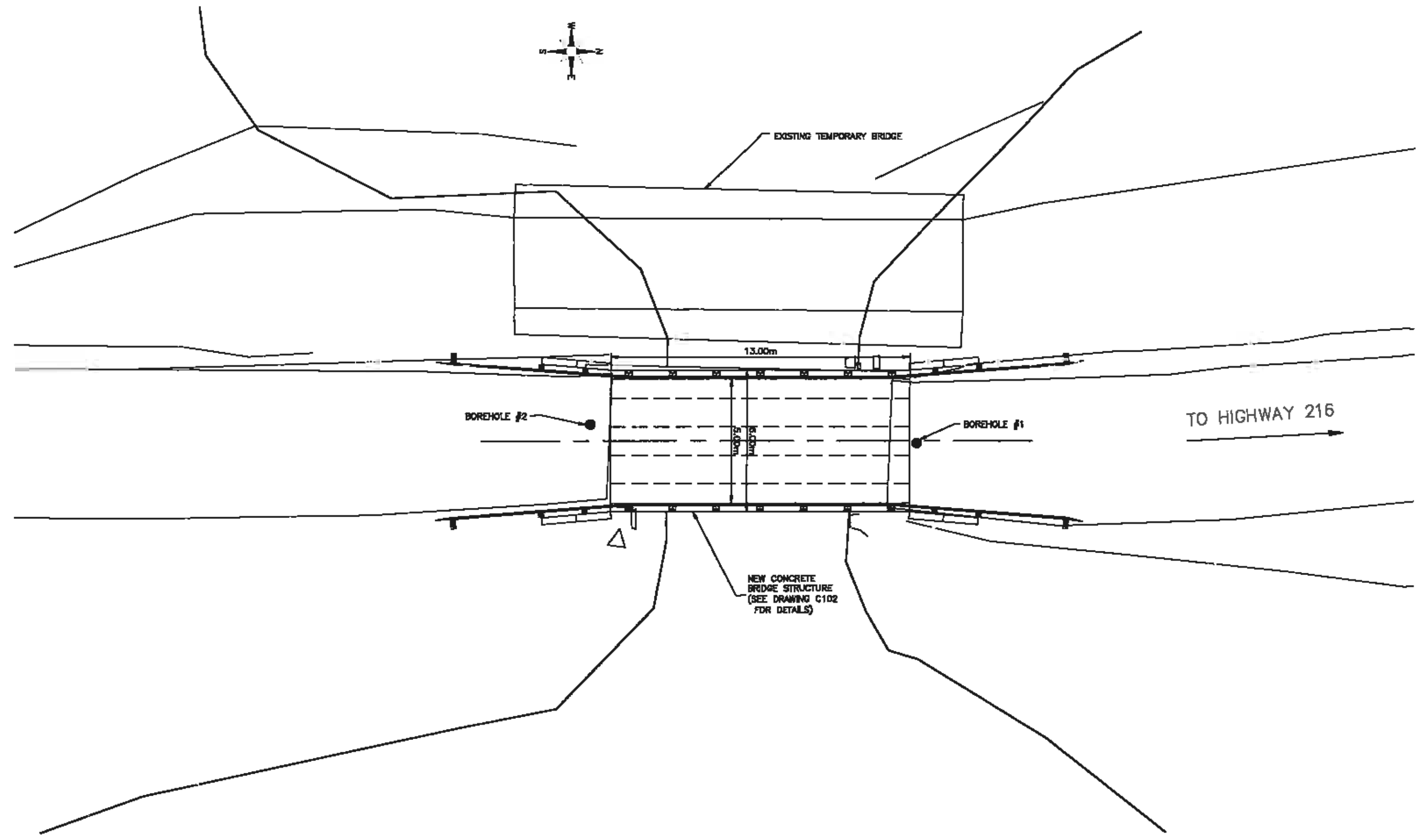
Scale	Date	Drawn	Designed	Checked	Approved	Contract
N.T.S.	NOVEMBER 2019	W. MORROW	D. WHEELER	T. MENZIES	T. MENZIES	193135

HARBOURSIDE Geotechnical Consultants

JOHN PAUL'S LANE BRIDGE
ESKASONI, N.S.
BOREHOLE LOCATION PLAN

SKETCH No.
G01

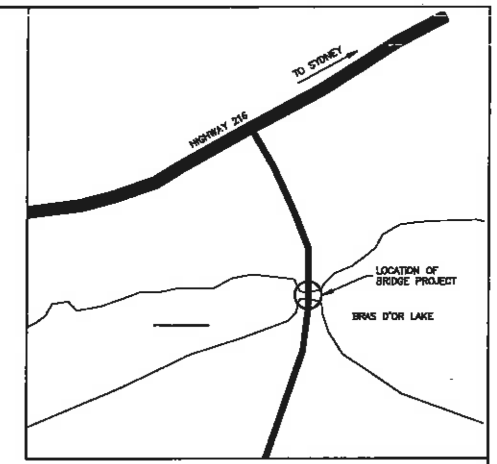
Z:\HARBOURSIDE_GEOTECHNICAL_CONSULTANTS\PROJECTS\193XXX\193135_JOHN_PAULS_BRIDGE_ESKASONI\9_DRAFTING\HGC\193135-G01.DWG, November 20, 2019, 9:17 AM



SITE PLAN

SCALE: 1:100

0 4 8 Meters



LOCALITY PLAN

SCALE: 1:20000

NO.	DATE	REVISIONS	DWG.	CHK.
1	[illegible]	CONCEPTUAL DESIGN ISSUED FOR REVIEW	GAU	GAU

STRAIT ENGINEERING LIMITED

PORT HAWKESBURY, NOVA SCOTIA

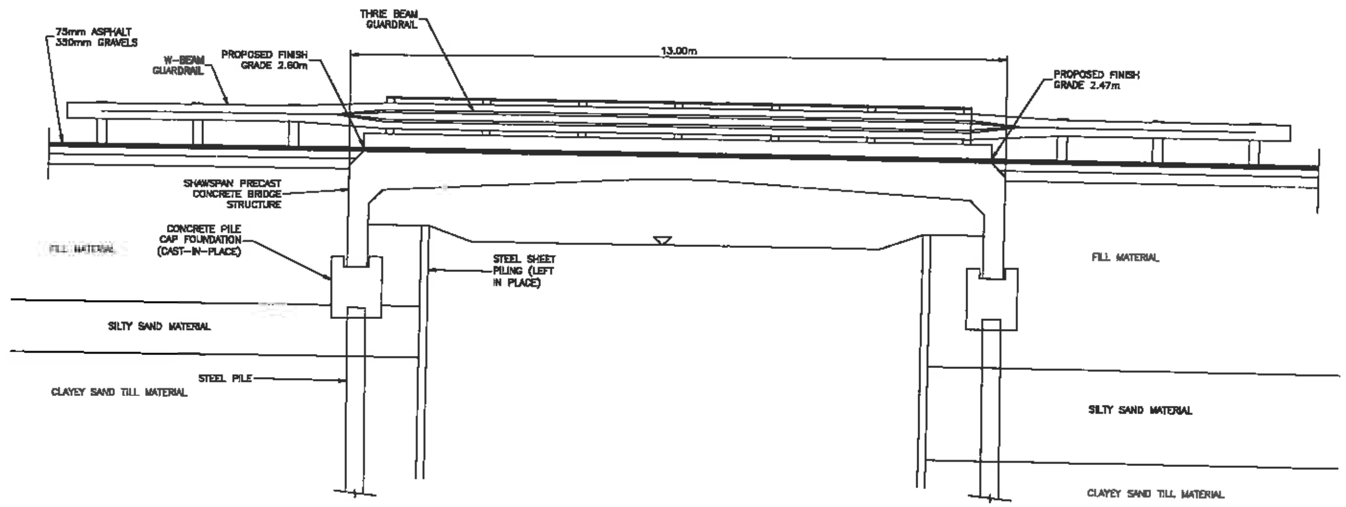
CLIENT:

ESKASONI FIRST NATION

PROJECT:

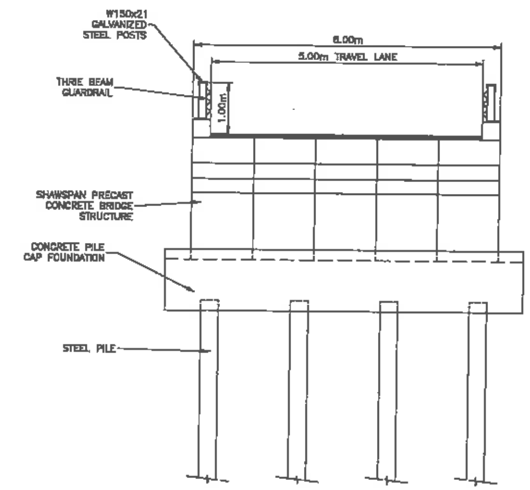
JOHN PAULS LANE
NEW CONCRETE BRIDGE

DR. GAU CH. GAU APP. HBM SCALE AS NOTED DATE JAN. 2020	LOCALITY AND SITE PLAN	PROJECT 19-39 DRAWING C101



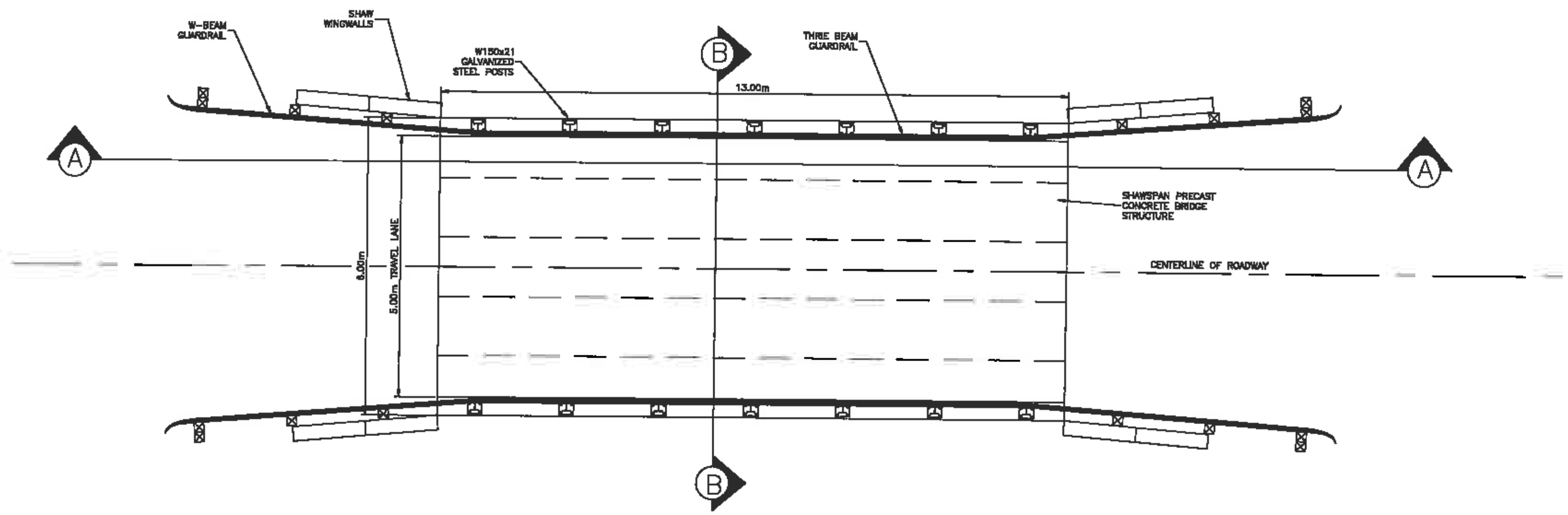
SECTION A–A

SCALE: 1:50



SECTION B–B

SCALE: 1:50



BRIDGE PLAN

SCALE: 1:50

NO.	DATE	REVISIONS	DWG.	CHK.
1		CONCEPTUAL DESIGN ISSUED FOR REVIEW	GAU	GAU

STRAIT ENGINEERING LIMITED

PORT HAWKESBURY, NOVA SCOTIA

CLIENT:

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PROJECT:

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JAN. 2020

BRIDGE PLAN AND SECTIONS

PROJECT 19–39

DRAWING C102

APPENDIX A