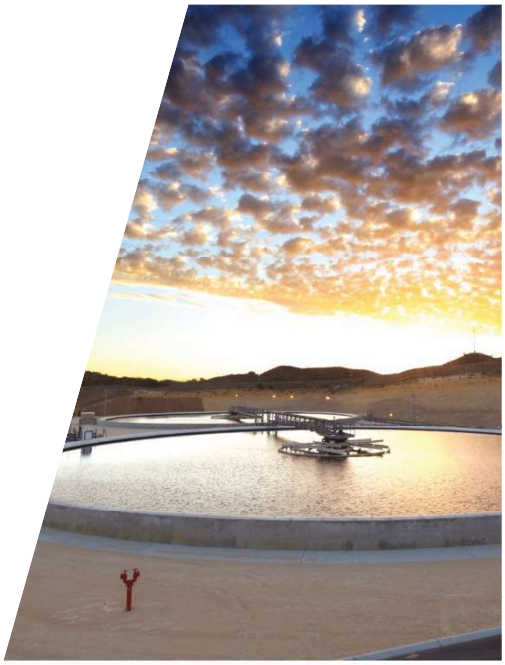


Geotechnical Investigation

Third Train Upgrade
Bruce Power LP Sewage Processing Plant

Bruce Power LP









Table of Contents

1. Introduction 1
2. Field and Laboratory Work Programs 1
   - 2.1 Field Work Program 1
   - 2.2 Laboratory Work Program 3
     - 2.2.1 Geotechnical Laboratory Testing 3
     - 2.2.2 Analytical Laboratory Analyses 3
     - 2.2.3 Single Well Response Test 3
     - 2.2.4 Sewer Use Sampling 4
3. Regional Geology 4
4. Subsurface Conditions 4
   - 4.1 Ground Cover 5
     - 4.1.1 Asphalt and Topsoil 5
     - 4.1.2 Fill Sand and Gravel Deposits 5
   - 4.2 Native Soil 5
     - 4.2.1 Sandy Silt/ Silty Sand/ Silty Sand Till/ Sandy Silt Till Deposits 5
   - 4.3 Groundwater 6
     - 4.3.1 Groundwater Quality 6
5. Discussion and Recommendations 7
   - 5.1 Excavation and Dewatering 7
   - 5.2 Tank Foundations 8
   - 5.3 Proposed Retaining Wall 9
   - 5.4 Site Servicing 9
   - 5.5 Pavement 10
     - 5.5.1 Subgrade Preparation 10
     - 5.5.2 Pavement Structure 11
   - 5.6 Seismic Site Class 11
6. Soil Environmental Testing 12
   - 6.1 Soil Characterization 12
   - 6.2 Corrosivity 13
7. Construction Monitoring 14
8. Limitations of the Report 15

Figure Index

Figure 1 Site Location Map

Figure 2 Borehole Location Plan

Table Index

Table 3.1 Investigation Locations and Drilled Depths 2

Table 5.1 OHSA Maximum Slope Inclinations 7

Table 5.2 Lateral Earth Pressure Soil Parameters 9

Table 5.3 Pavement Structure Component Thicknesses 11

Table 6.1 Soil Corrosivity Test Results 13

Table 6.2 Construction Concrete Degree (Class) of Exposure 14

Table 6.3 Sulphate and Chloride Concentrations 14

Appendix Index

Appendix A Borehole Logs

Appendix B Analytical Laboratory Test Results

Appendix C Single Well Response Test Results

Appendix D Water Quality Test Results

1. Introduction

GHD was retained by Bruce Power LP (BPLP) to complete a geotechnical investigation as part of the detailed design for the proposed sewage process plant (SPP) upgrades at The Bruce Power Generating Station (BPGS) located at 177 Tie Road Tiverton, Ontario (Site or Project). The Site Location Map is shown as Figure 1.

As per the design drawings under preparation with GHD Design Group, the proposed Third Train infrastructure will comprise:

- Primary settling tank (15.85 m x 3.66 m)
- Two primary aeration tanks of (9.6 m x 3.66 m)
- Primary Aerobic Digester Stage-1 (10.36 m x 3.66 m)
- Primary Aerobic Digester, Stage -2 (5.18 m x 3.66 m)
- Effluent Channel
- Effluent Trough
- Decant/Sludge transfer sump (3.66 m x 1.3 m), and
- Asphalt paved access road along the perimeter of Third Train

GHD has carried out a geotechnical investigation to determine to subsurface soil and groundwater conditions for the proposed Third Train infrastructure described above. This report includes the findings of the geotechnical investigations and provides our geotechnical design and construction recommendations to support the design of these structures.

2. Field and Laboratory Work Programs

Details of the field and laboratory work programs are provided in Sections 2.1 and 2.2 below.

2.1 Field Work Program

The scope of work (SOW) comprised of advancing six boreholes, referenced as BH1-19 to BH6-19 at the Site, to depth varying from 6.40 m below the existing ground surface (bgs) to 9.20 m bgs. Boreholes BH1-19, BH2-19 & BH6-19 were drilled to 6.70 m bgs, BH3-19 & BH4-19 were terminated at 6.40 m bgs, and BH5-19(MW) was drilled to a depth of 9.2 m bgs. The boreholes were located and surveyed in the field by GHD staff. The surveyed borehole locations are shown on Borehole Location Plan, presented as Figure 2.

The drilling crew and the GHD supervisor providing drilling oversight completed the BPLP mandatory safety training prior to commencement of field drilling work. A Site-specific Health and Safety Plan (HASP) was developed by GHD for implementation prior to commencement of any field activities and associated investigation program. A tailgate meeting was conducted with GHD and our drilling subcontractor on a daily basis to discuss the associated health and safety requirements,

any changes in the physical and environmental conditions associated with current weather, ground conditions and drilling equipment and machinery.

Prior to commencement of field drilling activities, underground utility locate marking and clearances were performed by Bruce Power. The drilled depths, coordinates and ground surface elevation of each borehole, based on the staked /drilled field locations are provided in the table below.

Table 3.1 Investigation Locations and Drilled Depths

Borehole No.	Northing (m)	Easting (m)	Ground Surface Elevation (m amsl)	Borehole Depth (m BGS)
BH1-19	4907978.648	453162.959	188.03	6.70
BH2-19	4907972.252	453174.107	188.08	6.70
BH3-19	4907965.582	453185.322	188.06	6.40
BH4-19	4907977.697	453193.683	188.00	6.40
BH5-19(MW)	4907982.186	453181.639	187.77	9.20
BH6-19	4907982.422	453164.975	187.90	6.70

The ground surface elevations were measured with reference to a benchmark described as:

Benchmark 89U093 Control Point 9: Rock post in NE corner of south utility duct manhole, located at NAD83_UTM17 Northing= 4907848.095, and Easting = 452809.151, with Orthometric Elevation = 187.841 metres

The boreholes were advanced by Walker Drilling Ltd., a Ministry of the Environment, Conservation and Parks (MECP) licensed unionized driller on August 20 to 22, 2019, using a track-mounted drill rig equipped with hollow stem augers. Drilling was completed under the full time supervision of GHD's geotechnical staff who logged the stratigraphy of overburden soils in each borehole. The borehole logs are presented in Appendix A.

Representative samples of the strata were obtained utilizing a 50 millimeter (mm) outer diameter split-barrel sampler, advanced by dropping a 63.5 kilogram (kg) hammer 760 mm, in accordance with the Standard Penetration Test (SPT) method (ASTM D1586). The SPT results are reported as 'N' values on the borehole logs at the corresponding depths.

BH5-19(MW) was completed as a groundwater monitoring well. The bottom approximately 3 m depth (from bottom of the drilled hole up to 6.4 m depth bgs was backfilled with hydrated bentonite pellets. The drilled borehole depth from 6.1 m bgs to 6.4 m bgs was backfilled with commercial silica sand. The monitoring well was instrumented in the remaining depth of borehole, with a 3 m (10-foot) long, 50 mm (2-inch) inside diameter, No. 10 slot, Schedule 40 poly vinyl chloride (PVC) screen, and a 50 mm (2-inch) solid wall, Schedule 40 PVC riser pipe of required length extended about 1.0 m above the existing ground surface. The silica sand pack was placed in the annular space between the PVC screen/riser pipe and the borehole, from the bottom of the well screen to at least 0.50 m above the top of the well screen. Bentonite plug was placed above the sand pack within the annular space in the borehole. A J-Plug was installed at the top of solid wall well casing riser, and the well was secured with a 100 mm square stick up steel protective casing installed in

concrete. The monitoring well installation detail is shown on the respective monitoring well log included in Appendix A. GHD measured the groundwater level in this monitoring well on August 29. This groundwater well was registered with the MECP. This well must be decommissioned by a licensed well driller in accordance with MECP regulations, once it is no longer monitored.

Boreholes not installed with monitoring well were backfilled with bentonite chips and soil cuttings in accordance with O. Reg. 903. Excess soil cuttings were containerized in steel drums for off-Site removal by BPLP at a later date.

Groundwater observations were made during and completion of drilling.

Soil samples obtained from the boreholes were observed in the field for type, texture, colour and moisture, and were sealed in clean plastic containers. All samples were transferred to GHD's laboratory Waterloo where they were re-examined by a senior geotechnical engineer to verify the accuracy of the initial soil description/identification and selection for laboratory testing.

2.2 Laboratory Work Program

2.2.1 Geotechnical Laboratory Testing

The geotechnical laboratory testing program consisted of moisture content tests on all recovered soil samples from split spoons. The laboratory moisture content results are noted at their corresponding depths on the respective borehole logs, presented in Appendix A.

2.2.2 Analytical Laboratory Analyses

Three samples (one from each of BH1-19, BH4-19 and BH6-19) were tested for a soil corrosivity package comprising of analyses for resistivity, Redox Potential, pH, and Sulphides. Three samples (one from each of BH2-19, BH3-19 and BH5-19(MW)) were tested for bulk environmental quality analysis comprising O.Reg. 153 metals and inorganics, volatile organic compounds (VOCs), semi-volatile organic compounds (SVOCs), and Polychlorinated Biphenyls (PCBs). One composite soil sample was also submitted for leachate toxicity characteristic leaching procedure (TCLP) in accordance with the General Waste Control Regulation 347/90. The TCLP was tested for metals and inorganics, VOCs, SVOCs, and PCBs.

The samples were submitted under chain of custody to ALS Laboratories in Waterloo, an MECP approved soil and groundwater analytical testing laboratory. The test results are discussed in Section 6, and the detailed laboratory results are provided in Appendix B.

2.2.3 Single Well Response Test

A single well response test (SWRT), was completed on BH5-19(MW) to estimate the horizontal hydraulic conductivity of the water bearing deposit underlying the Site. Single well response tests involve the injection or removal of a known volume of water into/from the well and measuring the water level response in the well until it returns to static conditions (i.e., falling/rising head test).

The results of the hydraulic testing were analyzed using the Bouwer and Rice (1976) and Hvorslev (1951) solution for unconfined conditions using the software package AQTESOLVTM. These solutions were used to determine the horizontal hydraulic conductivity of the geologic deposits within

the immediate vicinity of the screened interval of the monitoring well. The SWRT results are presented in Appendix C.

2.2.4 Sewer Use Sampling

On September 12, 2019 a groundwater sample was collected from BH5-19(MW) for laboratory analysis of the following selected parameters, as required by Bruce Power:

- BOD
- TSS
- Total Phosphorus
- Total Ammonia Nitrogen
- Total Metals (ICP Metals Scan)
- Total Organic Carbon
- Total Oil&Grease
- pH
- E. Coli (Fecal Coliforms)
- Acute Lethality (Rainbow Trout)

The groundwater sample was submitted under chain of custody procedures to Bureau Veritas analytical laboratory of Mississauga, Ontario a Canadian Association for Laboratory Accreditation Inc. (CALA) accredited analytical laboratory. Results from the laboratory analyses are provided in Appendix D.

3. Regional Geology

A review of the Quaternary geology map[1] shows that the Silurian dolostone bedrock of the Guelph Formation is shallow in this area, locally covered with a thin veneer of laterally discontinuous stony sandy silty Elma-Catfish Creek till.

4. Subsurface Conditions

The subsurface conditions are detailed on borehole logs included in Appendix A, and are generally consistent with the regional geology. Subsurface conditions were determined at the borehole locations only, and may vary at other locations. The boundaries between various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between various strata, rather than a precise plane of geological change.

1 Sharpe D.R, Edwards, W.A.D (1979): 'Quaternary Geology of the Chesley-Tiverton Area' Map 2314, Ontario Geological Survey, Scale 1:50,000

A brief description of the soil stratigraphy encountered at each borehole location is summarized in order of depth in the following sections.

4.1 Ground Cover

4.1.1 Asphalt and Topsoil

Boreholes BH1-19, BH2-19 and BH3-19 were advanced through the existing asphalt paved surface. A 100 mm thickness of asphalt was measured in these three borehole locations.

A surficial layer of topsoil was encountered in BH5-19(MW) and BH6-19. A 75 mm thickness of topsoil was noted at these two borehole locations.

4.1.2 Fill Sand and Gravel Deposits

The surficial asphalt layer in boreholes BH1-19, BH2-19, and BH3-19 is underlain by a granular pavement structure comprising gravelly sand fill materials. The thickness of granular pavement structure was noted to be 600 mm in each of the three boreholes. Gravelly sand fill deposits were also encountered in BH4-19 at surface, and under the surficial topsoil layer in BH-5(MW) and BH6-19. The thickness of these sandy fill soils were noted to be 700 mm, 800 mm and 100 mm in BH4-19, BH5-19(MW) and BH6-19, respectively. These existing fills are associated with the pavement construction and previous grading activities at the site.

The SPT 'N' values in the gravelly sand fill soils very from 29 to 48, which indicates the gravelly sand fill soils are in a compact to dense state,

The moisture content of the samples collected from the gravelly sandy soils ranged from 5 percent to 9 percent indicating moist conditions.

4.2 Native Soil

4.2.1 Sandy Silt/ Silty Sand/ Silty Sand Till/ Sandy Silt Till Deposits

Below the asphalt/granular pavement structure in BH1-19, BH2-19 and BH3-19, and gravelly sand fill materials encountered near surface in BH4-19 and under the topsoil/gravelly sand fill materials encountered in BH5-19(MW) and BH6-19, undisturbed native deposits were contacted in each borehole.

The upper layer of native deposits comprise sandy silt to silty sand materials, transitioning into sandy silt till, and continue to the termination depth of the boreholes, except in BH1-19 and BH4-19 where the native deposits comprise silty sand till/ sandy silt till in BH1-19 and sandy silt till in BH4-19, respectively.

The SPT 'N' values recorded in the native sandy silt to silty sand deposits (except in BH6-19) ranged from 8 to 46, indicating that these upper sandy soil layers are generally compact to dense, with some loose wet zones (due to perched groundwater). The SPT 'N' values recorded in the upper sandy silt native soils encountered in borehole BH6-19 ranged from 29 to split spoon refusal, indicating these soils are in a compact to very dense state of relative density. The SPT 'N' values recorded in the sandy silt till and silty sand till deposits ranged generally from 30 to greater than 100, indicating the till deposits are in general dense to very dense. Split spoon refusal to penetration was

noted at many of the sampling attempts in the till deposit, indicating very dense conditions, and the potential presence of cobbles and boulders embedded in the till deposit.

The moisture content of the samples collected from the native deposits ranged from 4 percent to 10 percent indicating moist conditions. The till deposit in BH1-19 becomes moist-wet at 5.1 m bgs, and a wet layer of silt was encountered at 4.2 m bgs in the till deposits in BH2-19.

BH-5(MW) encountered a shaley limestone layer near its termination depth of 9.0, which was identified as possible bedrock. Confirmation of bedrock through rock coring was not part of the mandate for this scope of work.

4.3 Groundwater

Groundwater observations were made in the boreholes during and upon completion of drilling. Groundwater was not encountered in any of the boreholes at the time of drilling other than the isolated wet silt layer encountered at 4.2 m bgs in BH2-19 and the native till deposits becoming moist/wet at 5.1 m bgs in BH1-19. The groundwater depth in BH5-19(MW) was measured on August 29, 2019 at 2.83 m bgs, corresponding to elevation 184.94 m amsl. This groundwater table is suspected to be perched within the upper sandy silt/silty sand layer. These perched groundwater tables should be expected to vary in depth/elevation depending on seasonal conditions, and in response to extreme weather events.

A single well response test completed on monitoring well BH5-19(MW), which is screened within the sandy silt till deposit. The hydraulic testing indicated the till has a hydraulic conductivity of 9.2×10^{-6} cm/sec (geomean). The low hydraulic conductivity is representative of the till deposit.

Based on the measured groundwater levels and the presence of low permeability till below the fill, minor groundwater control requirements are anticipated for the construction. The seepage rate into the excavation is expected to be low and it should be possible to control the seepage by conventional construction dewatering techniques, (i.e., gravity drainage and pumping from open sumps) or, if necessary pumping from deeper sumps with proper filter.

The groundwater seepage does not take into account potentially 'perched' water contained within surficial fill or bedding material for adjacent utilities, parking or road subgrade material, building foundations or other saturated fill material that may be intersected by the building excavation.

An MECP Environmental Activity and Sector Registry (EASR) is required for temporary construction groundwater takings (combined groundwater and surface water) of between 50,000 to 400,000 L/day, and a MECP Permit to Take Water (PTTW) is required for takings of more than 400,000 L/day. Based on the low permeability till, it is anticipated that a construction water taking EASR would be required for removal of a minor amount of groundwater, potentially perched water and surface water during rainfall storm events.

4.3.1 Groundwater Quality

The groundwater sample collected from monitoring well BH5-19(MW) was assessed with respect to the Corporation of the Municipality of Kincardin Sewer Use Criteria (By-Law No. 2016 – 128). Results from the laboratory analysis are presented in **Appendix D - Table D1**.

All of the parameters tested had concentrations below the Municipality of Kincardin Storm Sewer Use By-Law criteria, except for manganese, and the Acute Lethality (Rainbow Trout) testing passed (greater than 95 percent confidence). The concentration of manganese was 0.062 mg/L, which slightly exceeds the storm sewer use criteria of 0.05 mg/L.

The discharge would be a combination of groundwater, surface water runoff and precipitation into the excavation and additional sampling would be required during construction to confirm the quality of the water that would be discharged. Pre-treatment may be required, such as, settlement, filtration and other treatment processes prior to discharge.

5. Discussion and Recommendations

The purpose of this geotechnical study was to determine subsurface soil and groundwater conditions at the Site and to provide geotechnical recommendations to aid in the design and construction of the proposed SPP Third Train structures. Our geotechnical group reviewed the draft design drawings dated June 21, 2019. The recommendations provided below should be reviewed by the GHD design team. Our geotechnical group can be consulted if additional recommendations or clarifications are required.

5.1 Excavation and Dewatering

The cross sections shown on the GHD draft Design Drawings S-005, S-006 and S-007 indicates that the depth of the foundation for the settling, aeration and digester tanks extend to an approximate depths varying from 4.8 m to 5.2 m below the finished grade. It is our understanding that the existing grades will be maintained, and excavation will extend to a depth of 5.2 m bgs, except at the settling tank sump location, where the excavation will extend to a depth of 7.2 m bgs.

For the excavations with unsupported sides, the OHSA regulations require that if workers must enter an excavation deeper than 1.2 m, the excavation must be suitably sloped and/or braced in accordance with the OHSA requirements. OHSA specifies maximum slope of the excavations for four broad soil types as summarized in the following table:

Table 5.1 OHSA Maximum Slope Inclinations

Soil Type	Base of Slope	Maximum Slope Inclination
1	Within 1.2 m of bottom of trench	1 horizontal to 1 vertical
2	Within 1.2 m of bottom of trench	1 horizontal to 1 vertical
3	From bottom of excavation	1 horizontal to 1 vertical
4	From bottom of excavation	3 horizontal to 1 vertical

The upper fill deposits and sandy silt/silty sand soils can be classified as Type 3 soils when the soils are in a moist state and/or properly dewatered if groundwater is encountered. The lower dense to very dense sandy silt to silty sand till soils are considered Type 2 soils. Excavated soil stockpiles must be kept at least 1 m away from the top of an excavation. Storm water should be directed away from an open excavation.

Based on the borehole investigation, the till deposits are generally dense to very dense, and will contain cobbles and boulders, and these conditions may slow the progress of excavations. Significant groundwater infiltration is not expected within the estimated excavations extending generally to a maximum depth of 5.2 m bgs and to 7.5 m bgs at the settling tank sump location. Groundwater may be present in the upper sandy soils, however, if this perched groundwater is expected to be able to be controlled by conventional sump pump methods within the excavation, during dryer construction weather. Surface water should be directed away from open excavations. The amount of groundwater to be controlled will depend on the season and weather conditions at the time of construction.

5.2 Tank Foundations

As per the design drawings under preparation with GHD Design Group, the proposed Third Train infrastructure will comprise:

- Primary settling tank (15.85 m x 3.66 m)
- Two primary aeration tanks of (9.6 m x 3.66 m)
- Primary Aerobic Digester Stage-1 (10.36 m x 3.66 m)
- Primary Aerobic Digester, Stage -2 (5.18 m x 3.66 m)
- Effluent Channel
- Effluent Trough
- Decant/Sludge transfer sump (3.66 m x 1.3 m)
- Asphalt paved access road along the perimeter of Third Train

The cross sections shown on the GHD draft Design Drawings S-005, S-006 and S-007 indicates that the depth of the foundation for the settling, aeration and digester tanks extend to an approximate depths varying from 4.8 m to 5.2 m below the finished grade, except for the settling tank sump which will be founded at a depth of 7.5 m bgs. The base of the tanks will consist of a 600 mm thick reinforced concrete slab, designed to resist upward hydrostatic pressures. The top of slab grade is Elevation 183.7 m amsl. The subsurface conditions expected at the underside of the tanks will consist of dense to very dense sandy silt till, and groundwater seepage is not anticipated. Based on these subsurface conditions, the design bearing pressures for the foundations, as noted on Drawing S-002, of 120 kPa under the serviceability limit state (SLS), and 180 kPa under the ultimate limit state (ULS) design, are acceptable.

The tank raft foundations are expected to be placed below the frost penetration depth of 1.2 m. Excavation to the requisite 1.2 m, and hence no additional frost protection insulation would be required. Any foundation with less than 1.2 m earth cover must be provided with equivalent frost protection.

The proposed tank structures will be subject to unbalanced earth pressures must be designed to resist lateral earth and hydraulic pressures. The following soil parameters may be used for the foundation wall design:

Table 5.2 Lateral Earth Pressure Soil Parameters

Soil	Bulk Density (kN/m^3)	Angle of internal friction ϕ' (degrees)	Rankin Coefficients of Lateral Earth Pressures		
			Ka	Ko	Kp
Compact Granular fill (existing or imported)	21	32	0.31	0.47	3.25
Existing sandy fill soils	19	30	0.33	0.50	3.00
Native sandy silt or sandy silt till	21	32	0.31	0.47	3.25

The K_0 earth pressure coefficients should be used for the design of the tank foundation walls. As there is no drainage provided in the design for the foundation walls and under the tank slabs, the design must consider full hydrostatic groundwater pressures acting on the walls (assuming the groundwater level is at the ground surface, and the tanks are empty, for the most conservative design scenario).

The tanks can be backfilled with the excavated on site soils, however, as these soils are not free-draining, the tanks must be designed for full hydrostatic pressures.

5.3 Proposed Retaining Wall

Based on the design drawings, the existing site grades drop down to about Elevation 186 or 186.5 m amsl in the northwest corner of the site, with respect to proposed finished grades of about Elevation 187.8 to 188.0. This will require construction of a 1.5 to 2 m high retaining wall to support the perimeter access road around the tanks in this corner of the site. The retaining wall can be designed using the Ka values in Table 5.2 above for the imported granular fill. The retaining wall base should be inspected by the Geotechnical Engineer, and should be designed considering an SLS bearing capacity of 100 kPa for the foundation. A drainage tile should be included at the base of the wall outleted by gravity, to prevent hydrostatic pressure acting on the retaining wall. The wall should be backfilled with free-draining Ontario Provincial Standard Specifications (OPSS) Granular B material, compacted to at least 95 standard Proctor maximum dry density (SPMDD).

5.4 Site Servicing

Based on the anticipated depths of the site service pipes at 1.5 m bgs, the subgrade soils in the open cut sections will primarily be in the native silty sand/sandy silt/ till soils in a compact to dense state. If loose/soft zones are encountered at the service pipe invert elevations, these should be removed to a competent stratum as directed by the Geotechnical Engineer, and backfilled with compacted granular material.

The bedding for trenched (open-cut) services should consist of materials meeting OPSS Granular 'A' requirements. The bedding should have a minimum thickness of 150 mm below the pipe and 300 mm above and adjacent to the pipe. The bedding and cover materials should be compacted to a minimum of 98 percent of the SPMDD to provide support and protection to the utility installations.

Backfilling of excavation trench can be accomplished by reusing the excavated soils or imported free draining granular material, provided the moisture content is maintained within 3 percent of the optimum moisture content determined by standard Proctor testing. The inorganic native soils or existing granular fill free of organic material are considered appropriate for reuse as backfill provided they are within 3 percent of their optimum moisture content. The excavation and backfill operation should be carried out during dry weather under geotechnical supervision.

Backfilling operations should be carried out with the following minimum requirements:

- Adequate vibratory compaction equipment is used to compact the material. A light equipment such as walk behind plate compactor be used for the backfilling around the tanks walls.
- Backfilling operation should be carried out evenly around these structures to avoid any lateral differential loading, and should not be carried out before the concrete has gained its specified strength.
- Loose lift thickness should not exceed 200 mm.
- The soils be at suitable moisture contents to achieve compaction to at least 95 percent of SPMDD.
- General backfill materials to consist of earth fill comprised of well-graded soils, with no particle sizes greater than 150 mm, and containing no topsoil, organics or other deleterious materials.

It is recommended that all backfilling be carried out in dry and warm weather conditions to promote optimal drying of the backfill materials. Some drier soils from isolated areas may need conditioning by addition of water to bring the fill materials to optimum moisture level.

All backfill operations and materials should be inspected and tested by qualified geotechnical personnel to confirm that proper material is utilized and that adequate compaction is attained.

Some settlement is to be expected in areas of trench backfill. Therefore, it is recommended that as long a period of time as possible be left between backfilling the trenches and completing the surface structures such as pavements, walkways etc.

5.5 Pavement

5.5.1 Subgrade Preparation

The subgrade for the proposed asphalt paved road along the perimeter of the proposed north train is generally expected to comprise existing granular fills, or on-site silty to sandy soils. Prior to constructing the new pavements, the exposed subgrade should be inspected and proof rolled in presence of the Geotechnical Engineer. If further increase in the Site grades is required, inorganic mineral soils free from deleterious, unsuitable materials and rocks larger than 150 mm in size, imported soils conforming to OPSS Granular 'B' or similar should be used to raise the grades. The fill should be placed in large areas where it can be compacted using heavy duty compaction equipment. Any fill placed to increase or level the grades must be compacted to a minimum 98 percent SPMDD, in lifts not exceeding 300 mm loose thickness. In-situ density testing to monitor

the effectiveness of the compaction equipment in achieving the required densities is also recommended.

After removal of the unsuitable materials, if encountered, the design subgrade should be compacted using a heavy duty smooth drum roller to minimum dry density of 98 percent SPMDD in the presence of the Geotechnical Engineer. During the compaction operations, loose and soft areas that cannot be compacted to 98 percent SPMDD should be sub-excavated to a competent stratum, and replaced with compacted approved on-site or imported well graded granular material. Any fill placed to increase or level the grade must be compacted to a minimum 98 percent SPMDD in lifts not exceeding 300 mm. In-situ density testing to monitor the effectiveness of the compaction equipment in achieving the required densities is also recommended.

5.5.2 Pavement Structure

The pavement structure should be constructed to be consistent with the existing pavement structure. Based on the borehole information, we recommend the following pavement structures can be used for light duty and heavy duty vehicular traffic.

Table 5.3 Pavement Structure Component Thicknesses

Pavement Layer	Minimum Compaction Requirements	Light Duty Pavement Design	Heavy Duty Pavement Design
Asphalt Concrete HL3	92% Marshall Density	40 mm	40 mm
Asphalt Concrete HL8	92% Marshall Density	50 mm	60 mm
Base Course: OPSS Granular 'A'	100% Standard Proctor Maximum Dry Density	150 mm	250 mm
Subbase Course: OPSS Granular 'B'	100% Standard Proctor Maximum Dry Density	300 mm	300 mm

If road base/subbase construction occurs in wet inclement weather, it may be necessary to provide additional subgrade support for construction traffic by increasing the thickness of the granular road subbase. Placement of asphaltic concrete in wet inclement weather or freezing conditions is not recommended.

The efficient drainage of water infiltrating through the pavement layers is essential for a durable pavement structure. Grading adjacent to pavement areas should be designed so that water is not allowed to pond adjacent to the outside edges of the pavement. The pavement subgrade should be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward the pavement edge ditches or catch basins. The finished pavement surface should be crowned to promote efficient cross-drainage toward the catch basins. Water should not be allowed to pond on the pavement surface or along its edges.

5.6 Seismic Site Class

The 2012 OBC requires the assignment of a Seismic Site Class for calculations of earthquake design forces and the structural design based on a two percent probability of exceedance in 50 years. According to the 2012 OBC, the Seismic Site Class is a function of soil profile, and is based on the average properties of the subsurface soil strata to a depth of 30 m below the ground

surface. The 2012 OBC provides the following three methods to obtain the average properties for the top 30 m of the subsoil strata:

- Average shear wave velocity
- Average Standard Penetration Test (SPT) values (uncorrected for overburden)
- Average undrained shear strength

The present geotechnical investigation comprised of boreholes with SPT 'N' value measurements that extended to a maximum depth of 9.2 m bgs. Based on our knowledge of the local geology, and using the criteria listed in Table 4.1.8.4.A. of the 2012 OBC, a Seismic Site Class 'C' can be used for preliminary design purposes. We can review classification with the Structural Engineer as the design is further advanced.

6. Soil Environmental Testing

6.1 Soil Characterization

Three samples (one from each of BH2-19, BH3-19 and BH5-19(MW)) were tested for environmental quality, consisting of analysis for O.Reg. 153 metals and inorganics, volatile organic compounds (VOCs), semi-volatile organic compounds (SVOCs), and Polychlorinated Biphenyls (PCBs).

The samples were obtained from split spoon samples recovered from the following depths:

- BH2-19: SS-5 (3.05 m – 3.66 m) – sandy silt till (native)
- BH3-19: SS-2 (0.76 m – 1.37 m) – sandy silt (native)
- BH5-19(MW): SS-9 (7.62 m – 7.72 m) – sandy silt till (native)

The soil samples were collected and submitted under chain of custody to ALS Environmental laboratory in Waterloo for analyses of the following parameters:

- Ontario Regulation 153 metals and inorganics
- Volatile organic compounds (VOCs)
- Semi-Volatile organic compounds (SVOCs)
- Petroleum hydrocarbon (F1 – F4)
- Total Polychlorinated Biphenyls (PCBs)
- Sodium Adsorption Ratio (SAR)
- Electrical Conductivity

The test results were compared with the MECP, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act, April 15, 2011, O. Reg. 153/04 against the following tables of the MECP document:

1. Table 1- Full Depth Background Site Condition Standards for Agriculture or other Property Use (MECP Table 1).

2. Table 1- Full Depth Background Site Condition Standards for Residential/Parkland/Institutional/Industrial/Commercial Property Use (MECP Table 1).
3. Table 2- Full Depth Generic Site Condition Standards for Residential/Parkland/Institutional Property Use (MECP Table 2).
4. Table 2- Full Depth Generic Site Condition Standards for Industrial/Commercial/Community Property Use (MECP Table 2).

The detailed analytical report is provided in Appendix B. The following conclusions can be made from the test results:

- The tested samples contained metals and inorganic compounds at levels that did not exceed the MECP Table 1 criteria (#1 and #2) and MECP Table 2 criteria (#3 and #4).
- The samples tested did not have any detection of VOCs, hydrocarbons, or total PCBs that exceeds MMECP Table 1 criteria (#1 and #2) and MECP Table 2 criteria (#3 and #4). All results of these compounds were either below the laboratory detection limits or within the #1, #2, #3 and #4 guideline limits.
- The electrical conductivity and SAR concentrations in the three tested samples were also below the guideline limits of MECP Table 1 criteria (#1 and #2) and MECP Table 2 criteria (#3 and #4).

One composite soil sample was also submitted for toxicity characteristic leaching procedure (TCLP) tests. The TCLP was tested for metals and inorganics, VOCs, SVOCs, and PCB. The purpose of the test was to evaluate for the disposal of surplus soils. The test results were compared with General Waste Control Regulation 347/90. A review of the test results indicate no exceedance from the guideline limits as specified in the O.Reg. 347/90.

Based on the environmental quality testing performed, we do not anticipate any environmental quality concerns with re-use of the existing soil. Should unusual conditions be noted during excavation, such as staining, odours, or oil sheens on groundwater, GHD should be notified, so we can re-assess the quality of the site soils.

6.2 Corrosivity

The following table summarizes the results of corrosivity tests on the samples submitted.

Table 6.1 Soil Corrosivity Test Results

Borehole	Sample Depth (m bgs)	Parameters				
		Resistivity (ohm.cm)	pH	Redox Potential (mV)	Moisture (%)	Sulphides [2] (%)
BH-1	4.6 – 5.2	4480	7.81	176	8.51	<0.00002
BH-4	4.6 – 5.0	4050	7.82	169	7.58	<0.00002
BH-6	4.6 – 5.2	5330	7.89	173	6.28	<0.00002

The potential for Sulphate attack on construction concrete (class of exposure) is determined using Table 3 of the Canadian Standards Association (CSA) document A23.1 04/A23.2 09 'Concrete Materials and Methods of Concrete Construction/Methods of Test and Standard Practices for Concrete' classifies the degree of exposure into the following three classes:

Table 6.2 Construction Concrete Degree (Class) of Exposure

Degree (Class) of Exposure	Water Soluble SO_4 in Soil Sample (%)
Very Severe (S-1)	>2.0
Severe (S-2)	0.20 - 2.0
Moderate (S-3)	0.10 - 0.20

The aggressiveness of the soil on concrete in contact with the soils was evaluated by performing water-soluble sulphate content tests on three sandy silt till samples, generally at and above the proposed foundation levels. The test results are summarized in the following table:

Table 6.3 Sulphate and Chloride Concentrations

Borehole	Sample Depth (m bgs)	Sulphate (%)	Class of Exposure (Ref. Table 3 of CSA A23.1-09)	Potential for Sulphate Attack (Ref. Table 3 of CSA A23.1-09)	Chloride (ug/g)
BH-1	4.6 – 5.2	0.0110	Below S-3	Negligible	9.8
BH-4	4.6 – 5.2	0.0073	Below S-3	Negligible	28.9
BH-6	4.6 – 5.2	0.0054	Below S-3	Negligible	17.5

Based on these results, the degree of exposure of the subsurface construction concrete to Sulphate is negligible, and no special concrete provisions are required. However, the designer should evaluate if the sewage effluent has sulphates that may require a concrete that has higher class of sulphate resistance.

The chloride results are also relatively low, and no special concrete provisions are required.

The detailed environmental test results are included in Appendix B.

7. Construction Monitoring

The foundation installations must be monitored and evaluated by qualified personnel to ensure that the founding achieved is consistent with the design bearing intended by the Geotechnical Engineer. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the 2012 Ontario Building Code.

8. Limitations of the Report

This report is intended solely for Bruce Power LP (Client) and other parties explicitly identified in the report and are prohibited for use by others without GHD's prior written consent. This report is considered GHD's professional work product and shall remain the sole property of GHD. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to GHD. Client shall defend, indemnify and hold GHD harmless from any liability arising from or related to Client's unauthorized distribution of the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevations and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, GHD will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

By issuing this report, GHD is the geotechnical engineer of record. It is recommended that GHD be retained during all earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the test locations only (six boreholes BH1-19 to BH6-19). The subsurface conditions confirmed at the six test locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on site (excavation, dewatering and drainage, blasting, pile driving, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods or frost. Soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by GHD is completed.

All of Which is Respectfully Submitted,

GHD



September 24, 2019

Bruce Polan, M.A.Sc., P.Eng.
Associate, Geotechnical Group

Abdul Hafeez Khan, P.Eng.



Philip Smart, M.Sc., P.Geo.



Source: IMAGE "BNPD SURVEY CONTROL INTACT MONUMENTS" DATED AUGUST 8 2016, RECEIVED FROM BRUCE POWER,

BRUCE NUCLEAR GENERATING STATION
SEWAGE PROCESSING PLANT
THIRD TREATMENT TRAIN

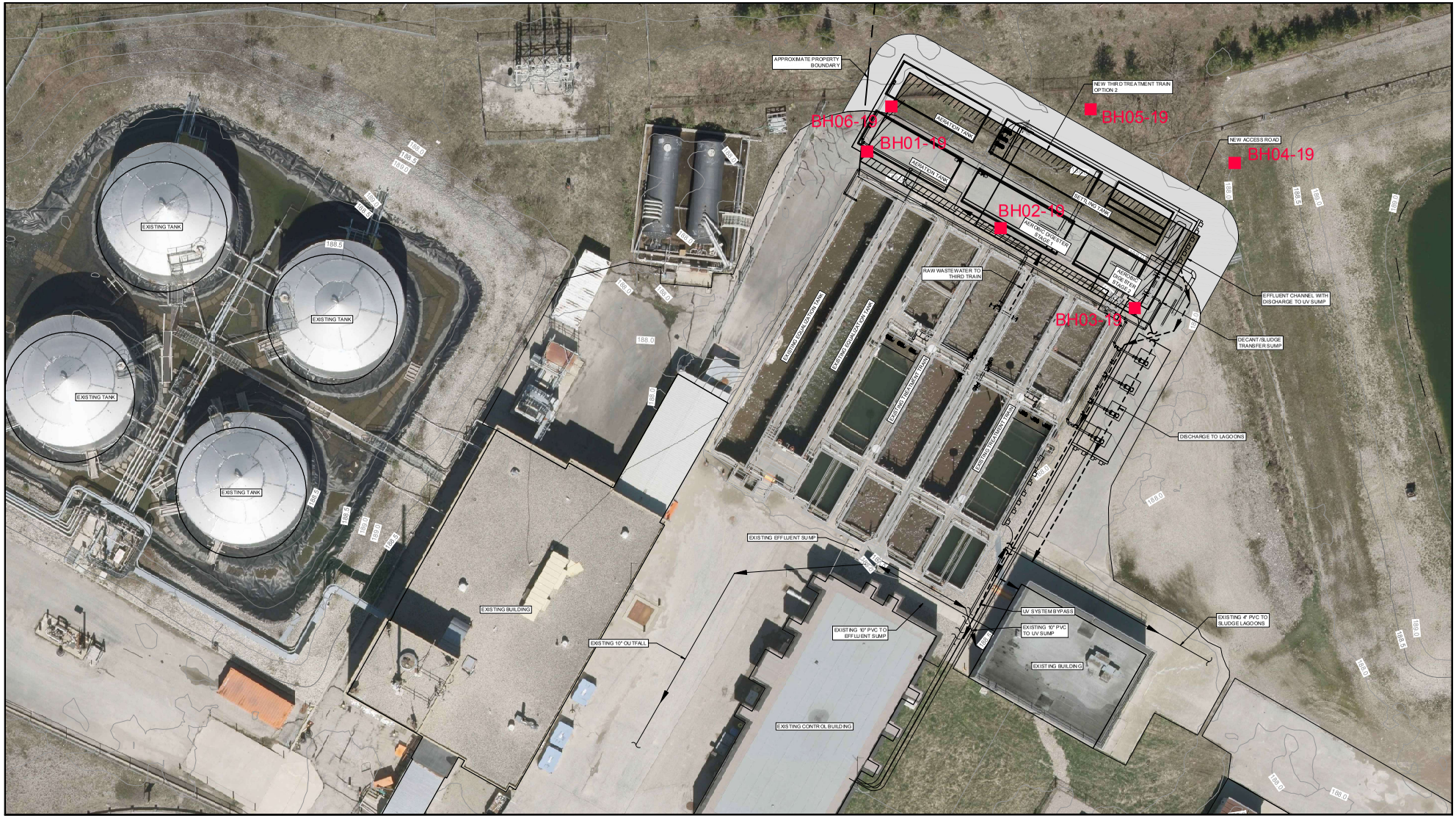
11185554-00

Sep 4, 2019

GEOTECHNICAL INVESTIGATION
SITE LOCATION MAP

FIGURE 1

CAD File: P:\drawings\11185000s\11185554\11185554-REPORTS\11185554-00(006)\11185554-00(006)GN\11185554-00(006)GN-WA001.dwg



0 3 6 9m

BH04-19 BOREHOLE LOCATION

LEGEND

BRUCE NUCLEAR GENERATING STATION
SEWAGE PROCESSING PLANT
THIRD TREATMENT TRAIN

GEOTECHNICAL INVESTIGATION
BOREHOLE LOCATION PLAN

11185554-00
Sep 5, 2019

FIGURE 2

CAD File: P:\drawings\11185000s\11185554\11185554-REPORTS\11185554-00(006)\11185554-00(006)GN\11185554-00(006)GN-WA004.dwg