GEOTECHNICAL INVESTIGATION

Proposed Sewage Pump Station and Water Treatment Plant
Within a Portion of SW 36-59-17 W3M
Flying Dust First Nation, Saskatchewan

Prepared for:

BCL Engineering Ltd.

Date:

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Project File #: PG22-1628

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation conducted for the proposed Sewage Pump Station (SPS) and Water Treatment Plant (WTP) within a portion of SW 36-59-17 W3M within the Flying Dust First Nation, Saskatchewan. The geotechnical investigation was carried out by SolidEarth Geotechnical Inc. (SolidEarth) at the authorization of Mr. Lawrence Lukey, P.Eng., of BCL Engineering Ltd. (BCL).

The purpose of the geotechnical investigation was to assess the subsurface conditions at selected locations within the SPS and WTP footprint, and to provide geotechnical recommendations for the SPS and WTP design and construction.

2.0 PROJECT DESCRIPTION AND INVESTIGATION SCOPE

Based on information provided to SolidEarth, it was understood that the project consists of constructing new SPS and WTP structures. The exact configuration of the new structures was not known. However, it was understood that the SPS and WTP will consist of concrete buildings with the base founded at approximate depths ranging between 6 to 8 m and 4 to 6 m below the existing ground surface, respectively.

The scope of work completed by SolidEarth included drilling boreholes, conducting laboratory review and testing on recovered soil samples, undertaking geotechnical engineering analysis, and preparation of this report.

3.0 SITE DESCRIPTION

The project site was located north of the Town of Meadow Lake approximately 450 m north of the intersection of 1 Avenue East and 2 Street East. A key plan showing the project area on a 2022 aerial photograph is presented as Figure 1. Photographs showing site conditions that existed at the time of the field investigation are presented in Appendix A.

4.0 FIELD AND LABORATORY INVESTIGATION

4.1 GROUND DISTURBANCE AND SAFETY PERFORMANCE

Prior to field drilling, a SolidEarth representative completed internal ground disturbance procedures, which included placing a Saskatchewan First Call. Before starting onsite work, a daily field level hazard assessment was conducted by the SolidEarth representative and was communicated with all workers involved during the tailgate meeting. The field work was completed without any near misses or incidents.

4.2 FIELD DRILLING AND TESTING

The borehole locations were marked in the field by SolidEarth based on the provided development plan, location of underground and aboveground utilities, and access restrictions. The borehole location plan on a 2022 aerial photograph is shown as Figure 2.

SolidEarth subcontracted All Service Drilling Inc., of Nisku, Alberta to drill the boreholes. Drilling was completed using a track-mounted auger drill rig utilizing 150 mm solid-stem continuous flight augers.

The field investigation was undertaken on 20 April 2022 and included drilling three (3) boreholes. BH22-1 was located within the proposed SPS footprint and drilled to an approximate depth of 12.6 m below ground surface. BH22-2 and -3 were located within the proposed WTP footprint and were drilled to approximate depths of 10.4 and 8.8 m below ground surface, respectively.

During drilling, soil samples were collected at approximately 0.75 m intervals along the depth of the boreholes. Standard Penetration Tests (SPT) were conducted at selected depths (typically every 1.5 m) to assess the in-situ strength of the soils encountered. The soil sampling and testing sequences are shown on the borehole logs, Appendix B.

A SolidEarth geotechnical technologist monitored the drilling operations and logged the recovered soil samples from the auger cuttings and the SPT samples. The soils were logged according to the Modified Unified Soil Classification System, which is described in the Explanation of Terms and Symbols in Appendix B. Due to the method by which the soil cuttings were returned to surface, the depths noted on the borehole logs may vary by ± 0.3 m from those recorded.

Groundwater seepage conditions were monitored during and immediately following completion of drilling. Slotted standpipe piezometers was installed at each borehole location at completion of drilling to monitor the short-term groundwater level.

Following completion of drilling, the lateral coordinates (northing and easting) of the borehole locations were recorded by the SolidEarth representative using a hand-held GPS unit. These coordinates are shown on the borehole logs.

4.3 LABORATORY INVESTIGATION

All collected samples were submitted to the laboratory for further examination and testing. Laboratory testing conducted included visual examination, determination of the natural moisture content on all collected samples; and grain size distribution analysis on selected samples. The results of the laboratory testing are presented on the borehole log, Appendix B.

5.0 SUBSURFACE CONDITIONS

The subsurface soil conditions encountered at the borehole locations consisted of topsoil at the ground surface followed by clay deposit and underlain by clay till. Interbedded sand layers were encountered within the clay till. A brief summary of the subsurface soil conditions encountered is presented below. A detailed description of the subsurface conditions encountered at the borehole location is provided on the borehole logs.

Topsoil

Topsoil was encountered at the ground surface at all borehole locations, and was generally less than 250 mm in thickness. It is to be noted that the thickness of topsoil across the site may vary from what was encountered at the borehole locations.

Clay

Clay was encountered below the topsoil at all borehole locations and extended to approximate depths ranged between 0.9 and 1.2 m below the ground surface. These soils were generally classified as “clay, silty to and silt, trace sand”, were medium to high plastic, grey-brown, and wet.

The natural moisture contents of these soils ranged between 26 and 31 percent, with an average of 28 percent. The liquid and plastic limits of samples of the clay soils were in the order of 45 percent and 16 percent, respectively. Based on comparison with the plastic limit, it is expected that the average in-situ moisture content of the clay soils was higher than the optimum moisture content of the soil.

The consistency of the clay soils was assessed based on the SPT “N” and pocket penetrometer values to be generally stiff.

Clay Till

Clay till was encountered below the clay deposit and extended to beyond the exploration depths in BH22-01 and -02, while BH22-03 was terminated in sand). The clay till was generally classified as “clay, silty to and silt, some sand to and sand, trace gravel”, was brown to grey, and moist to very moist. The upper approximately 1 to 2 m of the clay till was low to medium plastic transitioning into medium plastic below that depth.

The natural moisture contents of the clay till soils ranged between 9 and 17 percent, with an average of 13 percent. The liquid and plastic limits of the tested samples of the clay till soils were in the order of 27 to 43 percent, and 9 to 12 percent, respectively. Based on comparison with the plastic limit, it is expected that the average in-situ moisture content of most of the clay till was near the optimum moisture content of the soil.

The consistency of the clay till was assessed based on the SPT "N" and pocket penetrometer values to be generally very stiff within the upper 5 m of the soil profile, becoming hard below that depth.

Interbedded Sand Layers

Interbedded sand layers were encountered within and/or below the clay till soil deposit at all borehole locations. The thickness and depth of these layers varied across the site.

The sand was generally classified as "sand, trace silt to some silt, trace clay to clayey, trace gravel", was fine to coarse grained, poorly graded, grey-brown, very moist to saturated and exhibited seepage and sloughing conditions during drilling. The density of the sand based on the SPT "N" values was assessed to be dense to very dense.

5.1 GROUNDWATER LEVEL

The ground water observation and measurements at the completion of drilling are summarized on the borehole logs. The measured groundwater levels at the borehole locations are shown in Table 1. The depth of the groundwater table is anticipated to fluctuate seasonally depending upon several factors that include the local geology, hydrogeology, and surface infiltration.

Table 1: Measured Groundwater Levels

Borehole ID	Depth of Borehole (mbgs) Note 1	Depth of Installed Standpipe (mbgs)	Groundwater Depth (mbgs)	
			At Drilling Completion	27 April 2022
BH22-1	12.6	10.7	3.4	1.9
BH22-2	10.4	6.7	5.5	1.8
BH22-3	8.8	8.8	7.3	1.9

Note 1: mbgs – metres below the existing grade/ground surface.

6.0 GEOTECHNICAL CONSIDERATION AND RECOMMENDATIONS

6.1 EXCAVATION AND GROUNDWATER MANAGEMENT

6.1.1 Foreword

Based on the information provided, it was understood that the base of the SPS and the WTP will be at approximate depths ranging between 6 to 8 m and 4 to 6 m below the existing ground surface, respectively. The subsurface conditions encountered at the proposed design depth were considered feasible but challenging for the proposed development.

6.1.2 Anticipated Subsurface Conditions

Sewage Pump Station (SPS)

The anticipated depth of the excavation for the SPS base construction was understood to be 6 to 8 m below existing ground surface. It is expected that the soil condition within the top approximately 5 m of the excavation will generally consist of stiff to very stiff clayey soil. Interbedded layers of sand with seepage and sloughing conditions are likely to be encountered below that depth within portions of the excavation.

Water Treatment Plant (WTP)

The anticipated depth of the excavation for the WTP base construction was understood to be 4 to 6 m below existing ground surface. It is expected that the soil conditions within the excavation depth will generally consist of stiff to hard clayey soil. Interbedded sand layers with seepage and sloughing conditions are likely to be encountered below an approximate depth of 5 m below existing ground surface.

It is expected that these soils can be readily excavated with conventional earth moving equipment.

6.1.3 Groundwater Management

The shallow groundwater levels measured in the installed standpipes at the borehole locations are summarized in Table 1. Considering the design base elevation, groundwater management during and following construction should be considered. The design of the base should also consider the hydrostatic pressure on the foundation and below grade wall.

Excavation within Clayey Soils

Seepage from the walls and bases of the excavations may be encountered but is expected to be relatively low and can be controlled with drainage sumps equipped with pumps. The volume of water seeping into the excavation will increase with increasing size and depth of the excavation. The rate of water seepage is also expected to increase if the excavation encountered saturated interbedded sand layers. The water storage and seepage from these sand units will depend on the vertical and lateral extents of the sand layers. If the lateral and vertical extents of such layers are relatively small, they can then be drained relatively easily with a sump pump system.

Excavation Intercepting Sand

Saturated interbedded sand layers and seepage should be anticipated within the lower portions of the excavations (below an approximate depth of 5 m). Generally, for excavations penetrating less than 1 m into the sand layer that will be open for short periods of time, conventional groundwater management (such as a drainage blanket at the base of the excavation with sumps and pumps) may be sufficient. Excavations that will penetrate deeper into saturated sand layers and/or that will be open for prolonged times, non-conventional groundwater management (such as well point system) may be required.

It is important that the groundwater table be maintained below the excavation base and below the side slopes for the duration of the construction and until the excavation is backfilled. Failure of the groundwater management system could result in slope instability, heaving of the base and cracking of the mud slab, flooding of the excavation, floating the utility pipes, and softening of the subgrade.

6.1.4 Excavation Considerations

The contractor may consider supported or unsupported excavation for the SPS and the WTP. Each system should be designed by a qualified engineer. Proper groundwater management (which may include well point system) will likely be required during excavation and building construction and should be incorporated into the overall excavation and/or excavation support plan. Such a system should be designed by a speciality contractor or engineer prior to start of construction. It is to be noted that significant seepage may adversely impact the excavation stability.

Detailed slope stability evaluation (combined with a groundwater management plan) will be required to determine the proper slope inclination and excavation configuration in order to achieve adequate factors of safety against instability. Detailed slope stability assessment of the excavation was outside the scope of this investigation. Once the preliminary excavation configuration is established, detailed slope stability assessment and additional recommendations should be completed. The assessment should be conducted on a case specific basis by a qualified geotechnical engineer to assess the stability of a given slope configuration.

The excavation should be checked regularly for drying and sloughing of the side slopes and for any cracking or surface settlement along top edges of the excavation. In addition, the degree of excavation stability decreases with time and, therefore, construction should be directed at minimizing the length of time excavations are left open.

The excavation should extend sufficient distance past the edge of the bottom slab to provide adequate space and protection for the workers. If space does not permit the slopes to be cut back, some form of temporary shoring must be installed to protect workers in the excavation. The following construction related recommendations affecting excavation stability should be followed:

- All temporary surcharge loads (including soil spoil stockpiles) should be kept back from the excavated faces a distance of at least 1 m or one-half the depth of the excavation (whichever is greater).
- Wheel loads should be kept back at least 2 m from the crests of excavation. Larger setback distances should be established for heavy trucks such as those hauling soil or concrete. Greater setbacks, and flatter side slopes, are recommended for excavations that remain open for extended periods of time.
- Surface grading should be undertaken to prevent surface water from ponding adjacent to or entering the excavation.
- Monitoring and maintenance of the excavation slope should be carried out on a regular basis.
- The latest edition of the *Construction Safety Regulations* of the Occupational Health and Safety Act of Saskatchewan should be followed.

An observational approach combined with local experience with similar subsurface conditions is recommended. It would be desirable for the excavation contractor to be experienced in similar conditions, and/or alternatively to excavate test pits in advance of construction to familiarize field personnel with subsurface conditions. Quality workmanship is essential.

6.1.5 Backfill Behind Building Walls

The soils excavated from the excavation may be used for backfill. All fill soils should be free from any organic materials, contamination, deleterious construction debris, and stones greater than 150 mm in diameter. Some risks associated with the use of high plastic clay and sand are outlined below.

The low to medium plastic native clayey soils sourced from the excavations may be used for backfill material. It is recommended that if layers of high plastic soils are encountered, these layers be isolated and not used in the backfill process.

The encountered native sands may be used as an engineered fill material. However, the native sand may be challenging for use (if it becomes wet due to rain) as these materials may become difficult to compact and require strict control of moisture content. Additionally, the sand materials are generally frost susceptible in the presence of water.

With all soils, moisture conditioning of these soils may be required during construction and will depend on weather conditions at the time of construction.

Excavation backfills should be uniformly compacted to a minimum of 95 percent of Standard Proctor Maximum Dry Density (SPMDD). The fill should be placed at moisture contents within three percent of optimum moisture content. The lift thicknesses should be governed by the ability of the selected compaction equipment to uniformly achieve the recommended density. It is recommended to use lifts with a maximum thickness of 300 mm loose. Fill placement procedures and quality of the fill soils should be monitored by geotechnical personnel on a full-time basis. Field monitoring should include compaction testing at regular frequencies.

Engineered fill should be thawed and placed during non-frozen conditions. If winter construction is proposed, SolidEarth can provide additional recommendations at the time and once the overall development plan has been finalized.

Generally, total settlement of one to three percent of backfill thickness is expected for cohesive soils compacted to between 100 and 95 percent of SPMDD. The magnitude and rate of settlement will be dependent on the backfill soil type, the moisture condition of the backfill at the time of placement, the depth of the excavation, drainage conditions, and the initial density achieved during compaction. It is expected, however, that most of the settlement under self-weight will occur within the first one to two years following construction.

6.1.6 Earth Pressure

Backfill soils behind the underground walls can exert significant horizontal pressures on the wall. Rather than heavily compacting the backfill around the walls, it is recommended to nominally compact the backfill recognizing that settlement of the backfill will occur, particularly in the first year or two following construction.

Assuming that the backfill against the walls will consist of moderately compacted soil (approximately 95 percent of standard Proctor maximum dry density (SPMDD)), the lateral pressure projected onto the sides of the structure may be determined by the following:

$P_h = K_o \gamma h$ above the water table

$P_h = K_o \gamma z + K_o \gamma' (h-z) + \gamma_{water} (h-z)$ below the water table

where:

P_h = lateral pressure at depth h (m) from finished surface grade

K_o = coefficient of lateral earth pressure "at rest", use K_0 = 0.55 and 0.4 for native clay till and local sand, respectively,

h = depth below finished grade

z = depth of groundwater below finished grade

γ = total unit weight of soil, use γ = 20 kN/m^3 and 19 kN/m^3 for native clay till and local sand, respectively,

γ' = buoyant or submerged unit weight of soil, use γ' = 10 kN/m^3 and 9 kN/m^3 for native clay till and local sand, respectively, (below groundwater)

It is assumed that installing a sub-drainage system around and below the SPS and the WTP structure is not preferred. From a geotechnical perspective, this is considered acceptable provided that the building base and walls are designed for hydrostatic pressures. For design purposes the groundwater table may be assumed as 1.5 m below the existing grade.

In addition to earth pressures, lateral stresses generated by surcharge loads, such as point loads from traffic, also need to be evaluated in the design. For line or point surcharge loads, the lateral pressures may be determined using the relationship given in Figure 3. In the case of uniformly distributed surcharge loads acting on the surface of the retained soil, the induced lateral earth pressure may be determined by multiplying the surcharge load by the appropriate earth pressure coefficient.

Loads from compaction equipment would also induce horizontal forces on the walls. Figure 4 shows the horizontal pressures on walls from compaction effort, and typical compaction equipment data for estimating compaction induced loads. It is recommended that only small compaction equipment be used within a distance of 1 to 1.5 m from the underground foundation walls. This will reduce the magnitude of the horizontal forces induced by the compaction equipment during backfilling.

It is also important that proper surface drainage be provided at the ground surface to prevent surface water from seeping and ponding against the underground walls. If water was allowed to saturate the fill behind the walls and subsequently freezes, then significant frost induced lateral earth pressures may be encountered.

For Limit States Design for walls, the earth pressure described above should be multiplied by the appropriate Load Factor listed in Table 2.

Table 2: Load Factors for Earth Pressures

Load Type	Load Factor
Sustained Earth Loads	1.25
Hydrostatic Loads	1.1
Live Surcharge Loads	1.5

Backfill around the walls of the underground structure should be sloped to shed water away from the structure. The slope of the backfill should be checked periodically to maintain the slope of the ground surface away from the foundation wall. It is recommended that the top 0.3 m of the backfill around the building should consist of compacted clay to act as a seal against runoff water. The clay should extend a minimum distance of 3 m past the edge of the underground wall.

Backfilling should be delayed until the concrete has gained sufficient strength to support the horizontal loads. The walls should be adequately braced prior to backfilling. Backfill should be brought up evenly around the building perimeter to minimize differential horizontal pressures on the walls of the underground structure.

6.2 FOUNDATION AND SLAB SYSTEMS DESIGN CONSIDERATION

A raft slab is considered suitable at this site to serve as a base slab as well as the foundation system. It was the understanding of SolidEarth that the foundation will be predominantly subject to vertical static loads with little resistance required for horizontal dynamic loading. If other foundation systems are proposed, or if the foundation is to support large lateral or dynamic loads, then SolidEarth should be contacted and additional recommendations will be provided, as required.

6.2.1 Foundation Design Method

The current design standard in foundation engineering is based on limit state design. Accordingly, geotechnical recommendations associated with such standard are provided in this report.

The *Canadian Foundation Engineering Manual* defines limit states “as conditions under which a structure or its component members no longer perform their intended function”. Limit states are generally classified into two main groups: ultimate limit state and serviceability limit state. Below is a brief discussion on both states.

Ultimate Limit State (ULS)

Ultimate limit states are primarily concerned with collapse mechanisms for the structure and, hence, safety. For foundation design, the ULS consists of: ultimate bearing capacity failure, sliding, overturning, loss of stability, uplift, or large deformation.

The basic foundation design equation using ULS approach is presented as:

$$\Phi R_n \geq \Sigma \alpha_i S_{ni}$$ [1]

where:

- ΦR_n - is the factored geotechnical resistance
- Φ - geotechnical resistance factor
- R_n - the nominal (ultimate) geotechnical resistance determined using unfactored values for geotechnical parameters or performance data (such as pile load test)
- $\Sigma\alpha_i S_{ni}$ - is the summation of the factored overall load effects for a given load combination condition
- α_i - is the load factor corresponding to a particular load
- S_{ni} - is a specified load component of the overall load effects (e.g. dead load due to weight of structure or live load due to wind)
- i - represents various types of loads such as dead load, live load, wind load, etc.

Geotechnical resistance factors as provided by the *National Building Code of Canada (NBCC)* for foundations are provided in Table 3. The critical design events and their corresponding load combination and load factors should be assessed and determined by the structural engineer.

Table 3: Geotechnical Resistance Factors for Foundations

Foundation Type	Loading Condition	Geotechnical Resistance Factor (ULS)
Raft Slab	vertical bearing resistance from semi-empirical analysis	0.5
	horizontal resistance against sliding (based on friction)	0.8

[1] Page 136 of the Canadian Foundation Engineering Manual – 4th Edition, January 2007.

Serviceability limit states are primarily concerned with mechanisms that restrict or constrain the intended use, occupancy, or function of the structure under working loads. For foundation design, SLS are usually associated with:

- excessive foundation movements (e.g. settlement, differential settlement, heave, etc.)
- unacceptable foundation vibrations
- local damage or deterioration

In general, the SLS criteria can be expressed as follows:

Serviceability Limit ≥ Effect of Service Loads

The soil bearing pressure under SLS conditions is evaluated using unfactored geotechnical parameters (settlement and compressibility properties), such that the bearing pressure does not cause the foundation to exceed the specified serviceability criteria.

The soil-structure interaction and load-deformation characteristics of soils are non-linear and complex and depend on several considerations (e.g., foundations size and configuration, range of movement, etc.). The number of possible combinations is infinite and generic design charts cannot be prepared. Specific design charts under SLS conditions can be provided upon request and once preliminary design requirements have been established.

6.2.2 Bearing Capacity of Raft Foundation

The raft is expected to exert loads in the order of 100 to 120 kPa on the subgrade. Given the size and depth of the raft and the nature of the soils below the bearing level, a relatively high ultimate bearing capacity will be available. The ultimate bearing capacity will be governed by excessive settlement, rather than shear failure. An un-factored (ultimate) bearing capacity of 500 kPa may be assumed in the design for a raft based on the sand and/or clay till deposits.

The amount of settlement under the raft is directly proportional to the soil bearing pressure under service limit state (SLS) conditions. Settlement of less than 15 to 25 mm should be anticipated for SLS loading of 150 kPa for a raft based on the sand and/or clay till deposits.

The raft should be structurally designed to carry the anticipated loading. A modulus of subgrade reaction of 35 MPa/m may be used in the design for the raft based on the protected and prepared subgrade soils.

The excavation should be carried out using an excavator with a smooth edge trimming bucket. Final cleanup of foundation subgrade soils by hand methods may be required. No loose, disturbed, remoulded or slough material should be allowed to remain on the foundation bearing surface. Should wet and/or soft soils be encountered at the design foundation depth, the excavations should be deepened and replaced with engineered fill such that foundations bear on competent soils.

Any over-excavation of unsuitable soils could be brought back to design grades using lean-mix concrete (minimum 28 days compressive strength of 5 MPa) or an approved engineered granular fill. Engineered fills should extend laterally 1 m or equal to full depth of fill (whichever is greater) beyond the edge of the raft and be compacted to 100 percent of the SPMDD at moisture content within two percent of the optimum moisture content.

The foundation excavation must be protected from drying, desiccation, rain/snow, freezing, and the ingress of water. Foundation subgrade soils that become frozen, dried, or softened, should be removed and replaced with concrete, or the excavation should be extended to reach soil in an unaffected condition.

It is recommended that the foundation bearing surface excavation be inspected and approved by a qualified geotechnical engineer prior to concrete placement to confirm soil conditions and bearing capacity of the soils.

To help protect the subgrade during construction, it is recommended to maintain construction traffic to a minimum and restricted to low pressure track equipment to the extent possible. Given that the excavation will likely be open for a few months, the placement of a mud slab to protect the inspected subgrade during construction is highly recommended.

It is important that the groundwater table be maintained below the excavation base for the entire duration of the construction and until the excavation is backfilled. Failure of the groundwater management system could result in slope instability, heaving of the base and cracking of the mud slab, flooding of the excavation, and softening of the subgrade.

6.3 SEISMIC SITE CLASSIFICATION

The *2015 National Building Code of Canada (NBBC)* divides sites into six classes (A to F) for seismic response evaluation. This classification is based on the average shear wave velocity, energy-corrected SPT “N” values, or undrained shear strength over the top 30 m of the soil profile.

The borehole advanced within the footprint of the proposed SPS footprint was approximately 12.6 m below the existing ground surface. Based on SPT data within the exploration depth and knowledge of clay till soils (which indicates that the soil consistency generally increases with depth), the general area was categorized as Class “C”.

6.4 SUBSURFACE CONCRETE

Two soil samples, collected from the boreholes drilled for the SPS and the WTP, were tested for water-soluble sulphate concentration. The test results indicate “negligible” potential of sulphate attack in the buried concrete. Therefore, type GU (General Use Hydraulic Cement) may be used for concrete in contact with the existing site soils. Should any material be imported to the site for use as backfill, it should be tested for the presence of sulphates and the above recommendations modified accordingly.

Additional restrictions may be required due to structural or other considerations. To enhance durability, an appropriate amount of air entrainment is recommended for all concrete exposed to freezing and thawing conditions, as per CAN/CSA specification CSA A23.1-09.

7.0 TESTING AND INSPECTION

Recommendations presented in this report may not be valid if adequate engineering inspection and testing programs during construction are not implemented or if other building code requirements are not followed. Testing and inspection programs should consist of:

- Review and approval of the excavation and groundwater management plan
- Foundation bearing surface inspection
- Concrete testing as per industry standards
- Full time monitoring and compaction testing during backfill

8.0 CLOSURE

The recommendations presented in this report are based on the results of soil sampling and testing at three (3) borehole locations advanced during this investigation. Soil conditions by nature can vary across any given site. If different soil conditions are encountered at subsequent phases of this project, SolidEarth should be notified immediately and given the opportunity to evaluate the situation and provide additional recommendations as necessary.

The recommendations presented in this report should not be used for another site or for a different application at the same site. If the intended application of the site is changed or if the assumptions outlined in this report become invalid, SolidEarth should be notified and given the opportunity to assess if the recommendations presented should be modified.

This report has been prepared for the exclusive use of BCL Engineering Ltd., and their authorized users for the specific application outlined in this report. No other warranties expressed or implied are provided. This report has been prepared within generally accepted geotechnical engineering practices.

Respectfully submitted,
SolidEarth Geotechnical Inc.

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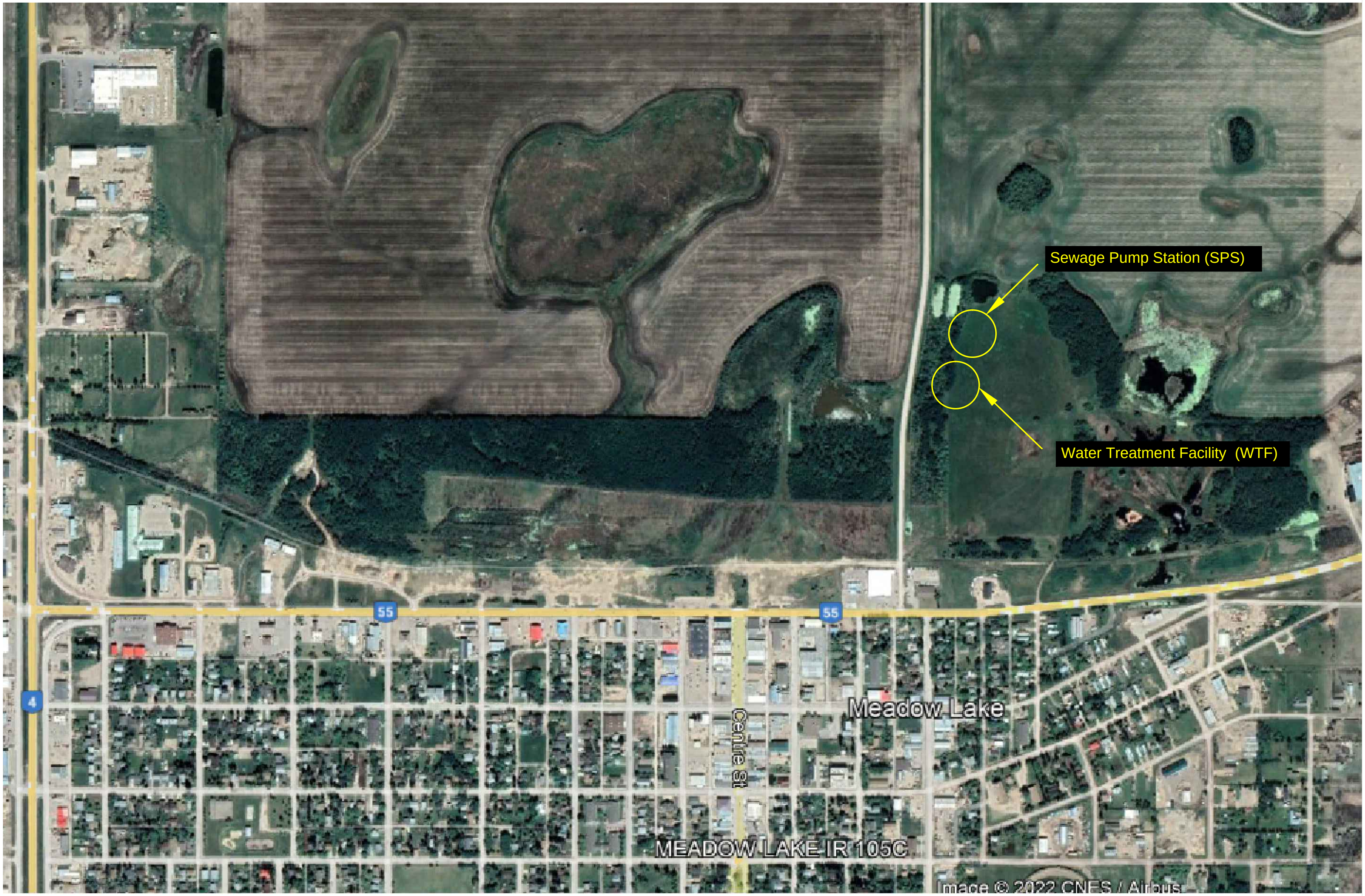
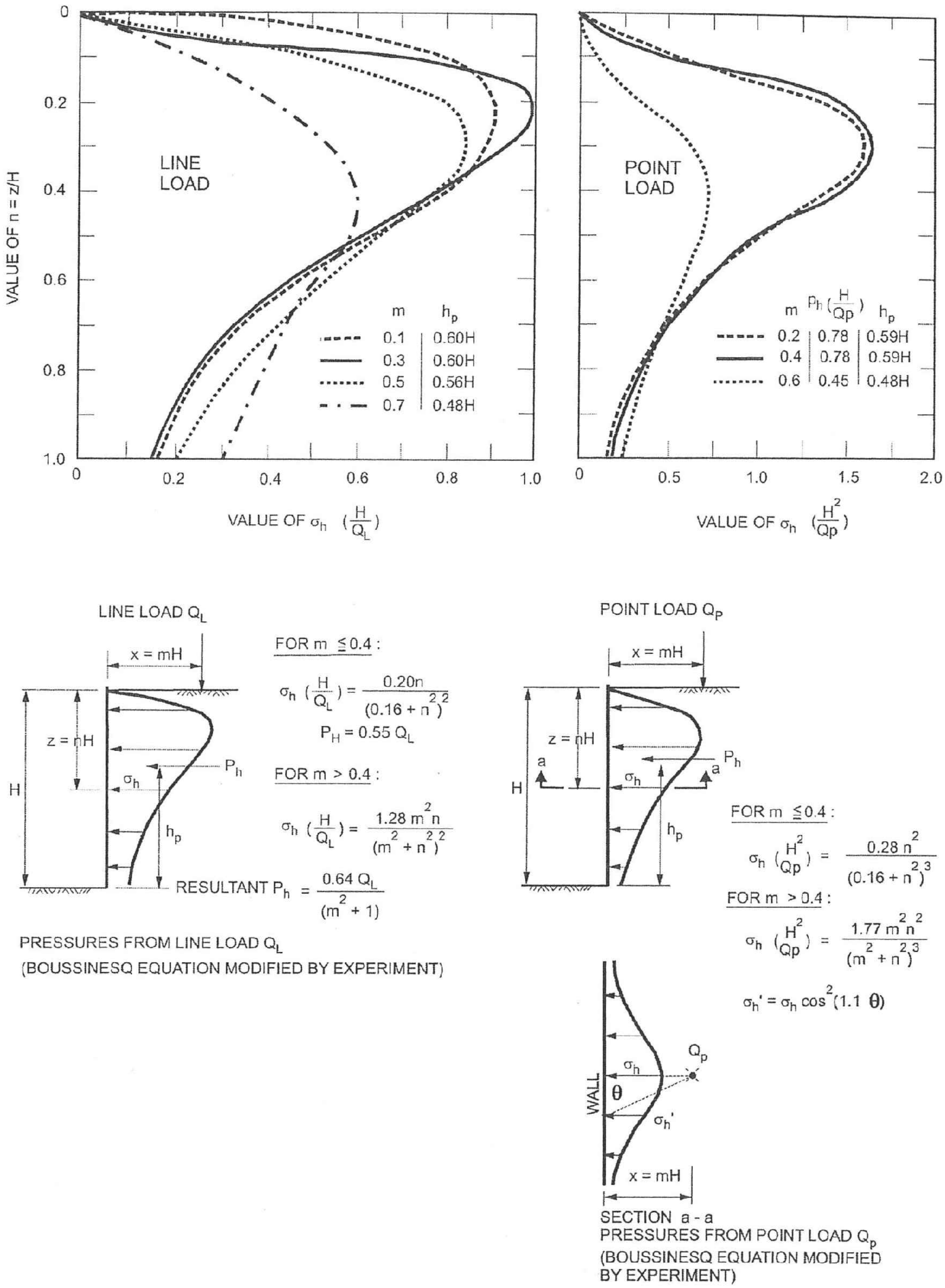
9 May. 2022

Figures



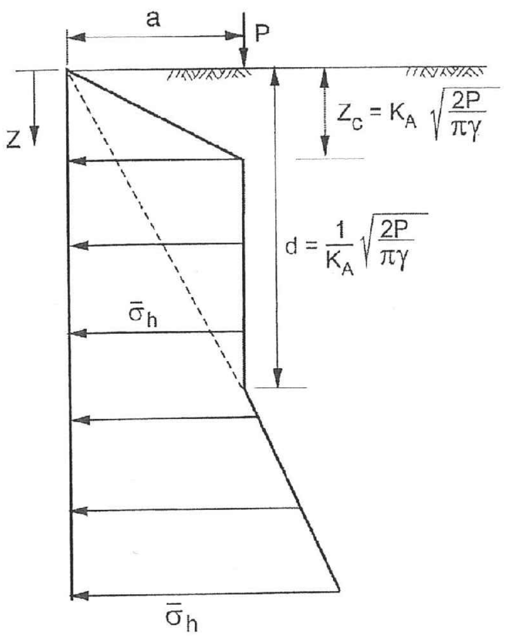
SCALE: NTS	DRAWN BY: JJ	FIGURE No.: 1	REVISION No.: 0	PROJECT NAME: Proposed WTP and PSP Within SW 36-59-17 W3M Flying Dust First Nation, Saskatchewan	CLIENT: BCL ENGINEERING LTD.
DATUM: XXX	DATE: May 2022	PROJECT No.: PG22-1628			
SolidEarth Geotechnical Inc. 4336 97 Street, Edmonton, AB, T6E 5R9		SolidEarth GEOTECHNICAL		DRAWING TITLE: Key Plan on a 2022 Aerial Photograph	

SCALE: NTS	DRAWN BY: JJ	FIGURE No.: 2	REVISION No.: 0	PROJECT NAME: Proposed WTP and PSP Within SW 36-59-17 W3M Flying Dust First Nation, Saskatchewan	CLIENT: BCL ENGINEERING LTD.
DATUM: XXX	DATE: May 2022	PROJECT No.: PG22-1628			
SolidEarth Geotechnical Inc. 4336 97 Street, Edmonton, AB, T6E 5R9		SolidEarth GEOTECHNICAL		DRAWING TITLE: Borehole Location on a 2022 Aerial Photograph	



Reference: Canadian Foundation Engineering Manual, 4th Edition, pg 385

SCALE: NTS	DRAWN BY: ND	FIGURE No: 3	PROJECT NAME: Proposed WTP and PSP Within SW 36-59-17 W3M Flying Dust First Nation, Saskatchewan	CLIENT: BCL ENGINEERING LTD.
DATUM: XXX	DATE: May 2022	PROJECT No: PG22-1628		
SolidEarth Geotechnical Inc. 4336 97 Street, Edmonton, AB, T6E 5R9		SolidEarth GEOTECHNICAL	DRAWING TITLE: Horizontal Pressure on a Wall due to Point and Line Load Surcharges	



For $Z_c \leq Z \leq d$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}}\ \frac{L}{a + L}$$

For $Z > d$

$$\bar{\sigma} = K_A \gamma z$$

$$P\ (\text{roller load}) = \frac{\text{dead weight of roller + centrifugal force}}{\text{weight of roller}}$$

a = distance of roller from wall

L = length of roller

Equipment Type	Dead Weight (kN)	Centrifugal Force (kN)	Roller Width (mm)	P (kN/m)
Single-drum walk-behind	2.3	8.3	560	18.9
Dual-drum walk-behind	1.6	10.1	560	20.9
Dual-drum walk-behind	12.1	8.8	760	27.5
Dual-drum walk-behind	9.2	19.8	750	38.7

Reference: Canadian Foundation Engineering Manual, 4th Edition, pg 386

SCALE: NTS	DRAWN BY: ND	FIGURE No: 4	PROJECT NAME: Proposed WTP and PSP Within SW 36-59-17 W3M Flying Dust First Nation, Saskatchewan	CLIENT: BCL ENGINEERING LTD.
DATUM: XXX	DATE: May 2022	PROJECT No: PG221-628		
SolidEarth Geotechnical Inc. 4336 97 Street, Edmonton, AB, T6E 5R9	SolidEarth GEOTECHNICAL		DRAWING TITLE: Horizontal Pressure on a Wall from Compaction Effort	

Appendix A

Site Photographs Taken During the Field Investigation

Photograph 1: Looking north towards BH22-1 (SPS Location)

Photograph 2: Looking west towards BH22-2 (WTP Location)

Photograph 3: Looking south towards BH22-3 (WTP Location)

Appendix B

Borehole Logs
Explanation of Terms and Symbols

Project Name: Proposed SPS and WTP	Borehole #: **BH22-01**
Client Name: BCL Engineering	Project #: PG22-1628
Site: Flying Dust First Nation - SW 36-59-17 W3M	Logged By: JS / Reviewed By: TF
Northing: 6001869 Easting: 668080	Driller: All Service Drilling Ltd.
Elevation:	Drill Method: 150 mm Solid Stem Auger

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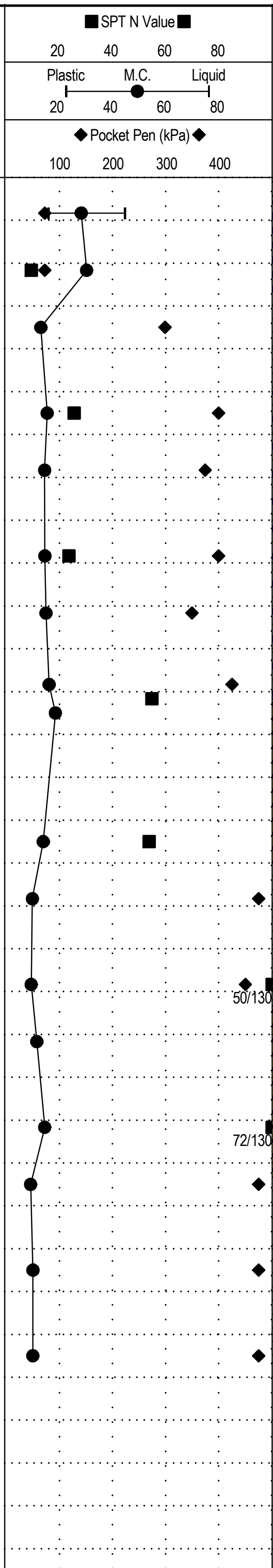
Completion Date: 22-4-20

Page 1 of 1



Depth (m)	USCS	Material Description	Sample #	SPT (N-Value)	Additional Data & Notes
0	OR	**TOPSOIL**, black, wet			
	CI/CH	**CLAY**, silty to and silt, trace sand, stiff, medium to high plastic, grey-brown, trace silt pockets, wet	1		Liquid Limit: 45% Plastic Limit: 16% Grain Size Distribution Gravel: 0 % Sand: 2% Silt: 48% Clay: 50%
1			2	10	
	CL	**CLAY (TILL)**, silty to and silt, sandy, trace gravel, very stiff, low plastic, grey-brown, trace oxides, moist	3		
2					
	CI	- becoming some sand to sandy, medium plastic	4	26	
3			5		
4			6	24	
			7		
5		- becoming very moist			
	SP	**SAND**, fine grained, some silt, trace clay, trace gravel, very dense, poorly graded, grey-brown, saturated	8	55	
6			9		Low recovery on the auger between 5.6 m and 6.9 m below ground surface
7		- becoming trace silt, trace clay nodules	10	54	
	CL	- becoming sandy to and sand, hard, low plastic	11		
8			12	50/130	
9	SP		13		
10			14	72/130	
	CL		15		
11		- becoming grey	16		
12			17		Water Level: 1.9 m below existing ground surface on 27 April 2022.

REFUSAL DEPTH: 12.6 m below ground surface

At Completion
Accumulation of water at 3.4 m below ground surface.
Accumulation of slough at 9.1 m below ground surface.
Slotted standpipe installed to 10.7 m below ground surface.
Borehole backfilled with drill cuttings and a bentonite plug.
- becoming some clay to clayey, very moist

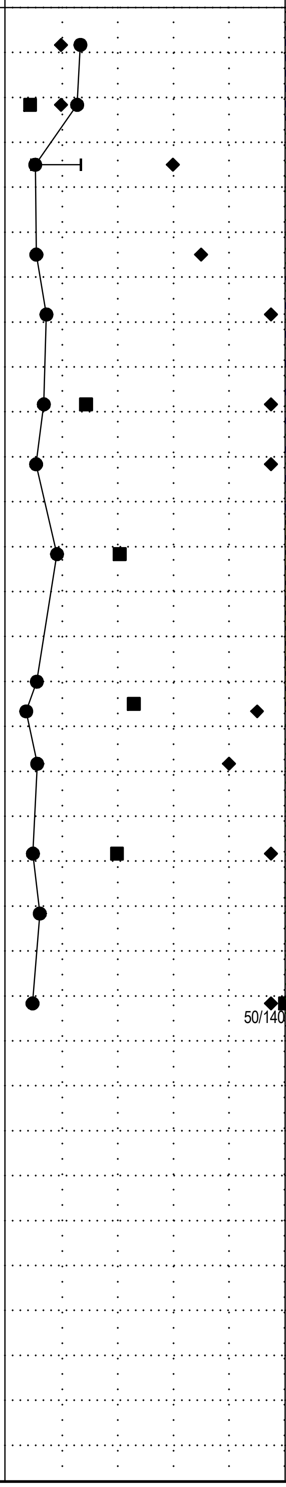
Depth (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Sample Symbol: Shelby Tube, No Recovery, SPT Test (N), Grab Sample, Split-Pen, Core

Backfill Symbol: Bentonite, Pea Gravel, Slough, Grout, Drill Cuttings, Sand

Project Name: Proposed SPS and WTP	Borehole #: **BH22-02**	SolidEarth GEOTECHNICAL	
Client Name: BCL Engineering	Project #: PG22-1628		
Site: Flying Dust First Nation - SW 36-59-17 W3M	Logged By: JS / Reviewed By: TF		
Northing: 6001818 Easting: 668092	Driller: All Service Drilling Ltd.	Completion Date: 22-4-20	
Elevation:	Drill Method: 150 mm Solid Stem Auger	Page 1 of 1	

Depth (m) | ■ SPT N Value ■ 20 40 60 80 | Plastic M.C. Liquid 20 40 60 80 | ◆ Pocket Pen (kPa) ◆ 100 200 300 400 | Soil Symbol | USCS | Material Description | Sample Symbol | Sample # | SPT (N-Value) | SLOTTED PIEZOMETER | Additional Data & Notes | Depth (m)



USCS	Material Description
OR	**TOPSOIL**, black, wet
CI/CH	**CLAY**, silty to and silt, trace sand, trace gravel, stiff, medium to high plastic, grey-brown, trace silt pockets, wet
CI/CL	**CLAY (TILL)**, silty to and silt, sandy, trace gravel, very stiff, low to medium plastic, grey-brown, trace oxides, moist
CI	- becoming some sand, medium plastic
SP	**SAND**, fine grained, some silt, trace clay, dense, poorly graded, grey-brown, very moist
	- becoming fine to medium grained, trace silt, moist
CL	**CLAY (TILL)**, sandy to and sand, some silt to silty, trace gravel, hard, low plastic, grey-brown, moist

Sample #	SPT (N-Value)
1	
2	9
3	
4	
5	
6	29
7	
8	41
9	46
10	
11	40
12	
13	50/140

Additional Data & Notes

Liquid Limit: 27%
Plastic Limit: 9%

SPT refusal - in contact with boulder

Low recovery on the auger between 5.3 m and 6.9 m below ground surface

Low recovery on the auger between 8.4 m and 9.9 m below ground surface

Water Level:
1.8 m below existing ground surface on 27 April 2022.

<u>**COMPLETION DEPTH: 10.4 m below ground surface**</u>

<u>At Completion</u>
Accumulation of water at 5.5 m below ground surface.
Accumulation of slough at 5.5 m below ground surface.
Slotted standpipe installed to 6.7 m below ground surface.
Borehole backfilled with drill cuttings and a bentonite plug.

Depth (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

Sample Symbol	Shelby Tube	No Recovery	SPT Test (N)	Grab Sample	Split-Pen	Core
Backfill Symbol	Bentonite	Pea Gravel	Slough	Grout	Drill Cuttings	Sand

Project Name: Proposed SPS and WTP	Borehole #: **BH22-03**
Client Name: BCL Engineering	Project #: PG22-1628
Site: Flying Dust First Nation - SW 36-59-17 W3M	Logged By: JS / Reviewed By: TF
Northing: 6001809 Easting: 668080	Driller: All Service Drilling Ltd.
Elevation:	Drill Method: 150 mm Solid Stem Auger

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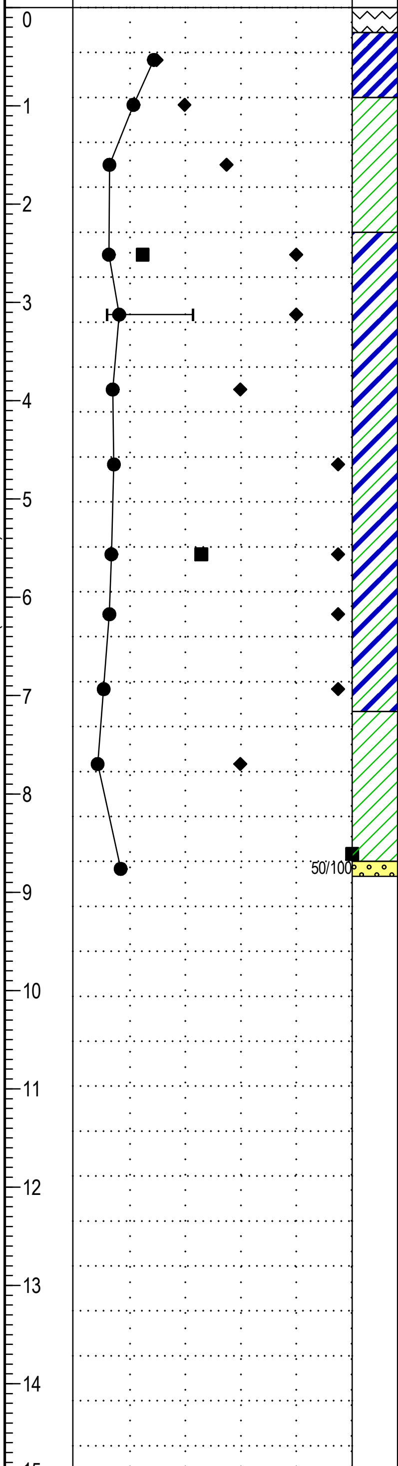
Completion Date: 22-4-20

Depth (m) | ■SPT N Value■ 20 40 60 80 | Plastic M.C. Liquid 20 40 60 80 | ◆Pocket Pen (kPa)◆ 100 200 300 400 | Soil Symbol | USCS | Material Description | Sample Symbol | Sample # | SPT (N-Value) | SLOTTED PIEZOMETER | Additional Data & Notes | Depth (m)



USCS	Material Description
OR	**TOPSOIL**, black, wet
CI/CH	**CLAY**, silty to and silt, trace sand, trace gravel, stiff, medium to high plastic, grey-brown, trace silt pockets, wet
CI/CL	**CLAY (TILL)**, silty to and silt, sandy, trace gravel, low to medium plastic, grey-brown, trace oxides, moist
CI	- becoming some sand, medium plastic, very stiff
	- becoming hard, grey, very moist
CL	- becoming sandy to and sand, low plastic, brown
SP	**SAND**, fine to coarse grained, trace to some silt, trace clay, trace gravel, very dense, poorly graded, grey-brown, saturated

COMPLETION DEPTH: 8.8 m below ground surface

At Completion
Accumulation of water at 7.3 m below ground surface.
No accumulation of slough material.
Slotted standpipe installed to 8.8 m below ground surface.
Borehole backfilled with drill cuttings and a bentonite plug.

Sample #	SPT (N-Value)
1	
2	
3	
4	25
5	
6	
7	
8	46
9	
10	
11	
12	50/100

Additional Data & Notes:

Liquid Limit: 43%
Plastic Limit: 12%
Grain Size Distribution
Gravel: 2 %
Sand: 26%
Silt: 30%
Clay: 42%

Water Level:
1.9 m below existing ground surface on 27 April 2022.

Sample Symbol: Shelby Tube, No Recovery, SPT Test (N), Grab Sample, Split-Pen, Core

Backfill Symbol: Bentonite, Pea Gravel, Slough, Grout, Drill Cuttings, Sand

EXPLANATION OF TERMS & SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of the field investigation and laboratory testing are described on the following two pages.

1. VISUAL TEXTURAL CLASSIFICATION ON MINERAL SOILS

CLASSIFICATION	APPARENT PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	> 200 mm	> 200 mm
Cobbles	75 mm to 200 mm	75 mm to 200 mm
Gravel	4.75 mm to 75 mm	5 mm to 75 mm
Sand	0.075 mm to 4.75 mm	Visible particles to 5 mm
Silt	0.002 mm to 0.075 mm	Non-plastic particles, not visible to naked eye
Clay	< 0.002 mm	Plastic particles, not visible to naked eye

2. TERMS FOR CONSISTENCY & DENSITY OF SOILS

Cohesionless Soils

DESCRIPTIVE TERM	APPROXIMATE SPT “N” VALUE
Very Dense	> 50
Dense	30 to 50
Compact	10 to 30
Loose	4 to 10
Very Loose	< 4

Cohesive Soils

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH	APPROXIMATE SPT “N” VALUE
Hard	>200 kPa	> 30
Very Stiff	100 to 200 kPa	15 to 30
Stiff	50 to 100 kPa	8 to 15
Firm	25 to 50 kPa	4 to 8
Soft	10 to 25 kPa	2 to 4
Very Soft	< 10 kPa	< 2

* SPT “N” Values – Refers to the number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter split spoon sampler for a distance of 300 mm after an initial penetration of 150 mm.

3. SYMBOLS USED ON BOREHOLE LOGS

SYMBOL	DESCRIPTION	SYMBOL	DESCRIPTION
N(■)	Standard Penetration Test (CSA A119 1-60)	SO_4	Concentration of Water-Soluble Sulphate
N_d	Dynamic Cone Penetration Test	C_u	Undrained Shear Strength
pp (♦)	Pocket Penetrometer Strength	γ	Unit Weight of Soil or Rock
q_u	Unconfined Compressive Strength	γ_d	Dry Unit Weight of Soil or Rock
w (●)	Natural Moisture Content (ASTM D2216)	ρ	Density of Soil or Rock
w_L	Liquid Limit (ASTM D 4318)	ρ_d	Dry Density of Soil or Rock
w_P	Plastic Limit (ASTM D 4318)	▽	Short-Term Water Level
I_P	Plastic Index	▼	Long-Term Water Level

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75 µm)	**GRAVELS** (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75mm)	CLEAN GRAVELS (LITTLE OR NO FINES)	**GW**	WELL GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = D_{60}/D_{10} > 4$ $C_c = (D_{30})^2/(D_{10} \times D_{60}) = 1$ to 3	
			GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		GRAVELS (WITH SOME FINES)	**GM**	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE I_p LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE I_p MORE THAN 7
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75mm)	CLEAN SANDS (LITTLE OR NO FINES)	**SW**	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = D_{60}/D_{10} > 6$ $C_c = (D_{30})^2/(D_{10} \times D_{60}) = 1$ to 3	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR SW	
		SANDS (WITH SOME FINES)	**SM**	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE I_p LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE I_p MORE THAN 7
FINE GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75 µm)	**SILTS** (BELOW 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50$ %	**ML**	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L > 50$ %	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS		
	CLAYS (ABOVE 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30$ %	**CL**	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS		
		$30 \% < W_L < 50 \%$	**CI**	INORGANIC CLAYS OR MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50$ %	**CH**	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (BELOW 'A' LINE)	$W_L < 50$ %	**OL**	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		$W_L > 50$ %	**OH**	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			**Pt**	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE	
BEDROCK			**BR**	SEE REPORT DESCRIPTION		

Soil Components			
Component	**Size Range (mm)**	**Descriptor**	**% by Weight**
Cobbles	> 76	and	> 35
Gravel	76 to 4.75		
Coarse	76 to 19	-y, -ey	35 to 20
Fine	19 to 4.75		
Sand	4.75 to 0.075	some	20 to 10
Coarse	4.75 to 2		
Medium	2 to 0.425	trace	10 to 1
Fine	0.425 to 0.075		
Fines (Silt or Clay)	< 0.075