Stantec Consulting Ltd.
115 Harrisville Boulevard, Moncton NB E1H 3T3

February 28, 2018
File: 121619795.400

Attention: Mr. Nathan LeBlanc, P.Eng
Crandall Engineering Ltd.
1077 St. George Boulevard, Suite 400
Moncton NB E1E 4C9

Dear Mr. LeBlanc,

Reference: Final Geotechnical Investigation Report
Lorne Street Storm Water Mitigation – Phase 2
Tantramar Marsh, Town of Sackville, New Brunswick

1. INTRODUCTION

As requested, Stantec Consulting Ltd. (Stantec), completed a geotechnical investigation for the proposed new storm water retention ponds and outfall ditching as part of the storm water mitigation plan. The purpose of the geotechnical investigation was to obtain subsurface information related to the construction and design of two new storm water retention ponds and outfall ditching to the new outfall structure at the Tantramar River. Our services were completed in general accordance with proposal dated November 22, 2017 and submitted under file: 121619795, as well as the additional services authorized by email on January 24 and January 31, 2018. This report supersedes our previously submitted report dated January 11, 2018.

Use of this report is subject to the attached Statement of General Conditions. It is the responsibility of Crandall Engineering Ltd. (Crandall), identified as "the Client" within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec should conditions not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying or unexpected site conditions
- Planning, design or construction

2. SITE AND PROJECT DESCRIPTION

The proposed project site is located in southeastern Sackville, New Brunswick as identified on the attached Drawing No. 1 – Site Location Plan and Drawing No. 2 – Borehole Location Plan. It involves a new storm water retention pond, a future storm water retention pond, and outfall ditching to the Tantramar River.

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The proposed new storm water retention pond is to be constructed southeast of the intersection of Lorne Street and St. James Street. The area is low-lying marsh land, presently covered with low vegetation and bordered by St. James Street to the north, Charles Street to the east, a Canadian National (CN) railway to the south, and Lorne Street to the west.

We understand the outfall ditching will be constructed east of the pond along the north side of the existing CN railway, across Charles Street, the CN railway, Industrial Drive and Crescent Street before discharging at a new outfall at the Tantramar River.

A future storm water retention pond is proposed on vacant land to the south of the CN railway, between Charles Street and Industrial Drive. The area is presently vacant and covered with low vegetation.

3. SUBSURFACE INVESTIGATION PROGRAM

3.1.1. Geotechnical Investigation Program

The proposed borehole investigation program included four boreholes (BH-301 to 304) for the proposed storm water retention pond, two boreholes (BH-305 and 307) for the CN railway crossing, two boreholes for the future storm water retention pond (BH-310 and 311), and four boreholes (BH-312 to 315) for the outfall ditching and aboiteau structure. BH-315 was not completed due to access restrictions at the time of drilling and BH-316 was added east of Crescent Street within the proposed outfall ditching. Two companion boreholes (BH-301A and 304A) were advanced near BH-301 and 304 to complete in-situ shear vane testing and obtain Shelby Tube samples for future analysis. Two boreholes (BH-110 and 204) completed in the initial investigation program for Lorne and St. James Streets reconstruction on January 4, 2017 are also included for reference. Three boreholes (BH-306, 308 and 309) were deleted from the program during the proposal stage as information from the previous and current investigation was considered to be sufficient by Crandall and Stantec.

The 14 boreholes were advanced from December 1 to 5, 2017 and February 1, 2018 with a rubber-track-mounted drill rig provided by MEG Drilling Ltd. of Killam Mills, New Brunswick. The boreholes were advanced to depths ranging from approximately 5.49 meters to 17.88 meters below existing grades at the locations shown on Drawing No. 2 – Borehole Location Plan. A standpipe piezometer was installed at BH-302 for future groundwater monitoring. The borehole locations were surveyed and staked in the field by Crandall, or established using a hand-held global positioning system unit by Stantec. The borehole elevations, except for BH-301A, 304A and 314, were provided by Crandall and are included on the borehole logs. Elevations (El.) at BH-301A and 304A were interpolated to match those at BH-301 and 304, respectively, due to their close proximity, and BH-314 was interpolated from preliminary design drawings provided by Crandall.

Personnel from our Moncton office monitored the drilling program, collected soil and bedrock samples, and logged the subsurface conditions encountered at the borehole locations. The sampling conducted is provided on the attached Borehole Records.

The overburden soils at the borehole locations were sampled at close intervals using Standard Penetration Test (SPT) techniques with a 50-millimetre outside-diameter split-barrel sampler and by

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Shelby tube sampling. Field shear vane tests were also performed at select intervals. SPT N-values were recorded for each split-barrel sample obtained. The sampling and testing methods were completed in general accordance with the standard test method American Society for Testing and Materials (ASTM) D1586 (*Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*). The determination of soil relative density and consistency as indicated on the Borehole Records, are based on the results of the SPT. Detailed descriptions of the soils encountered, the sampling, and the testing carried out, are given on the attached Borehole Records.

Upon encountering split-barrel sampler refusal in BH-312 and 314, bedrock samples were retrieved in general accordance with ASTM D2113 (*Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration*). Bedrock was inferred at the remaining locations by means of split-barrel sampling and/or auger performance.

Soil samples were stored in moisture tight bags and returned to our Moncton laboratory for further classification and testing. Bedrock cores were stored in wooden boxes. If requested, the samples may be kept in storage for a period of three months from the date of issuance of this report, otherwise the samples will be discarded.

3.1.2. Falling Head Permeability Testing

Falling head permeability testing was completed in the field at a central location for each of the proposed ponds. The test locations identified as FHP-1 and FHP-2, are shown on Drawing No. 2. The testing was completed within HW-sized (76.2-millimeter-inside-diameter) casing. Each test was completed within the native lean CLAY (CL) deposit. FHP-1 was completed at a depth of 2.1 meters (El. 3.45 meters) and results indicate a hydraulic conductivity of approximately 3.2×10^{-6} centimeters per second (cm/s). FHP-2 was completed at a depth of 2.4 meters (El. 2.80 meters) and results indicate a hydraulic conductivity of approximately 5.1×10^{-6} cm/s.

3.2. SITE GEOLOGY

A surficial geology map, prepared by the New Brunswick Department of Natural Resources and Energy indicates the presence of two soil deposits at the site. The predominant deposit consists of intertidal plains and salt marshes and is associated with the Holocene epoch. This deposit is generally found to consist of clay, silt, fine sand, minor peat, and organic sediment. The second deposit at the site is associated with the late Wisconsinan glaciation and consists of hummocky, ribbed and rolling ablation moraines. The deposits generally include loamy ablation till, some lodgment till, minor silt, sand, gravel and boulders. Bedrock mapping, prepared by the New Brunswick Department of Natural Resources and Energy indicates bedrock at the site consists of moderately indurated siliceous sedimentary rocks from the late carboniferous group.

3.3. SUBSURFACE CONDITIONS

The subsurface conditions observed at the borehole locations are described in detail on the Borehole Records and are summarized herein. The attached *Symbols and Terms used on Borehole and Test Pit Records* provide a brief explanation of the terminology and graphics used by Stantec.

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Soil classification was based on the procedures described in ASTM D 2487 (*Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*) and ASTM D 2488 (*Standard Practice for Description and Identification of Soils, Visual-Manual Procedure*).

The subsurface conditions encountered at the borehole locations were variable and include:

- ORGANIC SOIL
- Lean CLAY (CL)
- Fat CLAY (CH)
- PEAT
- Poorly graded SAND (SP-SC) with clay to poorly graded SAND (SP-SM) with silt and gravel
- SILTY SAND (SM)
- CLAYEY SAND (SC) to CLAYEY SAND (SC) with gravel
- Sandy lean CLAY (CL)
- Inferred sedimentary BEDROCK
- SANDSTONE
- LIMESTONE

3.3.1. ORGANICS SOIL

Organic soil was observed at the ground surface and ranged from 50 to 100 millimeters in thickness. The soil was described as very soft, brown to black ORGANIC SOIL.

3.3.2. Lean CLAY (CL)

Lean CLAY (CL) was typically observed below the ORGANIC SOIL and varied in thickness. The lean CLAY (CL) ranged in color from light brown to light grey to grey and contained frequent organics. PEAT layers were also observed within the deposit. N-values from SPT performed within the deposit ranged from 0 to 4, indicating a very soft to soft consistency. One sample submitted for laboratory testing indicated a liquid limit of 37 percent and a plastic limit of 21 percent. Gradation analysis indicated it contained 75 percent silt and 25 percent clay. The natural moisture content on tested samples ranged from 47 to 57 percent with an average of 52 percent.

3.3.3. Fat CLAY (CH)

Fat CLAY (CH) was observed below the ORGANIC SOIL at BH-307 and was approximately 5.5 meters thick. The fat CLAY (CL) ranged in color from grey to light grey and contained some to frequent organics. N-values from SPT performed within the deposit ranged from 0 to 3, indicating a very soft to soft consistency. Two samples submitted for laboratory testing indicated a liquid limit of 68 to 70 percent and a plastic limit of 30 to 31 percent. Gradation analysis indicated it contained 89 to 100 percent silt and clay, 0 to 9 percent sand, and 0 to 2 percent gravel. The natural moisture content on tested samples ranged from 50 to 81 percent with an average of 66 percent.

3.3.4. PEAT

Layers of PEAT were observed within the lean CLAY (CL) deposit at BH-301, 302, 304, 305, 307, 311 and 314. The peat was brown to dark brown to black in color and is classified as highly organic soil (OL/OH). The thickness and depth to peat varied at the borehole locations. The moisture content on two samples submitted for testing ranged from 284 to 571 percent.

3.3.5. Poorly graded SAND (SP-SC) with clay to poorly graded SAND (SP-SM) with silt and gravel

A layer of poorly graded sand was encountered within and below the lean CLAY deposit at BH-301 and 302. The soil was light brown to brown in color and is classified as poorly graded sand with clay to poorly grade sand with silt and gravel. N-values performed within the deposit ranged from 3 to 16, indicating a very loose to compact condition.

3.3.6. SILTY SAND (SM)

A layer of light grey to greyish white SILTY SAND (SM) was observed at BH-311 and 313. N-values performed within the deposit ranged from 1 to 2, indicating a very loose condition.

3.3.7. CLAYEY SAND (SC) to CLAYEY SAND (SC) with gravel

A deposit of CLAYEY SAND to CLAYEY SAND with gravel was encountered at variable depths and thicknesses at 7 of the 11 borehole locations. The clayey sand was observed below the lean clay at BH-303 to 305, and 310, above the bedrock at BH-311 and 314, and near the surface of BH-312 and 313. N-values performed within the layer generally ranged from 11 to refusal, with an average of 20, indicating a compact to dense, but predominantly compact condition.

Samples were submitted for grain size analysis and moisture content testing. Test results are presented in Table 1 and on the attached Gradation Curves, and Borehole Records. The moisture contents of the select samples ranged from 15 to 22 percent with an average of 17 percent.

Table 1: Laboratory Testing Results

Borehole ID - Sample ID	Depth (m)	Moisture Content (%)	Gravel (%)	Sand (%)	Silt/Clay (%)
BH-310 – SS3	1.8	14.6	1.2	65.5	33.3
BH-311 – SS4	3.1	15.3	11.4	58.6	30.3
BH-316 – SS5	4.3	21.5	11.4	47.5	41.1

3.3.8. Sandy Lean CLAY

Sandy lean CLAY (CL) was observed at BH-302, 312, 313, 314 and 316 at varying depths and thicknesses. The deposit ranged in color from reddish brown to grey and is classified as sandy lean CLAY (CL) to sandy lean CLAY (CL) with gravel. Trace to frequent organics were observed within the deposit. N-values from SPT performed within the deposit ranged from 7 to 48, with an average of 21, indicating a stiff to hard consistency.

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Gradation analysis completed on one sample indicated it contained 35 percent sand and 65 percent fines (silt/clay sized particles). The natural moisture content on three samples ranged from 14 to 22 percent with an average of 17 percent.

3.3.9. SEDIMENTARY BEDROCK

Inferred sedimentary bedrock was observed at BH-303, 310, 311, 312, 313, and 314 at depths ranging from approximately 1.2 to 14.8 meters below existing grades. The bedrock was generally inferred by split-barrel sampler refusal and/or auger performance and was confirmed by coring at BH-312 and BH-314. The inferred bedrock consisted of mudstone at BH-303, and sandstone at the remaining borehole locations.

Where cores were retrieved, the bedrock consisted of excellent quality SANDSTONE, overlying good to excellent quality LIMESTONE. The SANDSTONE rock mass is described as light grey to brown to reddish brown, slightly weathered, with discontinuities spaced very close to moderate, with a dip angle between 0 and 5 degrees. The LIMESTONE rock mass is described as reddish brown, fresh to slightly weathered, with close to wide joint spacing with a dip angle between 5 and 25 degrees. One sample of the limestone was submitted for unconfined compressive strength testing and results indicate a strength of 52.4 megapascals.

3.3.10. Groundwater

Groundwater was observed at each borehole location at depths ranging from approximately 0.6 to 4.6 meters below the existing ground surface at the time of drilling. A groundwater monitoring well was installed at BH-302, however, a site visit for monitoring has not been completed. Please note that groundwater levels observed over a short duration may not be representative of the actual site conditions. Groundwater levels can be expected to fluctuate during periods of heavy precipitation associated with seasonal weather trends, tidal fluctuations, or a particular event, site use, adjacent site use, and construction activity.

3.4. LABORATORY SHEAR VANE TESTING

A hand-held shear vane was used to test select Shelby tube samples obtained within the lean to fat clay. Test results are summarized in Table 2.

Table 2: Shear Vane Test Results

Borehole ID - Sample ID	Depth (m)	Shear Strength (kPa)
BH-304A – ST1	0.3	4.7
BH-304A – ST1	0.9	15.7
BH-304A – ST3	1.5	7.8
BH-314 – ST9	6.1	16.7

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4. DISCUSSION AND RECOMMENDAIONS

4.1. PROPOSED STORM WATER RETENTION POND (LORNE AND ST. JAMES STREETS)

As previously discussed, a storm water retention pond is currently proposed southeast of the intersection of Lorne and St. James Streets (BH-301 to 304). We understand preliminary design of the proposed new storm water retention pond involves the construction of two sediment ponds and associated storm water pond channels. A central island is proposed between the channels. We understand the island and land adjacent to the channels is to remain at or near the existing grades, with the pond bottom (EL. 2.51m) established by excavating approximately 3 meters to achieve the design grade. We further understand a minimum water elevation of 3.51 meters is proposed to meet environmental requirements. The capacity of the proposed storage pond was not provided.

4.1.1. Subsurface Conditions and Construction Considerations

Deep layers (up to approximately 7.5 meters in thickness where fully penetrated) of very soft to soft lean CLAY (CL) were observed in three of the four boreholes (BH-301, 302 and 304) advanced in the area of the proposed storm water retention pond. Frequent organics and peat layers were also observed within the lean CLAY (CL) deposit and are anticipated to be reflective of the predominant soil condition within this area.

N-values ranging from 0 to 3 were typically measured within the lean CLAY (CL) and as such, it is unlikely to be able to support the ground pressures of conventional construction equipment. We anticipate that excavations within the soft areas will need to be completed by means of specialized equipment such as long reach excavation equipment or drag lines. Further, a platform or rock mattress may be required to support long reach excavation or drag line equipment. Winter construction may improve working conditions; however, specialized equipment is still anticipated when excavating the soft soils.

4.1.2. Haul Roads

We anticipate winter construction, haul roads and specialized equipment will be required throughout construction.

As a minimum, the design of haul roads should include a non-woven geotextile placed directly over the native layer of vegetation, followed by a bi-axial geogrid, and 600-millimeters of well-graded crushed rock with a nominal particle size of 150 to 200-millimeters. The crushed rock should be placed and compacted in lifts. The geotextile and geogrid should be placed in accordance with the manufactures specifications. We anticipate differential settlement of the haul road will occur and regular maintenance, including the placement of additional crushed rock, should be expected. Haul roads may need to be thickened in some areas to support construction equipment. Haul roads should not be constructed at the crest of excavations to avoid surcharging the slopes. As a minimum, haul roads should be constructed outside the zone of influence to the slope.

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4.1.3. Retention Pond Liner

We understand an Environmental Impact Assessment (EIA) is currently in progress and environmental concerns may require a minimum water level within the retention pond to avoid drawdown of the water table in adjacent wetlands.

As previously discussed, in-situ falling head permeability testing completed within the native lean clay deposit indicates a hydraulic conductivity of approximately 3 to 5 x 10^{-6} cm/s. A soil liner with a minimum soil thickness of 0.5 meters and an in-situ hydraulic conductivity of 1 x 10^{-7} cm/s or less, or a geosynthetic/synthetic liner are typically recommended for the purpose of storing water.

Due to the presence of very soft soils and high water table, it is unlikely that a soil liner could be installed. As previously noted, we do not anticipate the soft soils can support construction traffic, and adequate compaction of the soil liner is considered to be impractical above the soft soils. Additionally, sloughing of the soil liner is anticipated below the water table due to unbalanced hydrostatic pressure. If a liner is necessary, an underdrain or ballast system would also be required to alleviate hydrostatic pressures associated with the surrounding high water table.

Consideration may be given to excavating the pond area to the required floor elevation in the native lean clay and foregoing the pond liner. Considering the groundwater table is anticipated to be at or above the design static water level, based on initial water level readings, we anticipate that natural inflow of groundwater will help to maintain the water level during dry periods. We recommend a hydrogeologic assessment of the groundwater and seepage conditions to confirm this condition.

The native lean clay was observed to contain frequent organics and peat layers throughout. If these layers are present in the pond floor or side walls, they may act as a natural flow path in the liner.

4.1.4. Slope Stability and Erosion Control

Excavation slopes are anticipated to be located within the very soft lean clay throughout a large portion of the pond and drainage channel. The water table was also observed to be located within the slopes (elevation 3.5 to 4.1 meters). We anticipate relatively flat slopes will be required to maintain the long-term stability within the lean clay deposit.

Preliminary slope stability analysis was completed on a generic cross section with slopes ranging from 2H:1V (2 Horizontal to 1 Vertical) to 5H:1V. The analysis was completed for clay shear strengths varying from 2 to 10 kPa. Our analysis also considered a rapid draw down of the ground water level following a rainfall event to the proposed minimum water elevation (EL. 3.51 meters). The results indicate the undrained shear strength of the clay will need to be a minimum of 6 kPa to provide a marginal factor of safety against failure of the modeled slopes. Figure 1, attached, presents the results of our analysis and each slope's factor of safety variation with respect to the assumed clay strength.

N-values from SPT testing completed within the very soft to soft lean clay frequently ranged from 0 to 2, indicating a shear strength of less than 12.5 kPa. Field shear vane testing was completed at boreholes BH-301A, 304A and 314. Hand-held shear vanes testing was also completed on Shelby tube samples obtained from the above noted boreholes. The results of the shear vane testing indicated the strength of the very soft to soft clay ranged from 4.7 to 16.7 kPa.

Based on the shear strength results obtained from shear vane testing and our slope stability analysis, we cannot recommend a practical design slope to provide a marginal factor of safety against slope failure in all areas.

The native lean clay soils are anticipated to be highly susceptible to erosion and as such, erosion control measures are recommended within the retention pond and drainage channels. Conventional erosion control measures such as erosion mats, rip rap, or gravel blankets will be difficult or impractical to install over slopes constructed within the soft soils. This should be evaluated in conjunction with slope stability analyses when a final design section is available.

In the event an open channel is constructed without erosion protection, we anticipate a natural meandering of the excavated channels will occur. Additionally, natural slope failures resulting from the soft soils and erosion/scour are expected to occur. Although this is a low-cost option, if the channel meanders and poses a risk to nearby infrastructure, we anticipate it would be difficult to repair and costly to mobilize equipment until winter conditions are present.

Should placement of rip rap or a gravel blanket be desired, we recommended the slopes of the pond and channel be lined with a non-woven geotextile prior to placement of rip rap or gravel layer. The size and thickness of the rip rap and/or gravel blanket should be selected based on the maximum design flow. The rip rap and/or gravel blanket should also consider the effects of wave action which may occur within the retention pond. In areas where surface drainage is anticipated into the retention pond or channels, we recommend the rip rap and/or gravel blanket be extended horizontally at the top of slopes. The use of a geomat, such as the MacMat line of products by Maccaferri, may also be considered for erosion control.

The installation of cofferdams, such as sheet piles, may be needed to eliminate the above noted concerns associated with slope stability and erosion within the lean clay slopes.

4.1.5. Settlement

We anticipate concrete headwalls will be required at the inlet and outlet of the retention ponds. As previously discussed, soft compressible soils were observed at the locations of the proposed storm water retention ponds and are anticipated to remain in place below the concrete headwalls at the inlet and outlets. As previously noted, we understand the proposed pond design will establish the top of the pond at or near the existing grades, with the pond bottom established by excavating approximately 3 meters to achieve the design grade. Localized fill placement (i.e. existing ditch infill) of approximately 1 meter is anticipated in select areas, including near the inlet and outlet structures. Fill placement will result in a net stress increase to the underlying clay and peat layers, inducing settlement. Preliminary settlement estimates are based on the thickness of the compressible layers observed at the borehole locations and relationships cited in literature using the natural moisture content of soils. The preliminary estimate associated with the placement of 1 meter of fill is 360 millimeters, with primary consolidation taking greater than 21 months to achieve. The preliminary estimate is approximate and meant to provide an order of magnitude that might be expected (+/-). To establish a more accurate settlement estimate, laboratory consolidation testing is required. Additionally, the potential exists for rebound in the peat and clay layer associated with excavation of the retention pond channel.

Construction of headwalls are also anticipated to result in a net stress increase to the underlying clay and peat layers, induction settlement. Design of the storm water retention ponds may require the headwalls be pile supported and/or the use of flexible connections at the inlet and outlet structures. Pre-loading or surcharging the site may also be considered to induce settlement and decrease post-construction settlements.

4.2. FUTURE STORM WATER RETENTION POND (CHARLES STREET TO INDUSTRIAL DRIVE)

We understand a future pond may be constructed south of the existing CN rail line between Charles Street and Industrial Drive (BH-204, 307, 310 and 311).

Design details for the future pond are not presently available. Subsurface conditions observed at BH-307 indicate very soft to soft clay was observed to depths greater than approximately 6.1 meters and as such, many of the concerns noted in section 4.1 are applicable to the design and construction of the future pond. The very soft to soft clay ranged in thickness from 1.5 to 2.7 meters at BH-204, 310 and 311, consequently, over-excavation may be economical in this area. Additional test pits could be advanced within the area to assess the depth of very soft to soft soils and relocate the pond.

4.3. DITCHING

We understand excavation of up to approximately 6 meters is required for the construction of outfall ditching from the proposed storm water retention pond to the Tantramar River. As previously discussed, the subsurface conditions within the proposed excavated soil section are variable and range from very soft lean clay, to sandy lean clay, to clayey sand, to inferred bedrock.

As previously noted, stable slopes are not anticipated to be practical and they will be susceptible to erosion. Recommendations regarding slope stability and erosion concerns within the native lean clay are discussed in Section 4.1.4.

Excavation of stable slopes and placement of erosion control measures may be practical within the compact clayey sand and firm to hard sandy lean clay encountered at BH-312, 313 and 316.

We understand open-channel flow is desired, however, consideration could be given to a conventional piped storm water system.

4.4. ABOITEAU STRUCTURE

A new aboiteau structure is proposed to the north of the existing aboiteau structure (BH-314). A dyke was previously constructed in the area to El. 8.25 meters, however, we understand the grade will be raised to El. 10.0 meters for flood prevention. Slope stability analysis associated with raising the dyke elevation was not included in our scope of services.

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Preliminary drawings provided by Crandall indicate the proposed aboiteau structure involves two stacked pipes with inline check valves. The pipes are proposed with invert elevations of approximately 5.126 and 2.172 meters and span 35.0 and 55.0 meters, respectively.

4.4.1. Settlement

Subsurface conditions observed at BH-314 indicate the presence of soft, compressible soils to approximately El. -0.3 meters. Based on drawings provided by Crandall, we understand up to approximately 3 meters of fill will be required to raise the dyke to El. 10.0 meters. Fill placement will result in a net stress increase to the underlying clay and peat layers, inducing settlement. Additionally, the fill thickness required to achieve the dyke elevation is not uniform and as such, differential settlement is expected. Preliminary settlement estimates are based on Crandall Drawing No. 16196-3D-C19, the thickness of the compressible layers observed at the borehole location and relationships cited in literature using the natural moisture content of soils. The preliminary estimate associated with the placement of 3 meters of fill is 200 millimeters, with primary consolidation taking greater than 2 months to achieve. The preliminary estimate is approximate and meant to provide an order of magnitude that might be expected (+/-). To establish a more accurate settlement estimate, laboratory consolidation testing is required.

We anticipate that settlement of the stacked aboiteau structure is not desirable and as such, we recommend the aboiteau pipes be pile-supported. Settlement of the pipes supported on piles is anticipated to be negligible, however, the embankment fill is anticipated to settle around the pipes resulting in voids below the pipes and cutoff walls (or similar) would be needed along the length of the pipes. Pre-loading or surcharging the site may also be considered to induce settlement and decrease post-construction settlements. In addition, cut off walls will be required at the inlet and outlets of the pipes for scour protection.

Based on the preliminary design drawings provided by Crandall, we anticipate approximately 2.3 meters of compressible soils below the lowest proposed pipe invert elevation. As such, over-excavation may be economical. If this is considered to be a more practical and economical approach the sandy lean clay, lean clay, and peat should be over-excavated to the surface of the clayey sand (approximately EL. -0.3 meters). Structural fill to support the pipes should extend outward and downward at a rate of 1H:1V from the crown of the pipe. We anticipate the use of additional shoring, such as driven sheet piles, will be required to facilitate over-excavation of the compressible soils as well as placement and compaction of structural fill and water control. Structural fill should consist of impermeable soil at a moisture content suitable to facilitate compaction to 95 percent of the standard Proctor maximum dry density as determined in accordance with ASTM D698.

4.4.2. Shoring System Design

A temporary excavation shoring system, consisting of driven sheet piles, is anticipated during construction of the aboiteau structure. Recommended soil parameters for shoring system design are included in the following table.

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Table 3: Soil Parameters for Shoring System Design

Soil	Dry Unit Weight (kN/m^3)	Effective Friction Angle (°)	Effective Undrained Shear Strength (kPa)
Very soft to soft CLAY	18	0	5
Loose to compact Clayey SAND	19	28	0

4.5. UTILITY CROSSING

A utility crossing is proposed for the existing CN railway at BH-305 and 307. As previously discussed, subsurface conditions in the area consist of native deposits of lean CLAY (CL), fat CLAY (CH), PEAT, and CLAYEY SAND (SC) with gravel. Based on drawings provided by Crandall, the pipe is anticipated to be installed within the lean CLAY and PEAT layers. The consistency of the native soils at the proposed crossing range from very soft to soft. We understand the use of jack and bore technology where the cutting head does not extend beyond the casing is proposed. With proper installation technique, final grades are not anticipated to change from utility installation; However, the potential exists for construction related settlements that cannot be accurately predicted. Settlement should be monitored as discussed in Section 4.5.1.

4.5.1. Settlement Monitoring

The ground monitoring program should consist of settlement monitoring control points established prior to pipe installation for the rail crossing. We recommend the control points should be installed above the proposed casing alignment at 2-meter intervals on both sides of the rail crossing to a distance of 10 meters from the crossing. Similarly, along the rail lines, we recommend establishing control points on both rails at 2-meter intervals starting at the utility crossing and extending out a distance of 10 meters on either side. Control points should also be established in the rail bed centerline by driving or screwing steel pins through the ballast. Both angular movements and vertical settlement should be monitored for rails.

The monitoring program should begin by establishing baseline elevations prior to construction. During construction, the settlement points should be surveyed twice per day, once in the morning between 7am and 10am and again in the afternoon prior to end of day. Following the completion of construction, monitoring should continue daily for one week and then at least once per week for a period of five weeks. The results should be communicated daily to the contractor, engineer and CN. The settlement monitoring frequency may then be decreased if movements have ceased at the discretion of CN.

At any time during settlement monitoring if the total elevation or lateral movement exceeds 15 or 10 millimeters respectively, the work is to be suspended and the project site inspector should notify CN immediately and implement the contingency plan to reinstate the rails to original grade in accordance with CN protocol.

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The settlement monitoring program discussed herein should be reviewed and approved by CN prior to beginning construction.

4.6. Reuse of Existing Soil

We understand the project will generate excess fill and reuse of the excavated soils is desirable. Excavated ORGANIC SOIL, lean CLAY, fat CLAY, PEAT, SILTY SAND and sandy lean CLAY are not suitable for reuse as structural fill due to the presence of organics, high moisture content and/or uniform gradation; however, some of these soils may be acceptable in landscaped areas with low slopes and no traffic.

The excavated CLAYEY SAND is not generally recommended for reuse as structural fill due high moisture content and compaction may be difficult to achieve due to its uniform gradation. If necessary, portions of the CLAYEY SAND, particularly above the groundwater table, maybe be selectively reused as structural fill. It is recommended that the soil be reused in applications where a lower level of compaction (such as 95 percent of the standard proctor value) is acceptable. A sheep's-foot roller may be required to adequately compact CLAYEY SAND reused as structural fill.

If desired for reuse, adequate laboratory testing of stockpiled soils should be performed to determine whether the soil moisture content will be appropriate to achieve the required compaction. Effort should be made during excavation activities to separate and protect the stockpiles from precipitation. Excavated soils above optimum moisture content will only dry readily during warm and dry weather; therefore, reuse of excavated soils should be limited to late spring to early fall construction.

4.7. General Construction Considerations

The earthwork contractors must familiarize themselves with subsurface conditions prior to tender submission.

4.7.1. Bedrock Removal

As previously discussed, inferred mudstone, sandstone, and limestone were encountered at 5 of 11 boreholes across the site at depths ranging from approximately 1.2 to 5.5 meters below existing grades. Based on preliminary drawings provided by Crandall, we anticipate bedrock excavation may be required within the inferred mudstone at BH-303 and the inferred sandstone at BH-312 and BH-313. Where encountered, the bedrock was penetrable with standard drilling augers, and in some cases, the split-barrel sampler could be driven within the bedrock. Additional details regarding the quality of the bedrock and the testing completed are shown on the Borehole Records.

It may be possible to remove bedrock with large conventional excavation equipment. In the event bedrock cannot be removed by digging with heavy excavators upon review and approval by the project engineer, hydraulic hoe ramming may be performed to fracture rock prior to excavation. The earthworks contractor must familiarize themselves with the subsurface conditions prior to tender submission to determine their means of bedrock removal. It should be the responsibility of the earthworks contractor to determine the most efficient means of bedrock removal prior to tender submission.

4.7.2. Temporary Excavations

The minimum requirements of the New Brunswick Occupational Health and Safety Guidelines may not provide stable slopes or a safe working environment within the soft soils encountered at this site. Safe slopes and excavations are the responsibility of the contractor. If an excavation cannot be properly sloped or benched, the contractor should install an engineered shoring system to safely support the temporary excavation. Engineered shoring should be designed and approved for trench work.

Excavation slopes should be checked regularly for signs of instability and flattened as required. Soil stockpiles should not be located near the excavation to avoid surcharging the excavation walls.

4.7.3. Construction Dewatering

Based on the groundwater elevations encountered during the subsurface investigation program, we anticipate construction dewatering will be required during excavation activities and that it may be accomplished using pumps and cofferdams. The use of ditching directed to pump-equipped sumps may also be required. Lowering of the water table with sumps is unlikely to be effective due to the low permeability of the in-situ clay soils.

4.7.4. Environmental Considerations

As the project will involve construction within a wetland we anticipate the works to be subject to the stipulations set forth in the WAWA permit issued by the New Brunswick Department for the Environmental and Local Government (NBDELG). NBDELG stipulations are anticipated to include but not limited to siltation control, fueling equipment outside the wetland buffer, and decommissioning of haul roads. The earthworks contractor should review the requirements of the WAWA permit prior to tender submission.

**Reference: Final Geotechnical Investigation Report
Lorne Street Storm Water Mitigation – Phase 2
Town of Sackville, New Brunswick**

5. CLOSURE

This report was prepared by Corey MacPhee, P.Eng., and reviewed by Christopher McQueen, P.Eng., PE and Dan McQuinn, P.Eng. We trust the information contained in this report meets your current needs. Should you have questions about the contents of this report, or if we can be of further assistance, please contact the undersigned.

Regards,

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Attachment: Statement of General Conditions
Drawing No. 1 – Site Location Plan
Drawing No. 2 – Exploration Location Plan
Symbols and Terms used on Borehole and Test Pit Records
Borehole Records
Laboratory Testing
Figure 1 – Summary of Slope Stability Analysis

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STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd and the Client. Any use which a third party makes of this report is the responsibility of such third party.

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VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd that differing site or sub-surface conditions are present upon becoming aware of such conditions.

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