NEWPCC
Primary Clarification Upgrade
New Scum Building
Geotechnical Investigation

NEWPCC

Project number: 60661262

May 24, 2022

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		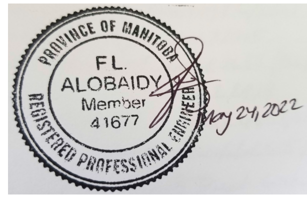	
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1. Introduction

AECOM Canada Ltd. (AECOM) was retained by the Winnipeg Sewage Treatment Program (WSTP) to undertake a geotechnical investigation as recommended in the previous draft geotechnical desktop review reports for the North End Sewage Treatment Plant (NEWPCC). The NEWPCC Primary Clarification Upgrade includes construction of a new scum dewatering building west of the existing Primary Clarification Facility.

This Report summarizes the field investigation, methodology, and subsurface conditions encountered. Based on the results of the geotechnical investigation program, general recommendations are provided. The test hole logs are included in **Appendix A**. Laboratory testing results are included in **Appendix B**.

1.1 Scope of Work

The scope of work for this geotechnical investigation program included the following:

- Execution of geotechnical field program which includes:
  - Preparation of safety documentation including a Health and Safety Plan
  - Drill a test hole at within the vicinity of the proposed new scum dewatering building
  - Arrange clearance for underground utilities and structures
- Preparation of this Report includes:
  - Description of the field investigation
  - General description of subsurface soils encountered
  - Observed groundwater conditions
  - Test hole layout plan showing the locations of nearby test holes
  - Logs of the test holes (including Standard Penetration Test and laboratory results)
  - Presentation of the laboratory testing results
  - Subgrade preparation recommendations
  - Foundation recommendations
  - Piling recommendations with pile parameters for cast-in-place concrete piles

2. Methodology

2.1 Review of Available Resources

Geotechnical and hydrogeological reports from previous projects at the NEWPCC were reviewed to provide general information about the site subsurface and groundwater conditions. The reviewed reports are listed below:

- Sludge Digestion Expansion Geotechnical Report (June 1984)[1]
- Hauled Liquid Waste Facility Geotechnical Report (December 2009)[2]
- NEWPCC Power Supply Upgrade Geotechnical Report (October 2015)[3]
- NEWPCC Upgrade Geotechnical Report (June 2017)[4]
- North End STP Headworks Facilities Project – Supplemental Field Investigation (July 2020)[5]

A drawing showing the closest test hole locations to the proposed scum dewatering building location from these reports is shown in **Figure 2.1**.

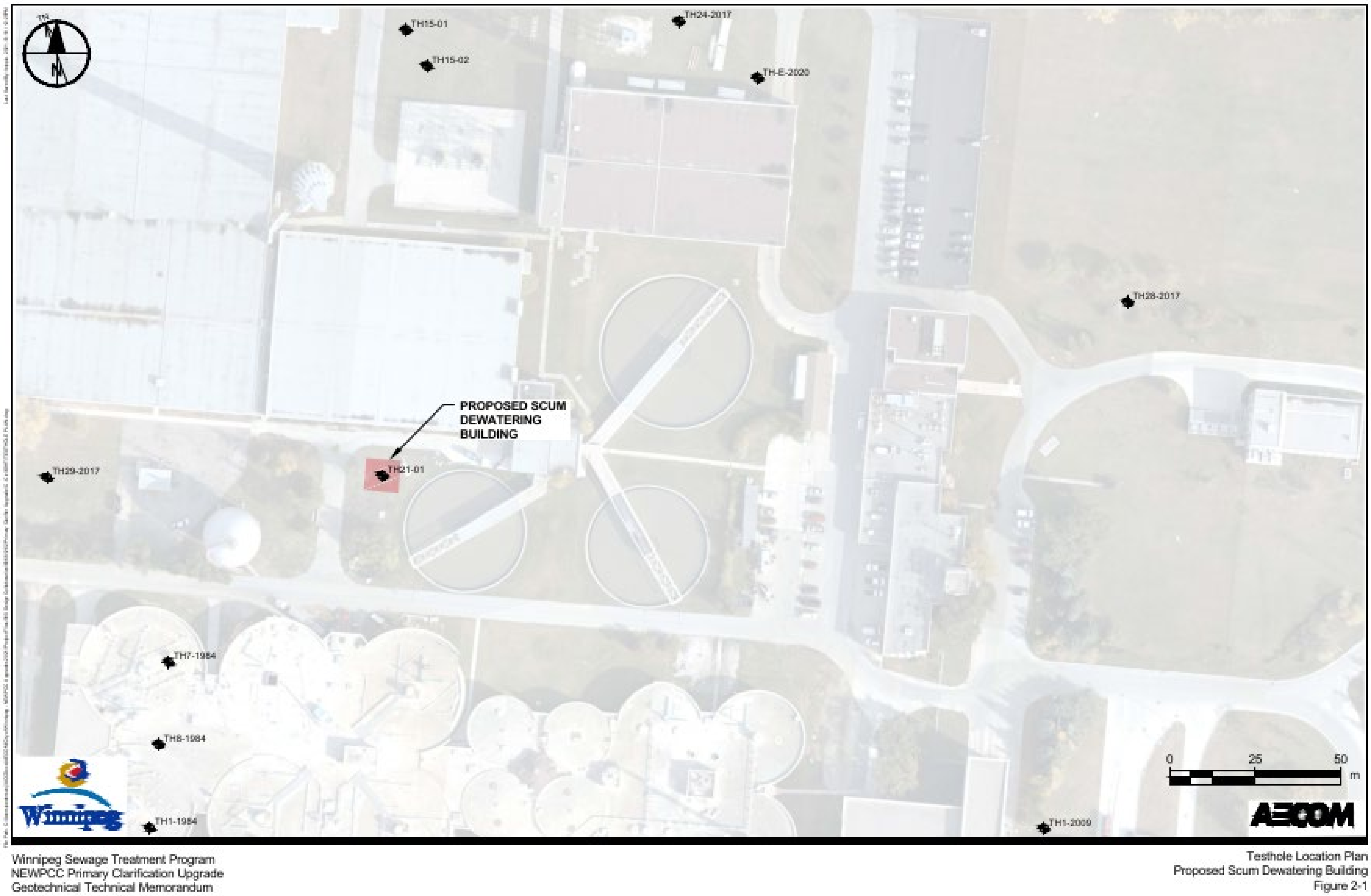

Figure 2.1: Test Hole Location Plan

The general subsurface conditions as determined from the nearest test holes TH1, TH7, and TH8 (from the 1984 Sludge Digestion Expansion Geotechnical Report), TH15-02 (from the 2015 PSU Geotechnical Report), and test holes TH24, TH28, and TH29 (from the 2017 NEWPCC Upgrade Geotechnical Report) consisted of 150 mm thick topsoil, underlain by dry to moist clay fill of 1.0 to 1.70 m thick, underlain by clay (CH) varying in thickness from 0.50 to 1.40 m, underlain by silt of 0.50 to 1.25 m thick. The underlying clay (CH) below the silt was stiff to firm, highly plastic, and had the tendency to swell and shrink with change in moisture content. A layer of glacial till of 1.53 to 7.06 m thick was encountered at approximately 18.8 meters Below Ground Surface (mBGS). The till was very hard near auger refusal depth. The upper 0.5 to 1 m of till was saturated. Occasional silty sand layer was also encountered within 5 m from the surface.

Table 2.1 summarizes the soil stratigraphy and parameters from the existing reports used for the preliminary geotechnical recommendations for the proposed scum dewatering building.

Table 2.1: Soil Stratigraphy and Parameters

Approximate Depth Below Ground Surface (mBGS)	Soil Description	Unconfined Compressive Strength	Undrained Shear Strength	Soil Index Properties
0 to 0.15	Topsoil	-	-	-
0.15 to 1.7	Clay Fill – trace silt and fine gravel, stiff, high plastic, and moist	-	-	Gravel: 0.1%, Sand: 1.1% Silt: 29.3% Clay: 69.5%
1.7 to 3.1	Clay – stiff, highly plastic, and moist	117.1 kPa	60 kPa	Gravel: 0 %, Sand: 5.0% Silt: 21.9% Clay: 73.1%
3.1 to 4.35	Silt – soft, low plasticity, and mostly saturated	-	15 kPa	Gravel: 0 %, Sand: 4.5% Silt: 80.3% Clay: 15.2%
4.35 to 18.8	Clay – firm to stiff, highly plastic, and moist. Soft below 17 mBGS.	80 to 57 kPa	40 to 28 kPa	Gravel: 0.8%, Sand: 7.7% Silt: 34.8% Clay: 56.7% LL: 60%, PL: 19%, PI: 41%
Below 18.8	Silt Till – fine to coarse gravel, compact, low plastic and moist. Presence of cobbles and boulders in some areas.	-	46+ kPa	-

LL= Liquid Limit, PL= Plastic Limit, PI= Plasticity Index

Source: All reviewed existing reports

2.1.1 Groundwater Conditions

It is understood that two water bearing layers exist at the NEWPCC site: the perched water table in the silt layer at shallower depths and the static groundwater level in the water bearing zone in the lower part of the till.

The groundwater level ranged from 1.04 to 7.27 mBGS (Elev. 227.26 to 196.13 m Above Sea Level (ASL)). Groundwater levels fluctuate during the year and will be dependant on precipitation, surface drainage, and regional groundwater regimes. Groundwater seepage was observed throughout the site within the silt layer. Soil sloughing was observed below depths of 3 m and 19 m (Elev. 227.46 to 211.46 mASL).

2.2 AECOM 2021 Design Investigation

AECOM conducted a geotechnical investigation which consisted of drilling a 20 m deep test hole TH 21-01 at the proposed location of the new scum building. Drilling was started and completed on November 9, 2021. The location of the test hole is shown on **Figure 2-1**.

Test hole was logged by AECOM personnel at the time of drilling based on observation of the drill cuttings. The soils were classified according to the Modified Unified Soil Classification System (mUSC). During auger drilling, samples were taken at regular intervals alternating between grab and split spoon samples. SPTs were carried out at 1.5 m intervals.

A standpipe piezometer was not installed in the test hole which was backfilled with drill cutting upon drilling completion.

2.2.1 Laboratory Testing

Selected samples from the test holes were taken to AECOM's Winnipeg Material Testing Laboratory for soil classification and determination of index properties. The laboratory testing included the following:

- Moisture Content Determination
- Atterberg Limits Testing
- Grain Size Analyses (Sieve/Hydrometer)
- Standard Proctor Maximum Dry Density (SPMDD) Tests
- Unconfined Compressive Strength Tests

The test results are shown on the test hole log (**Appendix A**) and are presented in **Appendix B**.

2.2.2 Utility Locates

Prior to any ground disturbance, a One Call request was submitted by AECOM. Third-party locates were also arranged by City employees, and a City ground disturbance checklist was completed prior to commencing the drilling test hole program.

3. Subsurface Conditions

The site investigation consisted of drilling a test hole TH21-01 to a depth of 20 mBGS. The following subsections provide a summary of the various soils encountered during the site investigation.

3.1.1 Topsoil

Topsoil was encountered at the ground surface and was 150 mm thick.

3.1.2 Fill

Granular fill was encountered below the topsoil. The fill was 450 mm thick and contained some organics.

3.1.3 Clay

Clay was encountered below fill at 0.6 mBGS and extended to a depth of 18.3 mBGS. Up to 2.6 mBGS the clay contained trace silt, trace sand and trace gravel. The clay was firm, moist and brown in color. At 2.6 mBGS the clay became silty.

At 3 mBGS a silt layer of 0.6 m thick was encountered. The silt contained trace sand, trace clay, was of medium plasticity, was soft to firm, moist and light brown in color.

Below 3.6 mBGS the clay changed in plasticity from high to medium. The clay was very stiff at 6.3 mBGS and soft between 9 and 11.5 mBGS. The clay was firm below 11.5 mBGS.

The moisture content in clay ranged between 23.4 and 67%, with an average of 48.2%.

Atterberg Limits and grain size analysis results are presented in **Table 3.1**.

Table 3.1: Summary of Atterberg Limits and Grain Size Analysis Test Results in Clay

Sample Number	Depth (mBGS)	mUSC	Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
S4	6.1	CH	51.4	78	28	50	0	2.58	23.27	74.15
S8	12.2	CI-CH	48.5	49	21	28	0	9.12	35.39	55.49

Two unconfined compressive strength tests carried out on samples of clay from 4.6 mBGS and 7.6 mBGS resulted in 83.1 kPa and 112.6 kPa, respectively.

3.1.4 Glacial Till

Glacial Till was encountered at 18.3 mBGS underlying clay and extended to test hole termination depth of 20.3 mBGS. The glacial till was moist and brown in color with an SPT ‘N’ number of 10 indicating that till was of firm to stiff consistency.

Moisture content in till ranged between 9.4 and 12.2% with an average of 10.8%.

3.2 Groundwater

Monitoring well was not installed in the test hole. Groundwater level was monitored during the drilling with seepage observed at 5.5 mBGS. No sloughing was observed during the drilling.

4. Site Preparation and Grading

4.1 General

The site is considered suitable for construction from a geotechnical perspective provided recommendations in this report are incorporated in the design and construction of the geotechnical elements of the project. The primary geotechnical concerns for the site are:

- Potential soil sloughing and groundwater seepage from the shallow silt layer during installation of cast-in-place piles and excavation for the pile caps/perimeter slabs during construction;
- High groundwater and potential soil sloughing and groundwater seepage from the shallow silt and till layers while performing open excavations for utility connections at depths ranging from 1 to 2 mBGS;
- Settlement of the subgrade due to the potential grading work and fill placement;
- Vibration caused by driven pile installation during construction may cause damage to existing structures and utilities; and
- Movement related to heave/shrink of the high plasticity clay with changing of moisture content.

In this report, existing ground elevation refers to unstripped, original ground elevation based on the survey provided to AECOM.

4.2 Site Preparation

Clearing and grubbing will be required for the site prior to grading activities. Throughout the proposed construction site, all topsoil, organics, and all unsuitable material should be removed. Organics should be stripped from development areas and stockpiled separately from inorganic material for use as future reclamation material or for landscaping purposes.

Initial grading operations should also be focused on providing surface drainage improvements such that precipitation and surface runoff is directed off the construction area. Also, soft and wet soils should be removed completely from the footprint of settlement-sensitive structures; this may require sub-excavation.

After removal of organic material and/or topsoil, the exposed subgrade will likely be sensitive to disturbance. Construction traffic over the exposed subgrade may cause subgrade deterioration such as rutting and subgrade failure and localized deep repairs may be required. Therefore, access of construction traffic should be restricted over the exposed subgrade until it has been tested and soft/weak areas stabilized by excavating and replacing them with competent material and compacting as described below.

Following stripping, the exposed subgrade should be proof-rolled to identify soft, wet, and weak areas (refer to **Section 6.3** for guidance pertaining to proof-rolling). Proof-rolling should be completed under the supervision of qualified geotechnical personnel. All identified soft or weak areas should be improved by removing all soft/weak soils and replacing them with compacted General Engineered Fill (See **Section 4.3** for the definition of General Engineered Fill) and/or by employing other suitable means.

Improvements to subgrade may include the following:

- Sub-excavate soft soil and backfill with low to medium plasticity clay/clay till. The clay/clay till should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD) within ± 2% of the Optimum Moisture Content (OMC) unless otherwise specified.

Under settlement-sensitive structures (such as buildings, foundations, etc.) the backfill should be compacted to at least 100% of the SPMDD. This option will be suitable if a competent subgrade is present within a relatively shallow depth (e.g. less than 1 m); or,

- Placement of a non-woven geotextile in combination with a geogrid (if granular material is used) or geogrid alone (if fine grained material is used) at the base of the sub-excavation and backfilling with compacted General Engineered Fill or Structural Fill. The backfill should be compacted to at least 98% of the SPMDD within ± 2% of the OMC unless otherwise specified. Under settlement-sensitive structures (such as tanks) the backfill should be compacted to at least 100% of the SPMDD. This option may be required if soft subgrade conditions extend to great depths (e.g. in excess of 1 m); however, complete removal of soft clay is recommended under settlement-sensitive structures.

Full-time monitoring and compaction testing should be provided during any fill placement to confirm that the conditioned material is prepared in accordance with the recommendations in this report. This monitoring should be carried out by qualified geotechnical personnel.

4.3 General Engineered Fill

General Engineered Fill should be comprised of clean, inorganic low to medium plasticity clays or well graded granular soil.

Low to medium plasticity inorganic clay or clay till having the following range of Atterberg Limits is generally considered suitable for use as General Engineered Fill.

- Liquid Limit .... 20 to 40%
- Plastic Limit .... 10 to 20%
- Plasticity Index .... 10 to 30%

All the Atterberg Limits test results completed on material obtained from within the proposed new building footprint were not within the above ranges. Silt and silt till samples obtained from this area should not be used as general engineered fill.

General Engineered Fill may be used as fill material for roads, berms, and general site grading. It is not recommended to be placed directly under the building foundations, heavily loaded floor slabs, or other potentially settlement-sensitive structures.

4.4 Structural Fill

Structural Fill should be used under foundations, heavily loaded floor slabs or any other settlement sensitive structures. Structural Fill should consist of well graded gravel free from organic or soft material. The structural fill should conform with the grading requirements provided in the City of Winnipeg Excavation and Backfill document CW 2030. Structural Fill may not be available at the site and must be imported from off-site sources.

Structural Fill should be compacted to 100% of SPMDD within ± 2% of the OMC. Layers of Structural Fill placed below structures shall extend outward away from the structure a minimum distance equal to the layer thickness. The placement and compaction of Structural Fill shall be monitored full-time by qualified geoechnical personnel, and the moisture content and density of the compacted Structural Fill shall be confirmed by field density testing.

If Structural Fill is placed and compacted in accordance with the recommendations in this report, then it is expected to settle less than 0.5% of the fill thickness under self-weight.

Recommended gradations for pit-run and crushed gravels are provided in **Table 4.1** and **Table 4.2**.

Table 4.1: Recommended Gradation – “Pit-Run” Gravel

Canadian Metric Sieve Size (mm)	Type 1 Material – Pit-Run
75	90% to 100%
28	80% to 100%
5	40% to 80%
0.315	10% to 35%
0.080	5% to 30%

Table 4.2: Recommended Gradation – Crushed Gravel

Canadian Metric Sieve Size (mm)	Type 2 Material
28	-
20	100%
5	40% to 70%
2.5	25% to 60%
0.315	8% to 25%
0.080	6% to 17%

4.5 Surface Drainage

Good surface drainage should be provided during and after construction to reduce ponding on or near the structures. Ponding on or near structures may result in foundation failure. Subgrade (i.e. the top of the clay layer) shall be sloped at 3% toward drainage pipes or ditches prior to covering with gravel. A minimum surface gradient of approximately 2% for gravel surfaced areas is recommended to reduce ponding and to direct water towards catch basins or ditches. A slope of 1% may be considered for areas located away from structures; however, ponding in areas containing shallow slopes may occur leading to degradation of the subgrade.

Ditches should also be properly graded to promote positive drainage. The native soils in the area are erosion susceptible; therefore, ditch gradients in excess of 2% may cause ditch erosion. If ditch gradients exceed 2%, analysis should be completed to determine the velocity of flow and whether erosion protection is required. It should be noted that gradients less than 2% may also cause erosion if the runoff gains sufficient velocity; however, the likelihood of erosion occurring is reduced.

To reduce potential for ponding, the desirable minimum longitudinal gradient is 0.5% for ditches with base width ≤ 3 m (or as per City of Winnipeg Specifications). However, at some locations a minimum longitudinal ditch gradient of 0.5% may not be achievable. In these situations, longitudinal gradients flatter than 0.5% may be considered; however, the longitudinal gradients should not be flatter than 0.2% in any case (or as per City of Winnipeg Specifications) with the understanding that some ponding may occur, and additional maintenance or ditch lining may be required.

Downspouts from buildings and structures may be discharged onto landscaped or gravel surface areas provided that water is carried by means of a concrete splash pad or extendable sections so that the point of discharge of water is at least 2 m from building walls. The ground surface adjacent to buildings should be graded to slope away from buildings at a gradient of at least 5% within 2 m of the structure perimeter.

4.6 Temporary Excavations and Backfill

The composition and consistency of the soils encountered at the site are such that conventional hydraulic excavators should be able to excavate these materials, but a ripper may be required to excavate seasonally frozen soils.

All excavations shall be in accordance with the provisions of the Occupation Health and Safety Regulations (OHS, latest edition). The excavation walls should be sloped or adequately shored. The appropriate side slope that is required will depend on the type of soil, depth of excavation, drainage method, the amount of groundwater seeping into the excavation, and the time interval the excavation is left open.

The Contractor shall be responsible for adequately shoring or sloping the temporary excavations in accordance with the soil conditions provided in this section and OHS guidelines. It should be noted that groundwater levels provided in this report (1.04 to 7.27 mBGS (Elev. 227.26 to 196.13 mASL) are high. The Contractor shall assume groundwater above the excavation base if excavations required (not finalized at the time of writing this report) and be responsible for temporary dewatering of the excavation during construction. The Contractor will be responsible for maintaining stability of the slopes or shoring system as well as protection of any existing infrastructure located near the temporary excavations.

The slopes should be checked regularly for signs of sloughing, especially if loose sand pockets are observed or after inclement weather conditions. The amount of time a trench is left open should be minimized as stability decreases over time. If there are signs of movement, the side slopes should be unloaded by benching the upper portion of the crest of the slope to relieve overburden pressure. The temporary cut slopes should also be protected against surface runoff and heavy rainfall. Small earth falls from the side slopes are a potential source of danger to workers and must be guarded against.

If seepage is encountered in the excavation extending through cohesive soils, groundwater accumulations should be handled by grading the base of the excavation to a sump from which water can be pumped away.

Fill should only be placed over dry, clean, stiff, unfrozen soils. The site soils are susceptible to softening and deterioration if left exposed in an excavation; therefore, traffic on the excavation base should be minimized, and construction should commence immediately after the excavation is complete. The time the excavation is left open should be minimized.

Temporary surcharge loads, such as construction materials or excavated soil (spoil piles), should not be allowed within 1.5 m or a distance equal to the depth of the excavation, whichever is greater, of an unsupported excavated face. Vehicles delivering materials should be kept back from faces by at least 3.0 m.

The method of excavation and safe support of excavations, selecting suitable slopes for excavations, selecting temporary shoring system, protection of the existing infrastructure and maintaining stability of the excavation slopes are the responsibility of the Contractor.

4.7 Sulphate Attack and Corrosion

During the review of reports mentioned in **Section 2.1**, it was found that the clay soils in Winnipeg area contain sulphates that will cause deterioration of concrete. All concrete utilized in foundation elements should have a minimum specified 56-day compressive strength of 35 MPa and class of exposure of S-1, corresponding to very severe sulphate attack as per CSA A23.1-19.

5. Recommendations for the Proposed Scum Dewatering Building

The scum dewatering building was originally designed as part of the NEWPCC Upgrade Enhanced Preliminary Design (EPD) in 2019 (S0972-12DP-RPT-0001). Based on the EPD, the building will be a two-story structure approximately 8.5 m wide by 10.5 m long and approximately 6.7 m high, all above grade. The main floor will consist of an open loading area containing two bins and a stairwell. The upper level will be a mezzanine 4.7 m wide by 10.5 m long with two dewatering units above the two bins on the main floor. The main floor will be a 300 mm thick concrete slab supported on perimeter slabs and pile foundation. The mezzanine will be a 250 mm thick concrete slab supported on the exterior walls and interior concrete columns. Walls of the new building will be 200 mm thick concrete masonry and the roof will be a 200 mm thick concrete slab supported on the exterior walls and interior concrete columns. No changes to the building design are anticipated as part of the Primary Clarification Upgrade.

At this stage of preliminary design, total structural and operation loads are not known. The design recommendations provided in the following sections are based on soil strength from the test hole TH21-01 drilled at the proposed building location.

5.1 Overview

The site is considered suitable for driven steel piles and cast-in-place (CIP) concrete piles. However, CIP concrete piles are considered preferable to driven steel piles due to close proximity to existing building structures and buried utilities. These conditions can result in potential for disturbance to the utility and/or damage to surrounding foundation from the vibrations resulting from driven pile installation.

The following subsections provide recommendations CIP concrete piles.

5.2 Cast-in-Place (CIP) Concrete Piles

5.2.1 General

Straight shaft drilled CIP concrete piles are preferred at this site subject to recommendations provided in the following subsections. These recommendations are subject to a concrete plant being located within a driving distance of 90 minutes for the concrete to be used within the recommended two hours of mixing. The recommendations presented in the following subsections may be used for design of CIP concrete piles.

5.2.2 Pile Design

The ultimate capacity of straight shaft CIP concrete piles may be determined from the following equation.

$Q_u = q_s\, P_s\, L + q_t\, A_t$

Where:

Q_u = ultimate capacity of the pile (kN)
q_s = ultimate skin friction between the pile and soil (kPa)
q_t = ultimate end bearing (kPa)
P_s = perimeter of the pile section (m)
= π x d, where "π" is 3.14 and "d" is the diameter of the pile in metres
L = effective pile embedment length (deducting the frost depth, height of fill, etc.)

A_t = cross sectional area of the pile (m^2)
= π $d^2/4$, where "π" is 3.14 and "d" is the diameter of the pile

For limit states design a resistance factor of 0.4 should be applied on the ultimate pile load capacity to obtain the factored pile load capacity. The resistance factor can be increased to 0.5, 0.55, or 0.6 if PDA, Statnamic, or static pile load testing are conducted, respectively (Skirrow and Wang, 2008).

For working stress design, a factor of safety of 2 and 3 should be applied on ultimate skin friction and ultimate end bearing, respectively, to obtain allowable skin friction and allowable end bearing.

5.2.3 CIP Concrete Pile Design Parameters

The axial capacity of CIP piles may be determined using parameters provided in **Table 5.1** and the equation provided in **Section 5.2.2**. The CIP concrete pile parameters are for the proposed new scum dewatering building location. A 600 mm diameter pile has been assumed. In order to reduce ambiguity, parameters are presented in terms of elevation for this area.

Table 5.1: Ultimate Design Parameters for CIP Concrete Piles

Elevation (m)	Ultimate Skin Friction (kPa)	Ultimate End Bearing Resistance (kPa)
231 - 228	-	-
228 – 226.5	32	-
226.5 – 223.2	40	-
223.2 – 219.5	15	-
Below 219.5	32	300

The pile design parameters in **Table 5.1** are considered applicable for downward (compressive) static loads. Recommendations for uplift loads are provided in **Section 5.4**. Recommendations for laterally loaded piles are provided in **Section 5.6**.

Negative skin friction due to settlement of fill and native soils should be considered in design of the piles in areas where fill will be placed (**Section 5.5**).

General design and construction recommendations for CIP concrete piles are provided in **Section 5.2.4**.

5.2.4 CIP Concrete Pile Design and Construction Recommendations

The following recommendations should be considered when designing and constructing the CIP concrete piles:

- Skin friction should be neglected within either the zone of seasonal frost penetration to account for the effects of soil desiccation and frost heave or the depth of fill, whichever is greater.
- Negative skin friction due to settlement of fill and soft subgrade should be considered in design of the piles.
- Piles should be founded at sufficient depth to resist uplift pressures due to frost. An uplift pressure of 65 kPa for frozen soils to concrete should be considered for the maximum frost penetration depth (3 mBGS). The minimum embedment depth to resist uplift due to frost will

be a function of the pile shape (e.g. whether bells are used or not) and of the size. For example, ignoring the effects of self-weight of a pile, a 600 mm diameter CIP concrete pile will require installation to approximately 9.5 mBGS to adequately resist uplift pressures due to frost assuming no dead load applied on the pile.

- Shaft and end-bearing resistance of CIP concrete piles should be designed using the site-specific design parameters provided in **Section 5.2.3**. These site-specific values were estimated based on SPT N-values in combination with laboratory testing and empirical correlations.
- A minimum pile spacing centre-to-centre of 3 times the shaft diameter is recommended for straight shaft piles.
- Piles within three shaft diameters should not be drilled or poured consecutively within the same 48-hour period to allow the concrete in the adjacent piles to set.
- Concrete piles must be reinforced for the full length of the pile.
- The contractor should be prepared to control seepage and sloughing and maintain clean pile holes. Temporary steel casing may be required to prevent excessive seepage and sloughing into the pile holes during excavation and pouring of concrete. Based on observations provided on the test hole logs, silt layers and corresponding seepage may be encountered at any depth. The contractor should bring enough casing to case the entire pile hole should the need arise.
- The contractor should evaluate means and methods to install/extract casing.
- The use of piles with an enlarged base (belled piles) is not recommended due to the presence of wet lenses of cohesionless material present at various locations throughout the site. There is a risk that the cohesionless material will slough into the reamed cavity.
- The foundation contract should have provisions for lengthening the pile, casing, and steel cage if required due to site subsurface conditions.
- Bases of all end bearing piles must be thoroughly cleaned of all loosened material and dewatered prior to pouring concrete. The base should be inspected by qualified personnel. End bearing will not be applicable if pile bases are not properly cleaned and inspected prior to placement of concrete.
- To avoid segregation of the concrete, a tremie tube should be used when placing concrete below the water table. The tremie tube should be watertight, and the outlet of the tremie tube should be at least 1 m below the concrete surface during pouring.
- Concrete should be poured immediately after drilling of the pile hole to reduce the risk of groundwater seepage and soil sloughing.
- Monitoring of the pile installation by qualified personnel is recommended to verify that the piles are installed in accordance with design assumptions. Inspection should be carried out before casting the pile.

5.3 Frost Design Considerations – Piles, Perimeter slabs and Pile Caps

5.3.1 General

CIP concrete piles designed using the parameters in **Section 5.2.3** should be founded at a minimum depth of 9.5 m below the finished grade to resist frost heave, depending on the size of pile.

The above minimum penetration depths should be reviewed once the final pile dimensions and applied loads are known.

5.3.2 Unheated Structures

Frost action, such as uplift due to frost heave on the underside of perimeter slab/pile caps, and adhesion freezing forces (adfreeze) along the pile shaft and sides of perimeter slab/pile caps within the seasonal frost zone, should be considered in pile design. In accordance with CFEM (2006) the adfreeze bond stresses on unheated pile shafts in the seasonal frost zone may range from 65 kPa (for concrete piles within frost depth) to 100 kPa (for steel piles within frost depth). Therefore, the pile embedment below the seasonal frost zone should be sufficient to resist the uplift due to frost heave. The minimum pile embedment to resist frost heave should be reviewed considering dead loads and ignoring the downdrag loading due to the placement of fill.

Perimeter slabs and pile caps in unheated areas should also be protected from frost heave by burial below the seasonal frost depth. Perimeter slabs and pile caps that do not have adequate soil cover should be protected from frost heave by providing a void form or a void space underneath them. Placing a compressible void form or providing a void space between the ground and the underside of the perimeter slabs/pile caps will reduce the potential for frost heave forces. The perimeter slabs and pile caps should be designed in accordance with the crushing strength of the void form.

It is important that water not be allowed to pond near or under the pile caps and perimeter slabs. Ponding near or adjacent to the structure may saturate and/or damage the void form resulting in uplift on the underside of perimeter slabs and pile caps. Therefore, the finished grade adjacent to perimeter slabs and pile caps should be capped with well-compacted clay and adequately sloped away from the structure.

Adfreeze will also act on the sides of the pile caps and perimeter slabs. Because adfreeze forces are a function of the soil type, moisture content, salinity of the pore water, composition of the vertical surface (concrete or steel), and the ground temperature, the adfreeze forces acting on the pile caps or perimeter slabs can be reduced by placing dry, non-frost-susceptible granular soil (with less than 5% fines) around pile caps/perimeter slabs, by providing good drainage, and/or by applying a frost bond breaker to the faces of pile caps and perimeter slabs.

5.3.3 Heated Structures

The frost effect on external perimeter slabs and pile caps of heated structures can be reduced by placing void form underneath the pile caps and perimeter slabs, placing dry non-frost-susceptible granular soils (with less than 5% fines) against them, and providing good drainage. Alternatively, perimeter insulation can be used for the external perimeter slabs and pile caps.

If insulation is used, it should be made of rigid polystyrene composition (Styrofoam HI-40 or equivalent). The Styrofoam is not resistant to hydrocarbons; therefore, hydrocarbon resistant insulation, such as Foamglas, may be required. The insulation should be at least 75 mm thick. Insulation should extend horizontally outwards away from the building a minimum distance of 2.5 m. Insulation should be placed over a layer of bedding sand, at least 300 mm in thickness, and should be sloped down away from the structure at 1%. The minimum burial depth of insulation outside the walls from finished grade to the top of insulation should be 500 mm. A compacted clay layer approximately 300 mm thick is recommended at the surface to reduce infiltration.

5.3.4 Drainage

Placement of a sub-drain (weeping tile system) below the base of pile caps should be used to provide drainage around the sides of the pile caps and perimeter slabs. The drainage system will reduce potential adfreeze forces on the sides of pile caps and perimeter slabs and maintain water below the base of the structures.

5.4 Tension Loading

Piles will be subject to uplift forces due to frost heave, tensile forces due to lateral loading, overturning movements due to wind, etc. The piles should be designed to resist these uplift forces. The resistance to uplift will be provided by pile self-weight, applied dead loads, and uplift skin resistance. Factors such as seasonal frost depth, heating and insulation, and soil type should be taken into account while designing the pile against uplift.

The resistance to uplift may be calculated using ultimate skin friction parameters provided in **Section 5.2.3** of this report. A resistance factor of 0.3 should be applied on ultimate parameters to obtain factored parameters. This resistance factor is in accordance with the CFEM (2006).

5.5 Downdrag Loading

In areas of fill placement, the ground surface will settle after placement of the fill. Piles will be subject to downdrag forces (negative skin friction) due to settlement of the fill and subgrade soil around the piles due to fill placement. The downdrag will add to the load on the pile; therefore, it should be considered in the design of the structural capacity of the piles. Downdrag loads are added to sustained loads, thus two loading cases must be considered: (1) permanent dead load plus downdrag load, and (2) permanent load plus transient live load. Downdrag increases the pile settlement and, therefore, should be accounted for when evaluating the serviceability limit state of the pile. The downdrag has no effect on the ultimate geotechnical axial capacity of the pile.

The CFEM (2006) uses the neutral plane concept to calculate the depth where shear stress along the pile changes from negative skin friction to positive shaft resistance. At the neutral plane, the relative movement between the pile and surrounding soil is zero. The depth of the neutral plane will depend on the load distribution along the pile in conjunction with the resistance distribution along the pile. For preliminary design of the pile, the neutral plane depth may be assumed at two-thirds times the pile length for skin friction piles; however, the neutral plane depth should be verified during design of the piles.

The pile length for downdrag loads should be determined using the following three criteria:

Geotechnical Capacity (ULS Condition): Use dead load plus live load but no downdrag load (Section 18.2.5.1 (4) of the CFEM (2006)). The pile is designed by applying appropriate factors to the loads and on geotechnical resistance. This case is similar to designing a pile without downdrag load.

Structural Capacity: Apply load factors to dead load and downdrag load. The factored dead load plus factored downdrag load should not exceed the structural capacity of the pile assuming an acceptable factor of safety based on structural design criteria.

Serviceability Limit States (SLS Condition): The pile length for SLS conditions should be estimated using service dead load plus downdrag load (without applying any load factors) and unfactored geotechnical capacity of the pile (below the neutral plane). The pile settlement below the neutral plane is expected to be approximately 20 mm for CIP concrete piles, assuming a pile diameter of 600 mm and a minimum pile length of 18 m. The elastic compression of the pile above the neutral plane should be added to this value to calculate the total estimated pile settlement.

Negative skin friction within fills should be considered while settlement of the fill and subgrade is ongoing. Settlement of fill is estimated to take less than one year for cohesionless or low plasticity clay fill. The downdrag load depends on the type of fill material. An average downdrag (in kPa) of 4.5 times the fill thickness (in m) for cohesionless fills and 40 kPa for cohesive fills should be applied on the pile length within the fill.

Negative skin friction within native soils that are subject to settlement should also be considered. A negative skin friction of 40 kPa should be applied on the pile length within the native compressible subgrade. The negative skin friction should be applied to the pile length within the native soils (from existing ground surface to the neutral plane). It should be noted that settlement of native soils may also occur during dewatering. If the groundwater is to be appreciably lowered for any length of time, negative skin friction resulting from the corresponding settlement of the native material must also be considered.

Downdrag loads from the fill can be minimized by providing a casing or sleeve around abutment piles (for driven piles) or can be reduced by 50% using a bituminous coating and/or polyethylene applied to the pile shaft within the fill.

In general, inclined or battered piles should be avoided in settling subgrade soils, or, at least, the angle of inclination of the piles should be limited to prevent excessive bending of the piles.

5.6 Lateral Loading

Vertical piles will be subjected to horizontal loads in addition to vertical loads; their lateral capacity should be checked by a proper analysis (i.e. LPile Analysis). Short term lateral loads may be imposed by construction, by seismic forces or by wind. Long term forces may be those acting on supports of an above ground structures.

Design of laterally loaded piles is generally governed by Serviceability Limit States limiting the top of pile movement to within tolerable limits.

Lateral load capacity of piles will depend upon the pile stiffness and geotechnical engineering properties of the native soil or fill material within the upper few metres of the pile. Lateral pile capacity can be determined using commercially available software such as LPile. The analysis using this software provides estimates of the lateral displacements, bending moments, shear forces and soil reaction along the depth of the piles, and it requires input pertaining to soil properties, pile properties, and applied loads on the pile.

Table 5.2, presents the recommended soil profiles for LPile analysis at the proposed scum dewatering building.

Table 5.2: Soil Parameters for LPile Analysis

Soil Type	Elevation (m)	γ (kN/m^3)	Undrained Shear Strength, c_u (kPa)	p-y Modulus, k (MPa/m)	Strain Factor, E_{50}, (%)
Medium Clay with Free Water	228 – 226.5	18	45	10	1
Stiff Clay with Free Water	226.5 – 223.2	19.0	80	15	0.7
Stiff Clay with Free Water	223.2 – 219.5	18.0	15	5	2
Stiff Clay with Free Water	Below 219.5	18.0	45	10	1

5.7 Pile Group Effects

5.7.1 Group Effects of Axial Capacity

The minimum pile spacing should be at least three times the pile diameter or flange width (measured centre to centre). Group load efficiency should be taken into account for piles spaced closer than three times their diameter, measured centre to centre; however, efficiency factors are largely dependent on the size of the group of piles and its configuration, and, for some configurations, efficiency factors may be required for piles spaced further apart than three times the pile diameter (e.g. 16 piles in a group of 4 by 4, as will be shown in **Table 5-3**). Spacing that is closer than 1.5 pile diameters is not recommended due to decreasing load capacity and practical installation considerations.

The efficiency of the pile group is given by the following equation:

$$Qg = \eta Qp$$

Where:

Qg = capacity of the group

η = group efficiency

Qp = capacity of individual pile

The efficiency factors can be computed using the following equation:

$$\eta = \frac{2s(m+n-2)+4D}{\pi mnD} \leq 1$$

Where:

s = spacing between piles, measured centre to centre

m = number of columns of piles

n = number of rows of piles

D = diameter of individual pile

π = 3.14

The efficiency factors have been calculated for several assumed square configurations of piles. These are presented in **Table 5.3**.

Table 5.3: Efficiency Factors for Assumed Pile Groups – Axial Pile Capacity

Pile Spacing – Measured Centre to Centre (Multiples of Flange Width or Pile Diameter)	Pile Group Size		
	2x2	3x3	4x4
4	1.00	1.00	1.00
3	1.00	1.00	0.80
2.5	1.00	0.85	0.65
2	0.95	0.70	0.55
1.5	0.80	0.55	0.40

5.7.2 Group Effects on Pile Settlement

The settlement of a single CIP will occur as the shaft friction and end bearing are mobilized under loading. The total settlement of a pile is determined by a combination of this movement to mobilize resistance of the soil as well as elastic shortening of the pile. The elastic compression of the pile due to structural loads should be evaluated by the structural engineer based on the elastic properties of the pile and pile cross section.

The interaction of pile groups must be considered from a settlement perspective as the supporting soil for groups is much larger than that for a single pile. In addition to the axial load capacity reductions due to pile spacing (**Section 5.7.1**), the capacity of pile groups of increasing size is strongly influenced by increased magnitude of settlement. For design consideration, the effect of group action can be expressed as a settlement ratio, Rs. The settlement ratio is defined by the following equation and is considered suitable to estimate pile group settlement.

$$R_s = \frac{\text{Average group settlement considering average load per pile}}{\text{Settlement of a single pile under the same average load}}$$

Group influences on settlement are considered to diminish at a pile spacing of at least seven pile diameters (i.e. Rs = 1). For typical centre to centre pile spacing within a group of three pile diameters (installed to the recommendations in **Sections 5.2.4**) with the pile tops connected within a rigid frame or pile cap, the settlement ratios in **Table 5.4** are recommended. The settlement ratio within groups with pile spacing between three and seven pile diameters may be linearly interpolated.

Table 5.4: Recommended Settlement Ratios (Rs)

Pile Group Size	Rs
2x2	2.3
3x3	3.9
4x4	5.9
5x5	8.1

Based on the exact number of piles in a group, Rs values for other number of piles may be determined during the detailed design stage. For groups containing more than 16 piles, it has been found that Rs increases linearly with the square root of number, n, of piles in group (Poulos and Davis, 1980). Thus, for a given value of pile spacing, Rs may be extrapolated from the values for a 16 pile group and a 25 pile group using the equation shown below.

$$R_s = (R_{25} - R_{16})\left(\sqrt{n} - 5\right) + R_{25}$$

5.7.3 Group Effects on Lateral Capacity

The lateral capacity of individual piles in a group is primarily affected by the spacing of the piles, measured centre-to-centre along an alignment parallel to the lateral load applied (provided that the pile spacing perpendicular to the applied load is at least three pile diameters). Depending on the number of rows, group effects diminish at a pile spacing of six pile diameters or greater. Similar to axial loading, reduction factors for lateral loading should be applied. The lateral load reduction factors (pile spacing parallel to applied load) are provided in **Table 5.5**.

Table 5.5: Recommended Lateral Load Reduction Factors for Pile Groups

Pile Spacing - Measured Centre to Centre (Multiples of Flange Width or Pile Diameter)	Lateral Load Reduction Factors		
	1st Row (Lead Row)	2nd Row	3rd and Subsequent Rows
7	1.00	1.00	0.92
6	0.97	0.93	0.83
5	0.92	0.84	0.72
4	0.86	0.72	0.58
3	0.79	0.57	0.41

The above **Sections 5.7.1, 5.7.2 and 5.7.3** should be reviewed and revised based on the final pile configuration.

6. Quality Assurance (QA) and Quality Control (QC)

6.1 General

Quality Assurance / Quality Control (QA/QC) is described in various sections of this report. The following sections provide a summary of the QA/QC elements required during construction. Diligently completing geotechnical QA/QC will reduce the potential for geotechnical related issues (settlement, bearing capacity failures, slope stability issues, etc.). All geotechnical QA/QC should be completed by qualified geotechnical personnel. The reader should refer to the appropriate report section for detailed QA/QC recommendations.

Traditionally, QC is completed on behalf of the contractor in order to control the quality of their work prior to inspection, and QA is completed on behalf of the client in order to assure them of the quality of the work done. The intent with this QA/QC section of the report is not to differentiate which testing belongs under QC and which belongs under QA, but to give an overall minimum requirement for the project that will help produce a quality product.

6.2 Geotechnical Inspections

The exposed/prepared subgrades (after excavation) for foundations should be inspected by qualified geotechnical personnel prior to any fill placement or foundation construction. The intent is to confirm that subgrade is prepared in accordance with the geotechnical recommendations. Foundations, pavements, embankments, and any other structures may experience distress during their life span if subgrade is not prepared according to recommendations and project specifications. Structure specific recommendations for subgrade preparation are provided within this report.

6.3 Proof-Rolling

Proof-rolling is a method of detecting soft areas in exposed/prepared subgrade for pavements, floors, or foundations prior to fill placement or construction of structures. Proof-rolling is also used for detecting non-uniformity of compacted embankments. The intent is to detect soft areas or areas of low strength not otherwise revealed by test holes, field density testing, or visual inspections. Proof-rolling should be observed by qualified geotechnical personnel.

Proof-rolling is generally accomplished by the use of a heavy (15 to 16 tonne) rubber-tired roller having 4 wheels abreast on independent axles with high contact wheel pressures (inflation pressures ranging from 550 kPa up to 1030 kPa).

A heavily loaded tandem axle gravel truck may be used in lieu of the equipment described above. The truck should be loaded to approximately 10 tonnes per axle with a minimum tire pressure of 550 kPa.

The recommended ground speed for proof-rolling is 4 km/hr.

The recommended procedure is two complete coverages with the proof-rolling equipment in one direction and a second series of two coverages made at right angles to the first series. Less rigorous procedures may be acceptable under certain conditions subject to the approval of a geotechnical engineer.

The surface of the grade under the action of the proof-roller should be observed, noting visible deflection and rebound of the surface, formation of a crack pattern in the compacted surface, or shear failure in the surface of granular soils as ridging between wheel tracks.

After proof-rolling and repair of all failed areas, construction should commence immediately to prevent the prepared subgrade from weather (sun, precipitation, etc.) or construction traffic which may damage the prepared subgrade. A re-inspection or proof-rolling may be required if construction is delayed or if the prepared subgrade is damaged.

The recommendations provided in this report may not be valid if proof-rolling is not conducted. Failing to conduct proof-rolling may result in failure to detect the soft/weak areas which may cause future settlements, subgrade and slope failures, and structure distress.

6.4 Field Compaction Testing

The fill placement and compaction of the material should be monitored by qualified geotechnical personnel using a nuclear densometer. Field compaction testing with a densometer provides percent compaction and moisture content of the fill being placed at the test locations. It does not provide compaction and moisture content of the area between test points; therefore, it is recommended that nuclear densometer testing be used in conjunction with proof-rolling. The intent of combining proof-rolling with field compaction testing is twofold: (1) to reduce subjectivity that may be associated with visual observations that take place during proof-rolling, and (2) to identify any weak areas not revealed by field compaction testing. The nuclear densometer must be operated and in the possession of qualified geotechnical personnel with the appropriate training. Like proof-rolling, if density testing is not undertaken, assumptions made during the design phase cannot be validated resulting in settlements and/or foundation failure.

6.5 CIP Concrete Piles

Material used for CIP concrete piles should be inspected by a qualified individual and stored on site in accordance with the manufacturer's specifications. Strength testing should be completed on concrete cylinders to verify the material's compliance with specifications. The air content and slump testing should also be conducted for concrete to confirm that air and slump are within specified limits. The steel reinforcing cage should be assembled, inspected, and handled by qualified personnel. If the pile is end bearing, inspection of the pile base should be completed by qualified geotechnical personnel prior to placement of reinforcement and pouring of concrete in pile hole. CIP concrete piles should be constructed to sufficient depth to adequately resist uplift forces caused by frost jacking.

The installation of CIP concrete piles should be monitored by qualified geotechnical personnel. Detailed recommendations for installation, testing, and monitoring of CIP concrete piles are in **Section 5.2**.

6.6 Pile Load Testing

Low strain impact integrity testing (also known as pile integrity testing or PIT) should be completed on at least half of CIP concrete piles to verify the integrity of the piles after construction. The testing should be completed in accordance with ASTM D5882.

6.7 Soil

Material testing should be performed throughout construction on native and imported soils. When new materials are used in construction, qualified geotechnical personnel should complete geotechnical testing to verify that the material complies with project specifications. These tests include but are not limited to:

- Atterberg limits
- Moisture content

- Standard Proctor Density
- Hydraulic conductivity
- Chemical analysis (sulphates, pH, resistivity, etc.)
- Grain size analysis

The material used for construction may not fall within project specifications if not confirmed by testing in accordance with project specifications.

6.8 Settlement

Regular and frequent survey of monitoring points in settlement sensitive areas should be completed in order to identify areas that may require attention before any significant settlement occurs.

7. Review of Design and Construction

Design drawings should be submitted to the AECOM geotechnical team for review before they are finalized.

All recommendations given in this report are based on the assumption that an adequate level of monitoring will be provided during construction, and that all construction will be carried out by suitably qualified contractors, experienced in earthworks and foundation construction in Manitoba. Adequate levels of monitoring are considered to be:

- For deep foundations (piles), full time inspection and design review during construction
- For earthworks, full time monitoring and compaction quality control

Qualified geotechnical personnel independent of the contractor should carry out all such quality assurance monitoring. The main purpose of monitoring is to check that the recommendations provided in this report, which are based on the findings at discrete test hole locations, are relevant to other areas of the site.

8. References

[1] *Sludge Digestion Expansion Geotechnical Report (June 1984) by The UMA Group.*

[2] *Geotechnical Investigation and Foundation Engineering Report for Hauled Liquid Waste Facility NEWPCC (December 2009) by The National Testing Laboratories Limited.*

[3] *NEWPCC Power Supply Upgrade Geotechnical Baseline Report (October 2015) by KGS Group.*

[4] *NEWPCC Upgrade Geotechnical Evaluation and Foundation Engineering Report (June 2017) by Stantec.*

[5] *North End STP Headworks Facilities Project – Supplemental Field Investigation (July 2020) by Stantec.*

CFEM (2006). Canadian Foundation Engineering Manual, 4th Edition.

Das, B.M. (2008). Fundamentals of Geotechnical Engineering, 3rd Edition.

Dow Chemical Company. Tech Solutions 602.0 Styrofoam Brand Highload Insulation for Buried Utility Lines.

Fellenius, B.H. (1984). Negative Skin Friction and Settlement of Piles. Second International Seminar, Pile Foundations, Nanyang Technological Institute, Singapore, 18 p.

NAVFAC Design Manual 7.1: Soil Mechanics (1986).

NAVFAC Design Manual 7.2: Foundations and Earth Structures (1986).

Rollins, K., Olsen, K., Jensen, D., Garrett, B., Olsen, R., and Egbert, J. (2006). "Pile Spacing Effects on Lateral Pile Group Behavior: Analysis." J. Geotech. Geoenviron. Eng., 132(10), 1272–1283.

Skirrow, R., Wang, R. (2008). Cost-Benefit Analysis for Selection of Pile Tests. Technical Standards Branch, Alberta Transportation.

CSA A23.1-19. Concrete materials and methods of concrete construction/Test methods and standard practices for concrete.

Appendix A

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Canadian Council of Independent Laboratories

MOISTURE CONTENT OF SOIL (ASTM D2216)

CLIENT:	**AECOM**	TEST NO:	1	PROJECT NO:	112-2117
PROJECT:	**NEWPCC PC Upgrade**	DATE SAMPLED:	**Nov. 9, 2021**	SAMPLED BY:	**Client**
PROJECT CONTACT:	**E. Manimbao**	DATE TESTED:	**Nov.. 24, 2021**	TESTED BY:	**A.Manalundong**

Test Hole No. **TH21-01**	**S1**	**S2**	**T3**	**S4**	**T5**
Depth	**5 ft.**	**10 ft.**	**15 ft.**	**20 ft.**	**25 ft.**
Tare No.					
Wt Wet Sample + Tare	154.8	153.7		152.9	
Wt Dry Sample + Tare	126.3	122.8		102.5	
Wt Water	28.5	30.9		50.4	
Wt Tare	4.4	4.4		4.4	
Wt Dry Sample	121.9	118.4		98.1	
Moisture Content (%)	**23.4**	**26.1**		**51.4**	
Test Hole No.	**S6**	**S7**	**S8**	**S9**	**S10**
Depth	**30 ft.**	**35 ft.**	**40 ft.**	**50 ft.**	**60 ft.**
Tare No.					
Wt Wet Sample + Tare	154.3	152.9	153	152.7	151.7
Wt Dry Sample + Tare	105.8	102.5	104.5	93.1	139
Wt Water	48.5	50.4	48.5	59.6	12.7
Wt Tare	4.3	4.3	4.6	4.2	4.4
Wt Dry Sample	101.5	98.2	99.9	88.9	134.6
Moisture Content (%)	**47.8**	**51.3**	**48.5**	**67.0**	**9.4**
Test Hole No.	**S11**				
Depth	**66 ft.**				
Tare No.					
Wt Wet Sample + Tare	152.4				
Wt Dry Sample + Tare	136.3				
Wt Water	16.1				
Wt Tare	4.2				
Wt Dry Sample	132.1				
Moisture Content (%)	**12.2**				
Test Hole No.					
Depth					
Tare No.					
Wt Wet Sample + Tare					
Wt Dry Sample + Tare					
Wt Water					
Wt Tare					
Wt Dry Sample					

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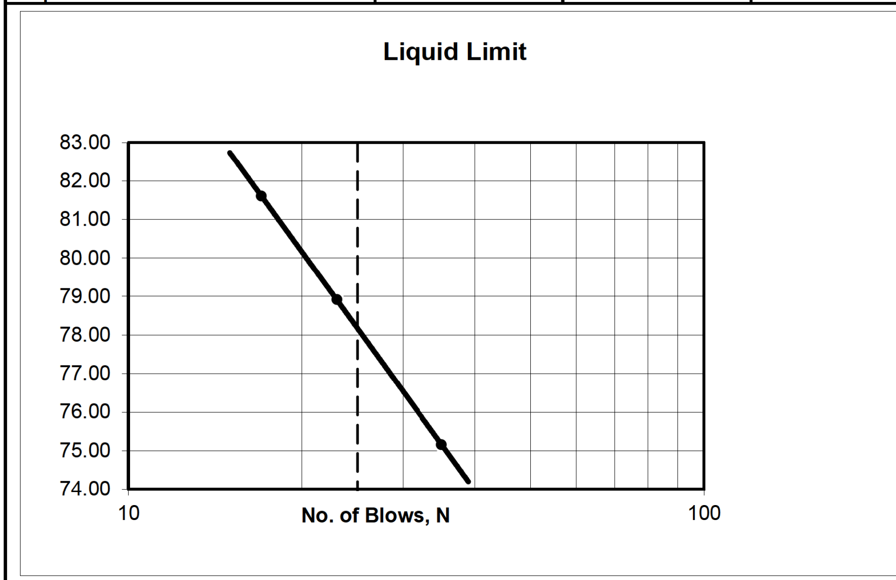
Canadian Council of Independent Laboratories

Atterberg Limits (ASTM D4318)

Client:	AECOM Canada Ltd. 99 Commerce Drive Winnipeg MB R3P 0Y7	PROJECT No.: PI Test No.: LAB No.:	112-2117 1 HM 570-1B
Attention.:	Enrico Manimbao	Date Received:	Nov. 24, 2021
Project:	NEWPCC PC Upgrade	Date Tested / By:	2021-11-29/G. Manalo

Liquid Limit Determination

Dish No.:	1	2	3		Liquid Limit 25 Blows
Wet Soil + Dish:	12.75	12.52	11.50		
Dry Soil + Dish:	9.24	9.00	8.26		
Moisture:	3.51	3.52	3.24		
Dish:	4.57	4.54	4.29		
Dry Soil:	4.67	4.46	3.97		
% Moisture:	75.16	78.92	81.61		
No. of Blows:	35	23	17		
Liquid Limit:					78



Material Identification:

TH21-01-S4

Depth:	**20 FT.**
Liquid Limit, %:	**78**
Plastic Limit, %:	**28**
Plasticity Index: (LL-PL)	**50**

Plastic Limit Determination

Dish No.:	1	2	3		
Wet Soil + Dish:	9.68	9.89	9.85		
Dry Soil + Dish:	8.53	8.68	8.60		
Moisture:	1.15	1.21	1.25		
Dish:	4.42	4.4	4.24		
Dry Soil:	4.11	4.28	4.36		
% Moisture:	27.98	28.27	28.67		
Average:					**28**

Test Method : ASTM: D4318, D2216

P. Bevel

Reviewed by: Paul Bevel

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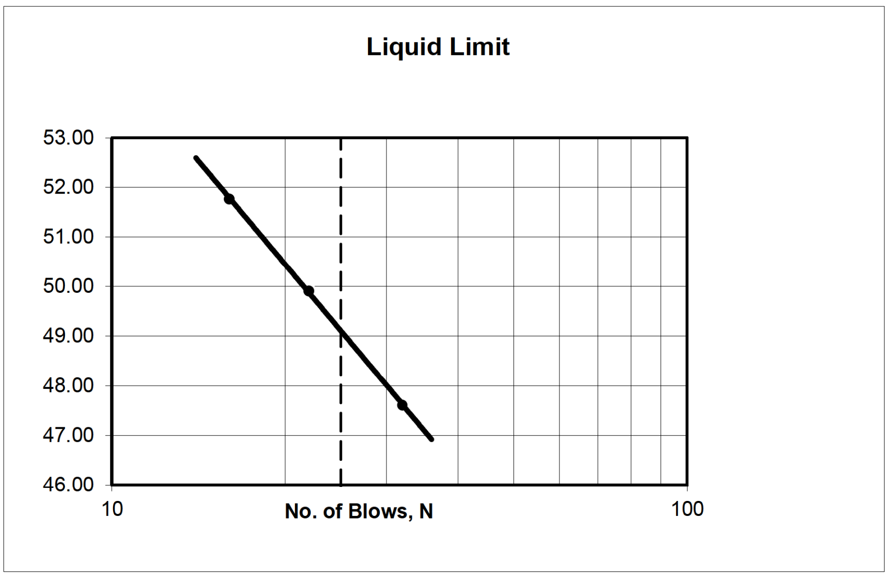
Canadian Council of Independent Laboratories

Atterberg Limits (ASTM D4318)

Client:	AECOM Canada Ltd.	PROJECT No.:	112-2117
	99 Commerce Drive	PI Test No.:	2
	Winnipeg MB R3P 0Y7	LAB No.:	HM 570-2B
Attention.:	Enrico Manimbao	Date Received:	Nov. 24, 2021
Project:	NEWPCC PC Upgrade	Date Tested / By:	2021-11-29/G. Manalo

Liquid Limit Determination

Dish No.:	1	2	3		Liquid Limit 25 Blows
Wet Soil + Dish:	10.75	12.81	12.99		
Dry Soil + Dish:	8.66	10.05	10.06		
Moisture:	2.09	2.76	2.93		
Dish:	4.27	4.52	4.4		
Dry Soil:	4.39	5.53	5.66		
% Moisture:	47.61	49.91	51.77		
No. of Blows:	32	22	16		
Liquid Limit:					49



Material Identification:

TH21-01-S8

Depth: **40 FT.**

Liquid Limit, %: **49**
Plastic Limit, %: **21**
Plasticity Index: **28**
(LL-PL)

Plastic Limit Determination

Dish No.:	1	2	3		
Wet Soil + Dish:	11.78	11.2	11.76		
Dry Soil + Dish:	10.48	9.98	10.44		
Moisture:	1.3	1.22	1.32		
Dish:	4.31	4.29	4.4		
Dry Soil:	6.17	5.69	6.04		
% Moisture:	21.07	21.44	21.85		
Average:					**21**

Test Method : ASTM: D4318, D2216

P. Bevel

Reviewed by: Paul Bevel

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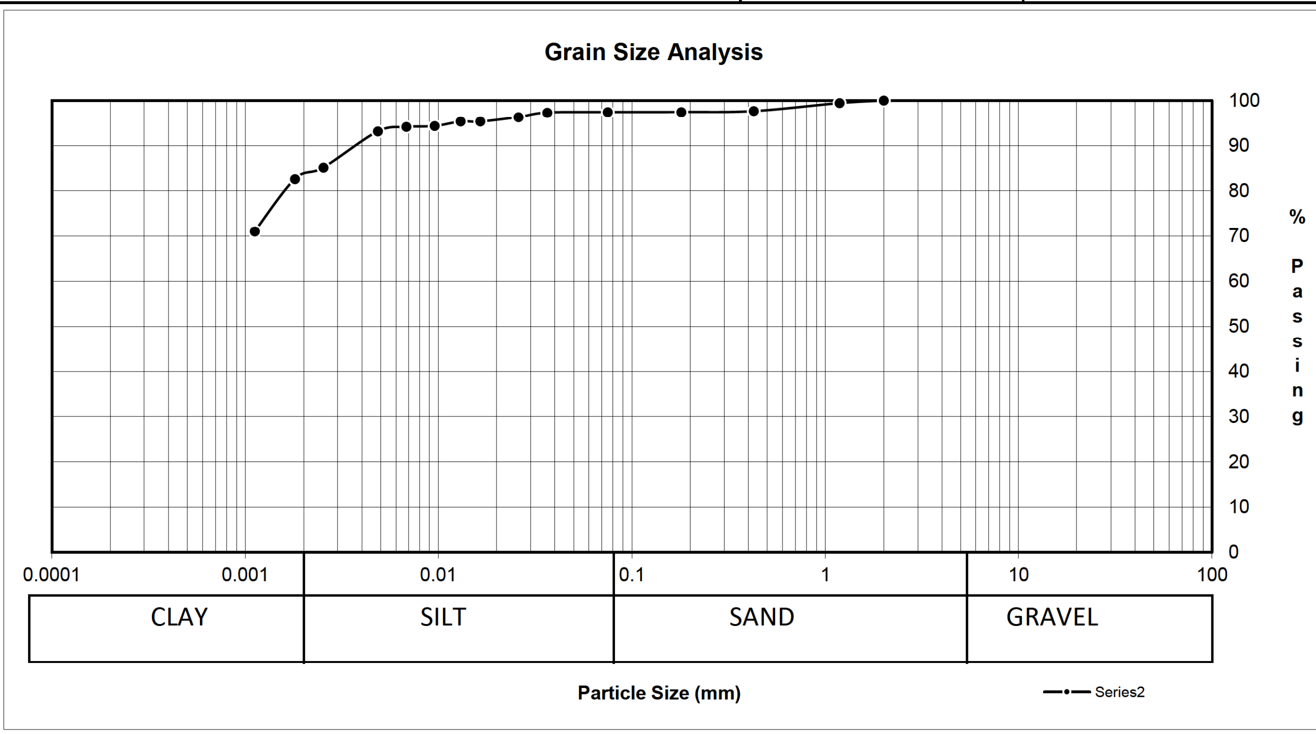
PARTICLE SIZE ANALYSIS OF SOILS TEST REPORT

CLIENT:	AECOM Canada Ltd.	PROJECT No.:	112-2117
	99 Commerce Drive	PSA Test No.:	1
	Winnipeg MB R3P 0Y7	LAB No.:	HM 570-1A
ATTENTION:	Enrico Manimbao		
PROJECT:	NEWPCC PC Upgrade		

				Sieve Analysis		Hydrometer Analysis	
Date Sampled:	09-Nov-21	Date Received:	24-Nov-21				
Sampled By:	Client	Date Tested:	29-Nov-21	Sieve (mm)	% Passing	Diameter	% Finer
				50.00	100.0		
				37.50	100.0		
				25.00	100.0		
				19.00	100.0		
				16.00	100.0		
Material Identification				12.50	100.0	0.0365	97.3
B.H./T.H. No.		**TH21-01-S4**		9.50	100.0	0.0259	96.3
Depth		**20 FT.**		4.75	100.0	0.0165	95.3
Sample Source				2.00	100.0	0.0130	95.3
Specific Gravity of Material:		2.65		1.18	99.4	0.0096	94.3
				0.425	97.6	0.0068	94.1
				0.180	97.4	0.0048	93.1
				0.075	97.4	0.0011	71.0



% Composition			
	Gravel	D10	
		D30	0.00614
2.58	Sand	D60	0.01550
23.27	Silt	Cu	
74.15	Clay	Cc	

Remarks: Test Method: ASTM D7928, D2216, D4318

Technician: G. Manalo

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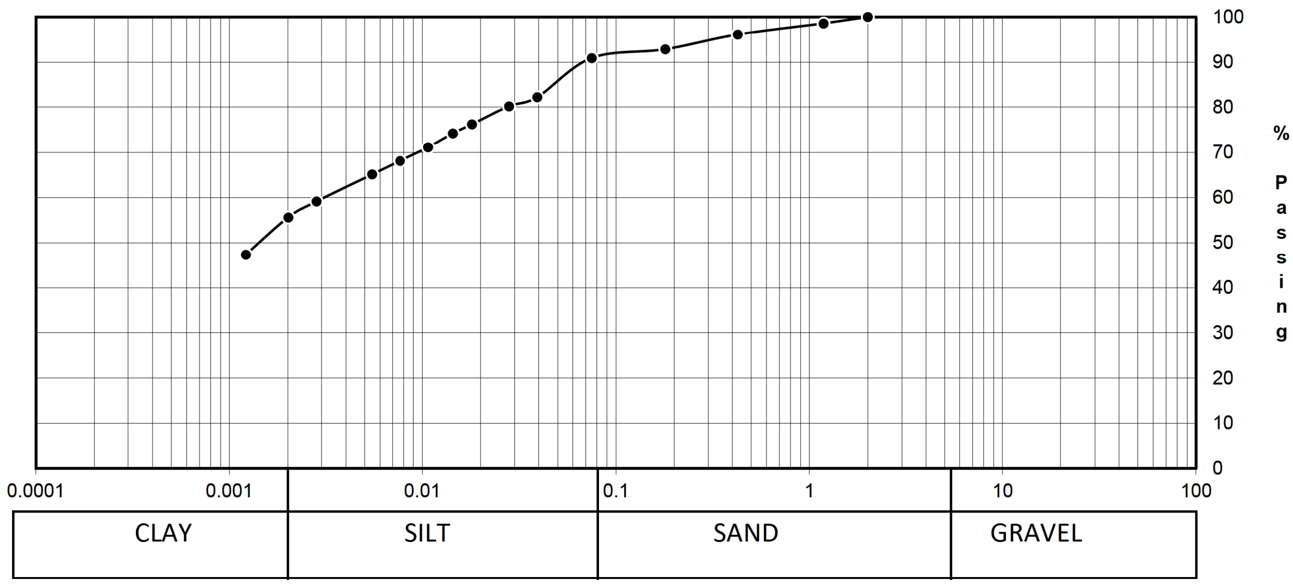
CCIL Canadian Council of Independent Laboratories

PARTICLE SIZE ANALYSIS OF SOILS TEST REPORT

CLIENT:	AECOM Canada Ltd.	PROJECT No.:	112-2117
	99 Commerce Drive	PSA Test No.:	2
	Winnipeg MB R3P 0Y7	LAB No.:	HM 570-2A
ATTENTION:	Enrico Manimbao		
PROJECT:	NEWPCC PC Upgrade		

				Sieve Analysis		Hydrometer Analysis	
Date Sampled:	09-Nov-21	Date Received:	24-Nov-21				
Sampled By:	Client	Date Tested:	29-Nov-21	Sieve (mm)	% Passing	Diameter	% Finer
				50.00	100.0		
				37.50	100.0		
				25.00	100.0		
				19.00	100.0		
				16.00	100.0		
Material Identification				12.50	100.0	0.0392	82.2
B.H./T.H. No.		**TH21-01-S8**		9.50	100.0	0.0280	80.2
Depth		**40 FT.**		4.75	100.0	0.0180	76.2
Sample Source				2.00	100.0	0.0144	74.2
Specific Gravity of Material:		2.65		1.18	98.6	0.0107	71.1
				0.425	96.1	0.0076	68.1
				0.180	92.8	0.0055	65.1
				0.075	90.9	0.0012	47.4



% Composition			
	Gravel	D10	
9.12	Sand	D30	
35.39	Silt	D60	0.00282
55.49	Clay	Cu	
		Cc	

Remarks: Test Method: ASTM D7928, D2216, D4318

Technician: G. Manalo

P. Bevel

Reviewed by: Paul Bevel

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UNCONFINED COMPRESSIVE STRENGTH TEST REPORT

CLIENT:	AECOM 99 Commerce Drive Winnipeg MB R3P 0Y7	PROJECT NO.:	112-2017
		Qu Test No.:	1
		Lab No.:	HM 570
ATTENTION:	Enrico Manimbao		
PROJECT:	NEWPCC PC Upgrade		

Date Sampled: 09-Nov-21 Date Received: 24-Nov-21
Sampled By: Client Date Tested: 06-Dec-21 Sample ID: TH 21-01 T3 (15')

Test Result: **Unconfined Compressive Strength 83.1 kPa**

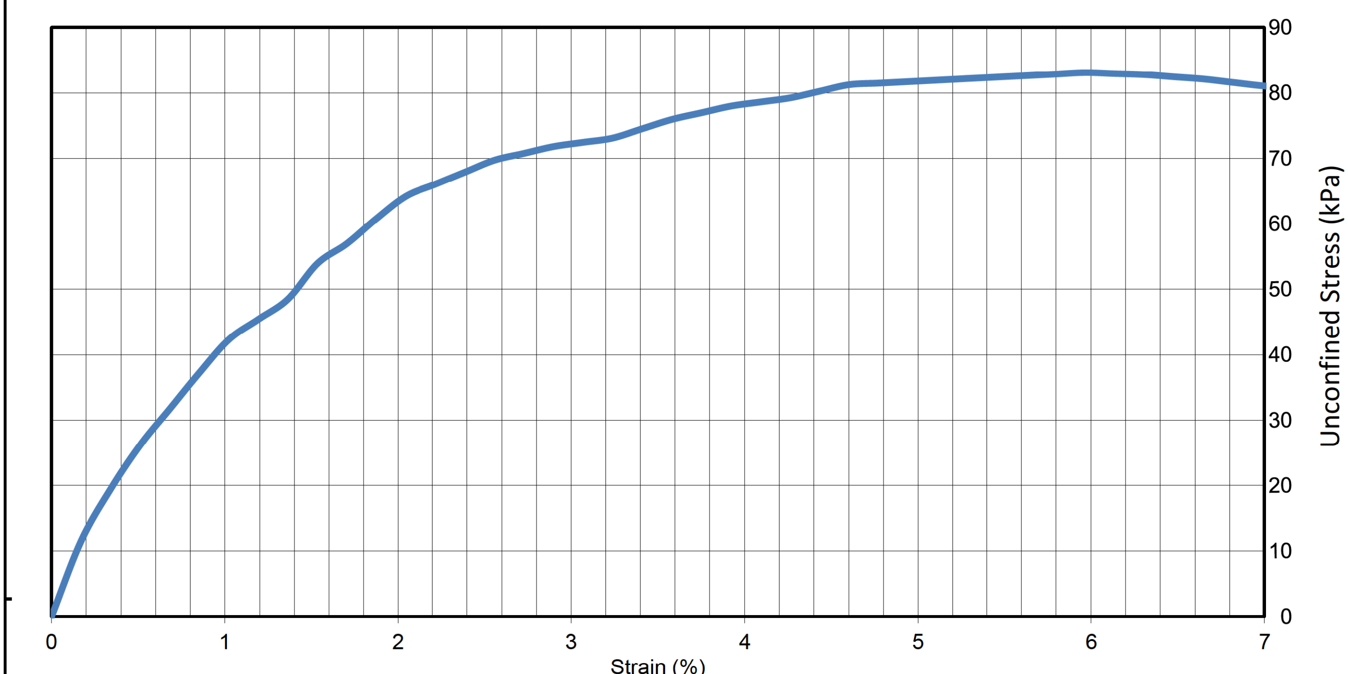
Test Sample Data

Sample Mass (g)	Average Height (m)	Average Diameter (m)	Moisture Content %	Wet Density (kg/m3)	Dry Density (kg/m3)	Strain rate (%/min)
1115.5	0.1490	0.0730	50.3	1788	1190	1.0

Test Sample Visual Description

CLAY, silty, dark brown, soft to firm, moist

Unconfined Stress (kPa) vs Strain (%)



Remarks: Test Method: ASTM D2166
Technician: ES

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Reviewed by: Paul Bevel

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UNCONFINED COMPRESSIVE STRENGTH TEST REPORT

CLIENT: AECOM
99 Commerce Drive
Winnipeg MB R3P 0Y7
ATTENTION: Enrico Manimbao
PROJECT: NEWPCC PC Upgrade

PROJECT NO.: 112-2017
Qu Test No.: 2
Lab No.: HM 570

Date Sampled: 09-Nov-21
Sampled By: Client
Date Received: 24-Nov-21
Date Tested: 06-Dec-21
Sample ID: TH 21-01 T5 (25')

Test Result: Unconfined Compressive Strength 112.6 kPa

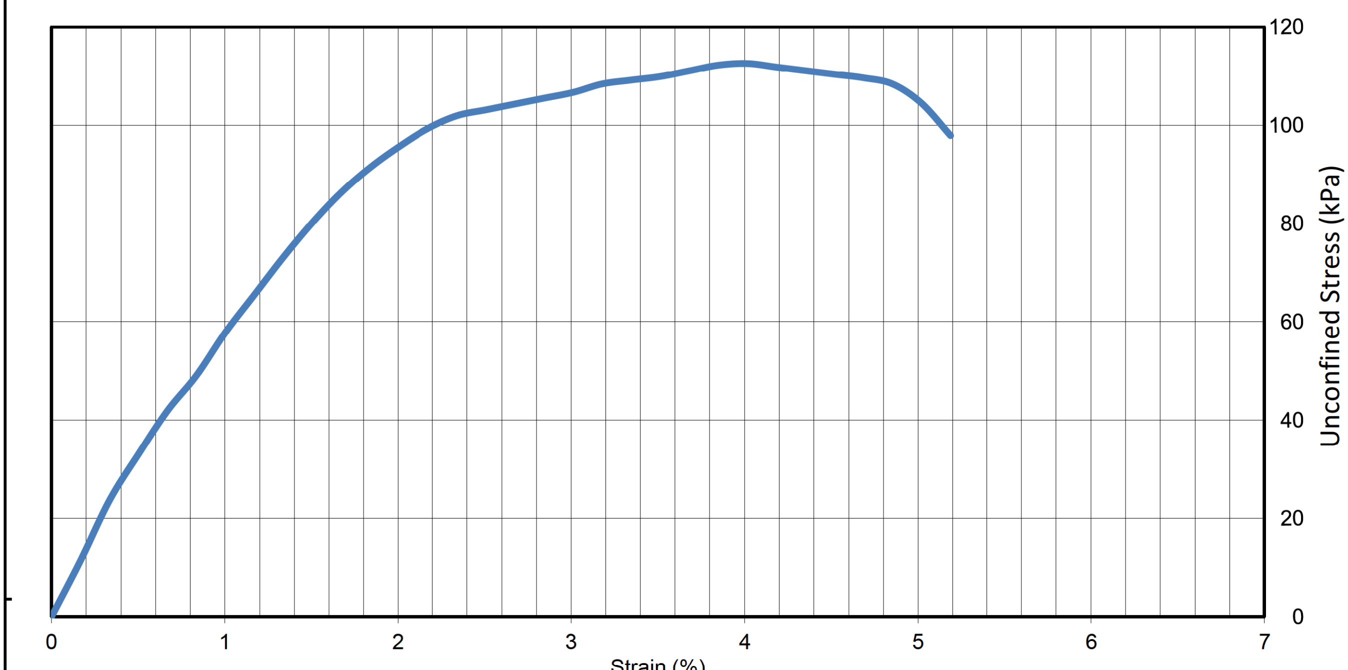
Test Sample Data

Sample Mass (g)	Average Height (m)	Average Diameter (m)	Moisture Content %	Wet Density (kg/m3)	Dry Density (kg/m3)	Strain rate (%/min)
1160.0	0.1518	0.0730	45.4	1825	1255	1.0

Test Sample Visual Description

CLAY, silty, dark brown, firm, moist

Unconfined Stress (kPa) vs Strain (%)



Remarks: Test Method: ASTM D2166
Technician: ES

P. Bevel

Reviewed by: Paul Bevel

Appendix B

PROJECT: NEWPCC Primary Clarification Upgrade	CLIENT: City of Winnipeg	TESTHOLE NO.: **TH21-01**
LOCATION: 14U 635609 m E, 5535031 m N	COORDINATES:	PROJECT NO.: 60661262
CONTRACTOR: Maple Leaf Drilling Ltd	METHOD: WP 030 Track Rig - 125 mm SSA	ELEVATION (m): 231

SAMPLE TYPE: GRAB | SHELBY TUBE | SPLIT SPOON | BULK | NO RECOVERY | CORE

DEPTH (m)	USC	SOIL DESCRIPTION
0	OR	TOPSOIL (150 mm thick)
	FILL	GRANULAR FILL, some organics
1	CH	CLAY - trace silt, trace sand, trace gravel - brown, firm, moist - high plasticity - silty, no sand, no gravel, pockets of light grey below 2.4 m
3	ML	SILT - trace sand, trace clay - light brown, soft to firm, moist - intermediate plasticity
4	CH	CLAY - some silt, trace sand - brown, firm, moist - intermediate to high plasticity - silty, very stiff, high plasticity below 6.1 m - (S4): Gravel 0.0%, Sand 2.6%, Silt 23.3%, Clay 74.1% - grey, firm, some silt pockets below 7.6 m - soft below 9.1 m - intermediate plasticity below 12.2 m - (S8): Gravel 0.0%, Sand 9.1%, Silt 35.4%, Clay 55.5%
18	TILL	GLACIAL TILL - brown, soft, moist
20		End of Testhole at 20.27 mBGS as per design requirement Notes: 1. Seepage was observed at 5.5 m BSG while drilling. 2. Sloughing was not observed during drilling. 3. Test hole was backfilled with auger cuttings and sealed with bentonite chips.

QU (kPa): 100, 200, 300, 400 — SPT (Standard Pen Test) (Blows/300mm): 25, 50, 75 — PLASTIC, M.C., LIQUID: 25, 50, 75

SAMPLE TYPE	SAMPLE #	SPT (N)	M.C.	COMMENTS
Split spoon	S1	9	23.4	SPT Blows: [2/4/5] Recovery: 30%
Split spoon	S2	9	28.1	SPT Blows: [10/6/3] Recovery: 50%
Shelby tube	T3			Q_u = 83.1 kPa
Split spoon	S4	20	51.4	SPT Blows: [6/8/12] Recovery: 100%
Shelby tube	T5			Q_u = 112.6 kPa
Split spoon	S6	4	47.8	SPT Blows: [0/1/3] Recovery: 100%
Split spoon	S7	2	51.3	SPT Blows: [0/1/1] Recovery: 100%
Split spoon	S8	8	48.5	SPT Blows: [3/4/4] Recovery: 100%
Split spoon	S9	8	67	SPT Blows: [0/0/8] Recovery: 60%
Split spoon	S10	10	9.4	SPT Blows: [3/6/4] Recovery: 50%
Split spoon	S11		12.2	SPT Blows: [>50/150mm] Recovery: 30%

AECOM

LOGGED BY: Enrico Manimbao	COMPLETION DEPTH: 20.27 m
REVIEWED BY: Ryan Harras	COMPLETION DATE: 11-9-2021
PROJECT MANAGER: Usman Raja	Page 1 of 1