Appendix K

Geotechnical Report

NEWPCC Primary Scum Building

Geotechnical Report

City of Winnipeg

60661262

August 2024

Delivering a better world

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, MB R3P 0Y7
Canada

T: 204.477.5381
F: 431.800.1210
www.aecom.com

Isaac Orah, P.Eng.
City of Winnipeg, Water and Waste Department
110-1199 Pacific Avenue
Winnipeg, MB R3E 3S8

August 15, 2024

Project #
60661262

Subject: NEWPCC Primary Scum Building – Geotechnical Report

Dear Mr. Orah:

AECOM Canada Ltd. is pleased to submit our Geotechnical Report for the above referenced project.

Should you have any queries, please contact German Leal directly at (204)-928-8479.

We appreciate the opportunity to provide the City of Winnipeg Water and Waste Department with our services and look forward to working together on this project and future projects.

Sincerely,
AECOM Canada Ltd.

German Leal, M.Eng., P.Eng.
Discipline Lead, Geotechnical
AECOM Canada Ltd.
T: 204-928-8479
M: 204-928-8479
E: german.leal@aecom.com

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AECOM: 2015-04-13

Quality Information

Prepared by

Colton Wooster, E.I.T., B.Eng.
Geotechnical

Verified by

German Leal, M.Eng., P.Eng.
Discipline Lead

Approved by

Leah Daniel, M.Sc., P.Eng.
Water/Wastewater Process Engineer

Revision History

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Distribution List

# Hard Copies	PDF Required	Association / Company Name
	✓	City of Winnipeg
	✓	AECOM Canada Ltd.

Prepared for:

City of Winnipeg
Water and Waste Department
110-1199 Pacific Avenue
Winnipeg, MB R3E 3S8

Prepared by:

Colton Wooster
Geotechnical EIT
M: 204-583-8797
E: colton.wooster@aecom.com

German Leal
Discipline Lead, Geotechnical
M: 204-928-8479
E: german.leal@aecom.com

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, MB R3P 0Y7
Canada

T: 204.477.5381
F: 431.800.1210
www.aecom.com

Executive Summary

AECOM Canada Ltd. ("AECOM") was retained to undertake a geotechnical investigation to evaluate the existing soil conditions and provide foundation recommendations for proposed construction of the new Primary Scum Building. The project site is located at 2230 Main Street at the North End Sewage Treatment Plant in Winnipeg, Manitoba.

The project involves the design and construction of the Primary Scum Building. The building footprint is approximately 286 m^2. The Primary Scum Building will be comprised of three floors. A lower floor, main floor, and upper floor. The lower floor finished floor elevation (FFE) is 227.05 meters above sea level (ASL), the main floor will have an elevation of 231.05 m ASL, and the second floor will have a FFE of 235.150 m ASL. The roof of the building will be accessible by door, with the top of the penthouse roof elevation at 245.35 m ASL.

Two (2) testholes were drilled on the project site on July 10 and 11, 2024. These testholes served various purposes:

- Testhole TH24-01 was drilled to auger refusal and a standpipe piezometer was installed to record the groundwater elevations at the depth of the lower FFE.
- Testhole TH24-02 was drilled to auger refusal, then coring took place until 3 m of bedrock was retrieved. This was to estimate pile embedment depths of the proposed piles and to determine rock quality within the project site.

Two foundation systems (driven precast concrete piles and driven steel H piles) were considered during the preparation of this report. The geotechnical report provides recommendations for the lateral earth pressure for the design of the foundation walls.

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1. Introduction

The proposed Primary Scum Building will consist of 3 floors, which includes a basement, main floor (loading bay), second floor, and a roof deck. The building is approximately 13 m wide by 22 m in length.

The building will be supported by grade beams, pile caps, and a pile foundation system. The current design included caissons, which requires redesign to the foundation elements. The basement floor will be a structural floor slab.

AECOM Canada Ltd. ("AECOM") was retained to undertake a geotechnical investigation to evaluate the existing soil conditions and provide foundation recommendations for proposed construction of the new Primary Scum Building. The project site is located at 2230 Main Street at the North End Sewage Treatment Plant in Winnipeg, Manitoba.

AECOM previously conducted a geotechnical investigation that consisted of one testhole within the footprint of the proposed Primary Scum Building. This testhole was drilled on November 9, 2021. Since then, the footprint of the building has moved approximately 10 m west of the proposed location in 2021, leaving the one testhole at the northeast corner of the proposed building footprint. AECOM's project team determined that an additional geotechnical investigation was required below this new proposed building footprint to provide a better understanding of the soil stratigraphy. Additionally, in the past design, a caisson foundation system was proposed. Caissons are cost prohibitive; to refine the design and budget of the project, AECOM's design team will be investigating driven precast concrete piles and driven steel H piles as the updated foundation system in the design. Two (2) testholes were drilled on the project site on July 10 and 11, 2024.

The work that was performed as part of this geotechnical study included the following:

- Private utility locator to locate existing utilities.
- Hydro-vacuum to expose unknown and abandoned utilities that were not detected by the private locator.
- A geotechnical drilling and soil sampling program at the proposed site to identify the existing soil and groundwater conditions.
- Laboratory testing program to determine the engineering properties relevant to the foundation design. The testing program included moisture contents on all collected samples, Atterberg limits, particle size analysis, and unconfined compression tests of intact soil specimen and of rock core samples.
- Evaluate the geotechnical capacity of driven precast concrete piles and driven steel H piles for the proposed structure.
- The preparation of this geotechnical report outlines the existing condition, frost implications and explores foundation design recommendations.

Use of this report is subject to the Statement of Qualifications and Limitations provided at the beginning of this report.

2. Project Site and Proposed Construction

The project site is located at 2230 Main Street in Winnipeg, MB at the North End Sewage Treatment Plant (NEWPCC). The project site terrain is comprised of grass and sparse trees. The proposed Primary Scum Building footprint has an existing concrete road running through it. To the north of the proposed building location are the existing Primary Clarifiers 4 and 5. To the east of the proposed building location is existing Primary Clarifier 1, and to the west is the existing digester gas handling structure. In previous design stages, concern with the Primary Scum Building's proximity to the existing Primary Clarifier 1 was identified.

The project involves the design and construction of the Primary Scum Building. The building footprint is approximately 286 m^2. The Primary Scum Building will be comprised of three floors. A lower floor, main floor, and upper floor. The lower floor finished floor elevation (FFE) is 227.05 meters above sea level (ASL), the main floor will have an elevation of 231.05 m ASL, and the second floor will have a FFE of 235.150 m ASL. The roof of the building will be accessible by door, with the top of the penthouse roof elevation at 245.35 m ASL.

The Primary Scum Building was originally designed using a caisson foundation, however, for efficiency of construction the designer has requested driven precast concrete piles and driven steel H piles be considered for the foundation.

3. Investigation Program

3.1 Testhole Drilling and Soil Sampling

AECOM obtained underground service clearances from public utility companies through ClickBeforeYouDigMB. A utility locator identified and marked the private utilities. On July 9, 2024, prior to drilling, a hydro-vacuum was utilized to safely confirm the testholes were drilled away from any existing utilities. Upon completion of the hydrovac excavations, the excavations were backfilled with sand.

The subsurface drilling and sampling program was conducted on July 10 to July 11, 2024. Drilling services were provided by Paddock Drilling under the supervision of AECOM geotechnical field personnel. The proposed testholes are shown on the attached location plan provided in **Appendix B**. Two testholes were drilled for the project using an Acker MP5 track mounted drill rig. The drill rig was equipped with 125 mm solid stem augers. TH24-01 was drilled to auger refusal at a depth of 19.66 meters below ground surface (m BGS) and TH24-02 was drilled to obtain 3 m of bedrock core samples, terminating at a depth of 27.58 m BGS.

Soil samples were obtained directly from the auger flights at depth intervals ranging from 0.3 to 1.5 m. Relatively undisturbed soil samples were also obtained with 75 mm diameter Shelby tubes. Standard Penetration Tests (SPTs) were conducted in both testholes to assess the relative density of cohesionless soils. The soil samples were visually classified in the field and returned to AECOM's soil laboratory in Winnipeg, MB, for additional examination and testing. Cohesive soil samples were tested using a mini torvane and pocket penetrometer to estimate the undrained shear strength and the compressive soil strength.

Upon completion of drilling, the testholes were examined for evidence of sloughing and groundwater seepage. TH24-01 was backfilled with bentonite from 211.23 m BGS to 224.18 m BGS, with sand from 224.18 m BGS to 228.45 m BGS, and top with bentonite to the surface. TH24-02 was backfilled with grout in the bedrock, and with auger cuttings and bentonite from bedrock to surface.

3.2 Laboratory Testing

A laboratory testing program was performed on soil samples obtained during the drilling program to determine the relevant engineering properties of the subsurface materials. Diagnostic testing included moisture contents (ASTM D2216), on all collected soil samples, as well as, particle size analysis (ASTM D422), Atterberg limits tests (ASTM D4318), unconfined compressive strength of intact cohesive soil (ASTM D2166) and unconfined compressive strength tests on rock core specimen (ASTM D2938). In addition, mini torvane and pocket penetrometer readings were taken on auger grab samples. The results of the laboratory testing are shown on the testhole logs in **Appendix C** and in the laboratory test report in **Appendix D**.

4. Investigation Results

Subsurface conditions observed during testhole drilling and sampling were visually documented by AECOM geotechnical personnel in accordance with the Unified Soil Classification System (USCS).

The conditions of the site have been based on the investigation results obtained during the field and laboratory investigation programs. The pertinent results from theses investigations are outlined in the subsequent section below.

4.1 Stratigraphy

The soil stratigraphy on the project site generally consisted of concrete pavement, followed by clay fill overlaying a silt layer. The silt layer was underlain by a thick clay layer. The clay layer was followed by a sandy silty clay till layer, prior to reaching bedrock. A description of the soil stratigraphy is provided below. The detailed testhole records are provided in **Appendix C**, which include a summary sheet outlining the symbols and terms of the testhole record.

4.1.1 Concrete

Concrete was encountered at the ground surface in both testholes. The concrete thickness was approximately 0.15 m.

4.1.2 Fill – Fat Clay (CH)

Fat clay (CH) fill material was encountered below the concrete in TH24-01 and TH24-02. The thickness of the fat clay (CH) fill ranged from 1.37 m to 2.13 m. The fat clay (CH) fill was black in color, moist, of high plasticity, and firm to stiff consistency. The moisture content of the fat clay (CH) fill ranged from 27.10% to 39.70% with an average of 35.48%

4.1.3 Silt (ML)

Silt (ML) was encountered directly below the clay fill in TH24-01 and TH24-02. The silt (ML) ranged in thickness from 1.22 m to 1.52 m. It was encountered at elevations ranging from 229.28 meters above sea level (m ASL) to 227.08 m ASL. The silt was observed to be tan, moist, of low plasticity, and soft. The moisture content of the silt (ML) fill ranged from 16.10% to 22.70% with an average of 19.40%.

4.1.4 Lean Clay (CL)

Lean clay (CL) was encountered directly below the silt (ML) layer in TH24-02. The lean clay (CL) was observed to be approximately 0.91 m thick. The lean clay layer was encountered at elevations ranging from 228.67 m ASL to 227.75 m ASL. The lean clay was observed to be grey, moist, low plasticity and soft to firm. The moisture content of lean clay (CL) ranged from 19.90% to 24.00% with an average of 21.95%.

4.1.5 Fat Clay (CH)

Fat clay (CH) was encountered directly below the silt (ML) layer in TH24-01 and below the lean clay (CL) layer in TH24-02. The fat clay (CH) ranged in thickness from approximately 14.47 to 15.24 m. The fat clay (CH) was encountered at elevations ranging from 227.75 m ASL to 212.51 m ASL. The fat clay (CH) was grey, moist, of high plasticity. The fat clay (CH) layer began as firm to stiff and transitioned to soft with depth. The moisture content of the fat clay (CH) ranged from 34.70% to 65.50% with an average of 53.03%.

4.1.6 Sandy Silty Clay (CL-ML) Till

Sandy silty clay (CL-ML) till was encountered below the fat clay (CH) in TH24-01 and TH24-02. The sandy silty clay (CL-ML) was encountered at elevations ranging from 212.6 m ASL to 206.26 m ASL. Auger refusal was met in the sandy silty clay (CL-ML) till in this range. The sandy silty clay (CL-ML) till was tan in color. SPTs completed within the sandy silty clay (CL-ML) till showed uncorrected "N" values ranging from 18 to >50 per 300 mm of penetration, classifying the materials as compact to very dense in relative density. The moisture content ranged from 8.10% to 14.60% with an average of 10.45%. In the sandy silty clay (CL-ML) till layer, it was common to find cobbles and boulders.

4.1.7 Bedrock

Bedrock (BR) was encountered in TH24-02. To advance through the bedrock, coring methods were used. The bedrock was observed to be mottled dolomitic limestone; a Selkirk Member of the Red River Formation. The limestone was observed at an elevation of 209.62 m ASL to 203.22 m ASL. It should be noted that the coring was terminated at this elevation based on the scope of work, however, the bedrock may advance further. At an elevation of 207.33 m ASL, the water used for coring was no longer returning to the surface. The quality and strength of the bedrock varied significantly which will be discussed further in **Section 4.3**. **Section 4.3.1** describes the total core recovery (TCR), **Section 4.3.2** describes the solid core recovery (SCR), **Section 4.3.3** describes the rock quality designation (RQD), and **Section 4.3.4** describes the bedrock classification results.

4.1.8 Groundwater and Sloughing Conditions

Groundwater seepage and soil sloughing conditions were observed in both testholes upon completion of drilling. Details of the location and nature of the sloughing, seepage, and groundwater encountered are provided in the testhole records in **Appendix C** and presented in **Table 1**.

Table 1: Observed Groundwater Seepage and Sloughing Conditions

Testhole No.	Depth of Groundwater Seepage (m BGS)	Groundwater Depth Upon Completion of Drilling (m BGS)	Groundwater Elevation (m ASL)	Depth of Soil Sloughing (m BGS)
TH24-01	-*	12.80	218.09	2.29
TH24-02	2.29	6.71	224.09	1.52

Note: Groundwater seepage was not observed due to switching of drilling method to hollow stem augers because of sloughing.

Groundwater readings were taken periodically using a standpipe installed in TH24-01. The readings recorded are summarized in

Table 2: Groundwater Readings

Standpipe	Stratum/Tip Elev.	Groundwater Elevation (m ASL)			
		July 12, 2024	July 16, 2024	July 19, 2024	July 24, 2024
SP24-01	fat clay/224.89	229.01	229.17	229.08	229.07

A graphical summary of these results are provided in **Figure 1.**

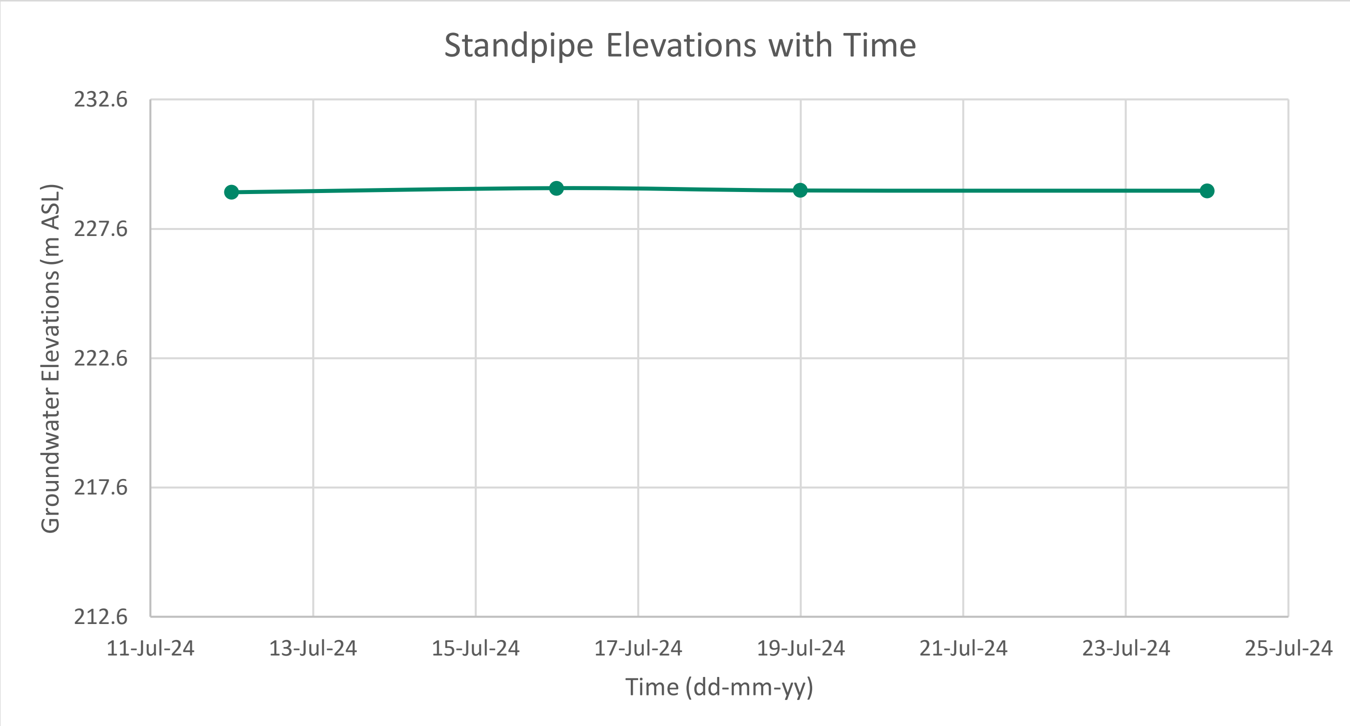


Figure 1 - Graph of Groundwater Elevations Versus Time

Only short-term seepage and sloughing conditions were observed in the testholes. Groundwater levels will normally fluctuate during the year and will be dependent on precipitation, surface drainage, and regional groundwater regimes. Groundwater seepage and soil sloughing should be expected from the sandy silty clay (CL-ML) till layer.

4.2 Laboratory Test Results

Table 3: Particle Size Analysis

Testhole No.	Sample Depth	Soil Type	Particle Size			
			Gravel 75 to 4.75 mm	Sand <4.75 to 0.075 mm	Silt <0.075 to 0.002 mm	Clay <0.002 mm
TH24-01	5.49 – 6.10 m	CH	0.0%	0.5%	22.4%	77.0%
TH24-02	2.13 – 2.29 m	CL	0.4%	5.4%	81.0%	13.1%
TH24-02	15.24 – 15.85 m	CH	1.0%	4.6%	22.7%	71.6%
TH24-02	19.81 – 19.96 m	CL-ML	4.8%	33.9%	46.3%	15.0%

Table 4: Atterberg Limits Test Data

Testhole No.	Sample Depth	Soil Type	Liquid Limit	Plastic Limit	Plasticity Index	Activity
TH24-01	5.49 – 6.10 m	CH	89	24	65	0.84
TH24-02	2.13 – 2.29 m	CL	23	15	8	0.61
TH24-02	15.24 – 15.85 m	CH	79	20	59	0.82
TH24-02	19.81 – 19.96 m	CL-ML	17	10	7	0.47

Table 5: Unconfined Compressive Strength Test (Soil)

Testhole No.	Sample Depth	Soil Type	Moisture Content (%)	Bulk Unit Weight (kN/m^3)	Unconfined Compressive Strength (kPa)	Undrained Shear Strength (kPa)
TH24-01	1.52 – 2.13 m	Clay	38.2	18.4	81.70	40.85
TH24-01	3.05 – 3.66 m	Silt	16.1	-	-	-
TH24-01	4.57 – 5.18 m	Clay	52.7	16.8	80.75	40.37
TH24-01	6.10 – 6.71 m	Clay	58.1	16.9	76.21	38.10
TH24-02	7.62 – 8.23 m	Clay	58.1	16.9	76.60	38.30
TH24-02	9.14 – 9.75 m	Clay	51.3	16.7	63.93	31.96
TH24-02	10.67 – 11.28 m	Clay	44.0	17.6	75.78	37.89
TH24-02	12-19 – 12.80 m	Clay	52.6	17.1	56.80	28.40

Table 6: Unconfined Compressive Strength of Intact Rock Core Specimens

Testhole No.	Sample Depth	Maximum Load (kN)	Compressive Strength (MPa)
TH24-02	26.11 – 26.37 m	417	134
TH24-02	26.37 – 26.66 m	363	117
TH24-02	26.67 – 26.92 m	359	115
TH24-02	27.17 – 27.39 m	321	103

4.3 Bedrock Classification

The rock strength can be categorized with the unconfined compressive strength of the rock based on International Society of Rock Mechanics (ISRM) Standard (1979) as shown in **Table 7**.

Table 7: Rock Strength Categorization

Grade	Term	Unconfined Compressive Strength (MPa)
R6	Extremely Strong	>250
R5	Very Strong	100 – 250
R4	Strong	50 – 100
R3	Medium Strong	25 – 50
R2	Weak	5 – 25
R1	Very Weak	1 – 5
R0	Extremely Weak	0.25 – 1

The results of the unconfined compressive strength tests ranged from 103 MPa to 134 MPa. AECOM can conclude the rock strength categorization was very strong.

4.3.1 Total Core Recovery (TCR)

Total core recovery (TCR) is the testhole core recovery percentage. TCR is expressed as follows:

$$TCR\ (\%) = \frac{sum\ of\ recovered\ core\ length}{total\ core\ length}\ x\ 100$$

The TCR was calculated for each bedrock core run advanced within the testholes. A summary of the TCR values is provided in **Table 9**. The TCR ranged from 73.3% to 100.0%.

4.3.2 Solid Core Recovery (SCR)

Solid core recovery (SCR) is the testhole core recovery percentage of solid cylindrical rock. SCR is expressed as follows:

$$SCR\ (\%) = \frac{sum\ of\ recovered\ solid\ cylindrical\ core\ lengths}{total\ core\ length}\ x\ 100$$

The SCR was calculated for each bedrock core run advanced within the testhole. A summary of the SCR values are provided in **Table 9**. The SCR ranged from 10.0% to 87.5%.

4.3.3 Rock Quality Designation (RQD)

RQD is based on the ISRM classification system. The RQD is an indirect measure of the number of fractures and the amount of jointing in the rock mass. The RQD is expressed as a percentage of the ratio of summed core lengths (greater than 10 cm) to the total length cored. The RQD index is used to provide a classification of the rock quality shown in **Table 8**.

Table 8: Rock Classification Ranges

RQD (%)	Rock Quality Designation
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Rock quality designation (RQD) is expressed as follows:

$$RQD\ (\%) = \frac{sum\ of\ recovered\ core\ lengths\ greather\ than\ 10\ cm}{total\ core\ length}\ x\ 100$$

The RQD was calculated for each core run advanced within TH24-02. A summary of the RQD values are provided in **Table 9**. The RQD ranged from 10.0% to 84.2%.

4.3.4 Bedrock Classification Results

Based on the rock classification and laboratory test results, the encountered bedrock classification was of very poor to good quality, with a rock strength of very strong.

Table 9: TCR, SCR and RQD Results

Testhole ID	Sample Number	Core Run No.	Core Run Depth (m BGS)	Elevation (m ASL)	TCR (%)	SCR (%)	RQD (%)
TH24-02	C17	1	21.24 – 21.49	209.56 – 209.31	Till/Boulders	Till/Boulders	Till/Boulders
	C18	2	21.49 – 23.01	209.31 – 207.79	Till/Boulders	Till/Boulders	Till/Boulders
	C19	3	23.01 – 24.54	207.79 – 206.26	Till/Boulders	Till/Boulders	Till/Boulders
	C20	4	24.54 – 26.06	206.26 – 204.74	73.3	10.0	10.0
	C21	5	26.06 – 27.58	204.74 – 203.22	100.0	87.5	84.2

Coring was required to advance through the till due to the density and presence of cobbles and boulders. At an elevation of 206.26 m ASL, evidence of bedrock became present. However, this initial bedrock was fractured and of very poor quality. The quality of bedrock increased with depth, where between elevations of 204.74 m ASL to 203.22 m ASL the quality of rock became good.

5. Geotechnical Concerns

Based on our current understanding of the proposed development and the results of our geotechnical investigation, the primary geotechnical concerns at the project site are:

- Movement related to volume change of the high plasticity clay fill and clay due to the moisture content or rebound.
- Potential for two water bearing layers: a perched water table in the silt (ML) layer at shallower depths and the static groundwater level in the water bearing zone in the lower part of the sandy silty clay (CL-ML) till layer.
- Vibration caused by driven pile installation during construction which may cause damage to existing structures and utilities

These issues will be discussed in the following sections.

6. Recommendations

6.1 Foundation Design

Based on the soil and groundwater conditions encountered at the testhole locations, driven precast concrete piles and driven steel H piles were evaluated for foundation options. Design parameters for driven piles are provided in the sections below. It is generally recommended that different foundation systems no be used to support the same structure unless they are used to support independent structural elements of the structure.

6.1.1 Limit States Design

The use of Limit States Design (LSD) is required for the design of buildings and their structural components including foundations according to the 2020 National Building Code of Canada (NBCC) The limit states are classified into two groups: the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

The Ultimate Limit State case is primarily concerned with structural collapse and hence, safety. For foundation design, ultimate limit state consists of:

- Exceeding the load carrying capacity of the foundation;
- Sliding;
- Large deformation of foundation, leading to an ultimate limit state being induced in the superstructure or building;
- Overturning; and,
- Loss of overall stability.

The factored resistance of the ULS is the ultimate geotechnical resistance multiplied by the appropriate resistance factor.

The Serviceability Limit State (SLS) case considers mechanisms that restrict or constrain the intended use or occupancy of the structure. They are typically associated with movements that interrupt or hinder the purpose of the structure. For foundation design, serviceability limit state consists of:

- Excessive movements; and,
- Unacceptable vibrations.

The SLS case is addressed by determining the maximum available resistance to keep the foundation under service loads within tolerable limits as provided by the structural engineer. Unfactored permanent and transitory loads are used for calculating total deformation in non-cohesive soils. Unfactored permanent loads and appropriate portions of transitory loads are used for the initial and time-dependent final deformations of cohesive soils. Therefore, the foundation loads and serviceability tolerances must be known to properly determine the SLS resistance values. In cases where tolerable movements are not provided by the structural engineer, the tolerable limit of the total settlement for foundations subject to compression is typically assumed to be 25 mm.

6.1.2 Frost

6.1.2.1 Frost Penetration

The depths of frost penetration have been estimated for a range of annual air freezing identified in **Table 10**. The annual freezing index was inferred from Figure K-4 of the National Building Code of Canada (2020) Commentary document. The ten-year return annual freezing index was calculated using the mean annual freezing index and recommendations outlined in the Canadian Foundation Engineering Manual (CFEM 4e). The fifty-year return annual freezing index was taken from Figure K-5 of the National Building Code of Canada (2020) Commentary document. Factors such as snow cover, vegetation at surface, soil type and groundwater conditions can all significantly impact the depth of frost penetration. The predominant soil type of the project site is fat clay.

Table 10: Frost Penetration Depth

Parameter	Period		
	Mean	10-Year Return	50-Year Return
Annual Air Freezing Index (°C-days)	1825	1875	2375
Estimated Frost Penetration (Fat Clay Subgrade) – gravel surface, no snow cover (m)	1.9	2.0	2.5
Estimated Frost Penetration (Fat Clay Subgrade) – grass with snow cover (m)	1.7	1.9	2.2

For foundation design considerations, the CFEM recommends using the ten-year return annual freezing index to predict frost penetration. It is the responsibility of the design team to select an adequate frost penetration depth to be incorporated into the design.

6.1.2.2 Frost Susceptibility

The qualitative frost susceptibility of a soil is typically assessed using guidelines developed by Casagrande (1932) based on the percentage by weight of the soil finer than 0.02 mm, and the plasticity index. The classification system has been adapted by the U.S. Army Corps of Engineers and the Canadian Foundation Engineering Manual (2006). Soils are classed as F1 through F4 in order of increasing frost susceptibility.

The soils (fat clay and silt) encountered during the geotechnical investigation fall mostly within the frost groups F3 and F4. The F3 group has high to very high susceptibility to frost and F4 has a very high susceptibility. Frost susceptibility has been assigned to the encountered soil type and is summarized in **Table 11**.

Table 11: Frost Susceptibility

Soil Unit	USCS Soil Type	Frost Group	Frost Susceptibility
Fat clay/Fat clay fill	CH	F3	High to very high susceptibility
Silt	ML	F4	Very high susceptibility

(1) Source: Canadian Foundation Engineering Manual (CFEM, 4e), Chapter 13 Frost Action

6.1.3 Adfreezing

Frozen soil in contact with foundation elements can develop an adfreeze bond which can result in uplift forces on the foundation. The CFEM (Canadian Foundation Engineering Manual, 4E) lists adfreeze bond stresses of 100 kPa for fine grained soils to steel and 65 kPa for fine grained soils to concrete.

This adfreeze stress should be applied to the perimeter of the piles for unheated structures to a depth of 2.0 m measured from final grade. The uplift forces from adfreeze stresses are resisted by the permanent dead load of the structure plus the uplift resistance of the foundation element. More details are provided in **Section 6.1.4** and **6.1.5**.

6.1.4 Driven Precast Concrete

A foundation system suitable for moderate to heavy foundation loads is system of driven, pre-stressed, precast concrete piles. These piles, when driven to practical refusal with a hammer capable of delivering a minimum rated energy of 40 kJ per blow, may be designed based on the factored geotechnical axial compression resistances and axial tension resistances shown in **Table 12**.

Table 12: Geotechnical Axial Resistance for Precast Concrete Piles

Nominal Pile Size	Factored Geotechnical Resistance in Axial Compression at ULS (1) $\Phi = 0.4$	Factored Geotechnical Resistance in Axial Tension at ULS (2) $\Phi = 0.3$	Refusal Criteria
305 mm	550 kN	112 kN	5 blows/25 mm
356 mm	750 kN	130 kN	8 blows/25 mm
406 mm	1000 kN	149 kN	12 blows/25 mm

Notes:

(1) As per 2020 NBCC, a resistance factor of 0.4 is used for calculating the factored geotechnical shaft resistance in compression at ULS.

(2) As per 2020 NBCC, a resistance factor of 0.3 is used for calculating the factored geotechnical shaft resistance in axial tension at ULS.

For piles end-bearing on dense till or bedrock, SLS conditions generally do not govern the design since the loads required to induce 25 mm of movement (i.e., the typical SLS criteria) exceed those at ULS.

Assuming a unit adfreeze bond of 65 kPa in the upper 2.0 m of precast concrete piles in unheated areas, uplift forces from frost adfreeze of 125 kN, 146 kN, and 166 kN are possible for pile sizes of 305 mm, 356 mm, and 406 mm, respectively. It should be noted by the structural engineer that these provided uplift forces have not been factored, and the structural engineer must apply appropriate load factors. If piles are left for a period of time during winter conditions, risk of the piles heaving due to frost heave is possible. It is the responsibility of the structural engineer to consider this heave potential and design for it.

The refusal criteria indicated in **Table 12** should be achieved at least three times for the final resistance. Due to the proximity of nearby structures, pre-boring to a depth of approximately 6.0 m should be considered for all driven piles to enhance pile alignment, and limit vibrations. The installation of driven precast concrete piles will cause vibration on adjacent structures, the contractor should document any cracks or settlement of existing structures to ensure no additional damage is incurred. More information regarding pile vibration monitoring is provided in **Section 6.1.6**. The pre-bored hole diameter should be slightly larger than the nominal pile diameter. Pre-boring the pile locations will reduce the lateral support along the pre-bored depth of the pile. To maintain lateral support along the pile, the annulus (i.e., space between the pile and the pre-bored soil) should be filled with grout.

All piles should be driven continuously to their required depth once driving is initiated. Pile heave for piles within five pile diameters of each other should be monitored and re-driving should be done where pile heave occurs. Pile heave more than 10 mm require redriving of the piles. A surveyor should record the pile elevations upon completion of pile driving, to correct the pile heave, if needed. Pile spacing should not be less than 2.5 pile diameters, measured center to center. In the Winnipeg area, precast concrete piles driven to practical refusal will develop most of their capacity from toe resistance, and therefore, a reduction in pile capacity is generally not required for group action. Settlement beyond the elastic compression of the pile is expected to be less than 10 mm with an end-bearing pile system for the anticipated geotechnical axial resistance.

Auger refusal was encountered at elevations ranging from 211.90 m ASL to 211.23 m ASL. From observations made during drilling, auger refusal was encountered in dense till with cobbles and boulders in the two testholes. In our experience in the Winnipeg area, driven precast concrete piles will typically reach the required refusal criteria at the depth of auger refusal on suspected dense till with cobbles and boulders (i.e., approximately 211 m ASL).

The depth of pile penetration at the project site will depend on localized till and bedrock conditions. Auger refusal was encountered at an approximate elevation of 211.23 m ASL. Based on our experience within Winnipeg, piles refuse at elevations where the till layer becomes dense to very dense, or near auger refusal elevations. Cobbles and boulders were both encountered during the site investigation; thus, cobbles and boulders may be encountered within the sandy silty clay (CL-ML) till layer during pile installation. There is therefore potential for piles to refuse in sandy silty clay (CL-ML) till due to the presence of boulders and develop insufficient lateral capacity.

A minimum void space of 150 mm should be provided beneath all pile caps and grade beams to accommodate potential heave of the high plasticity clay. To ensure that the piles achieve their design capacities, full time inspection by AECOM geotechnical personnel is recommended during pile installation. It is generally recommended that different foundation systems not be used to support the same structure, unless they are used to support independent structural elements of the structure.

6.1.5 Driven Steel H Piles

6.1.5.1 Pile Capacity

The capacity of steel H piles driven to practical refusal on the underlying bedrock could potentially approach the structural capacity of the steel member. Based on AECOM's experience, it has been observed that the capacities of steel H piles driven to practical refusal on dense till or fractured bedrock materials are generally within the range of 40% to 60% of the structural capacity of the steel member. It is assumed that the ultimate axial capacity is assumed to be 50% of the structural capacity of the steel, therefore:

$$Q_u = 0.5A_tF_y'$$

Where:

A_t = 0.0141 m^2 for HP310x110 and 0.0222 m^2 for HP360x174 (cross sectional area of the pile tip).

F_y' = 350 MPa (yield stress of the pile)

For driven HP310x110 piles and HP360x174 piles, potential axial compression capacities at ULS based on 50% of the structural capacity of the steel is given in **Table 13**.

Table 13: Driven Steel H Pile Capacity Based on Structural Strength

Pile Size	Estimated Pile Embedment Length Below Existing Grade (1)	Axial Compression at ULS		Axial Tension at ULS
		RF = 0.4 (2)	RF = 0.5 (3)(5)	RF = 0.3 (4)(6)
HP310x110	26.06 m	987 kN	1234 kN	144 kN
HP360x174	26.06 m	1554 kN	1943 kN	172 kN

Notes:

(1) Based on ground elevation for TH24-02.

(2) As per 2020 NBCC, when semi-empirical analysis using laboratory and in situ test data is available, a resistance factor of 0.4 is used for calculating the geotechnical shaft resistance in compression at ULS.

(3) As per 2020 NBCC, when analysis using dynamic monitoring results is available, a resistance factor of 0.5 is used for calculating the factored geotechnical shaft resistance in compression at ULS.

(4) As per 2020 NBCC, when uplift resistance by semi-empirical analysis is available, a resistance factor of 0.3 is used for calculating the factored geotechnical shaft resistance in tension at ULS.

(5) To use axial compression at ULS value using an RF of 0.5, PDA must be completed on at least 5% of the production piles.

(6) An assumption for the thickness of clay was made based off TH24-01 and TH24-02 using a clay thickness of 12 m.

As stated above, SLS conditions generally do not govern the design since the loads required to induce 25 mm of movement exceed those at ULS. Vertical settlements of steel H piles driven to refusal are expected to be negligible.

Assuming a unit adfreeze bond of 100 kPa in the upper 2.0 m of steel HP310x110 and HP360x174 piles in unheated areas, uplift forces from frost adhesion of 365 kN and 439 kN, respectively are possible. It should be noted by the structural engineer that these provided uplift forces have not been factored, and the structural engineer must apply the proper load factors. This capacity does not include the buoyant weight of the pile or potential permanent loading.

The estimated axial capacities for the driven steel HP310x110 and HP360x174 piles are given in **Table 13** have been based on the following assumptions:

1. For the calculations of resistance in axial tension at ULS (excluding adfreeze) and frost adhesion uplift resistance, the frictional capacity in the upper 2.0 m of the pile has been ignored to account for potential soil drying and shrinking near the ground surface.

2. Geotechnical resistance factors (RF) of 0.4 and 0.5 for axial compression and 0.3 for axial tension have been used as per the NBCC (2020).

3. To use the axial compression at ULS value using an RF of 0.5, Pile Driving Analyzer (PDA) testing must be completed on at least 5% of the production piles. Refer to **Section 6.1.5.4** for complete details.

4. A minimum void space of 150 mm should be provided beneath all structural elements to accommodate potential heave of the high plasticity clay fill and clay.

The piles should be driven with a minimum pile spacing of 2.5 diameters measured center to center within pile groups. Pile heave should be monitored, and piles should be re-driven when pile heave is observed. Pile heave more than 10 mm require redriving of the piles. A surveyor should record the pile elevations upon completion of pile driving, to correct the pile heave, if needed. The installation of driven steel H piles will cause vibration on adjacent structures, the contractor should document any cracks or settlement of existing structures to ensure no additional damage is incurred. More information regarding pile vibration monitoring is provided in **Section 6.1.6.**

To help minimize the damage to the end of the pile during the driving process, a driving shoe should be installed at the end of each pile. The driving shoe should not extend beyond the pile perimeter tip area of the steel H pile to prevent disturbance of the soils during installation of the pile.

6.1.5.2 Pile Type

Prior to the pile installation, the piles should be inspected to confirm that the material specifications are satisfied. As a minimum, steel piles should meet the requirements of CAN/CSA-G40.20/G40.21, Grade 350W. The piles should be free from protrusions, which could create voids in the soil around the pile during driving.

6.1.5.3 Pile Driving Criteria

During the installation of the driven steel piles, the maximum compression and tension stresses developed within any pile (commonly referred to as the driving stresses) should be limited to $0.9F'_y$.

The hammer energy delivered to the pile head for driving the steel piles should be a minimum of 60 kJ for piles based on structural strength. This hammer energy is for a hydraulic hammer. For other hammer types, the required energy may vary depending on the energy transfer ration.

On a preliminary basis, the definition of practical refusal may be taken as 15 blows per each 25 mm interval for three consecutive sets. The driving criteria can be developed using a wave equation analysis program (GRLWEAP) once the hammer type, hammer energy and pile type are confirmed, and the pile loads have been proven by PDA tests.

6.1.5.4 Pile Driving Analyzer Tests

To use a geotechnical resistance factor of 0.5 for axial compression, Pile Driving Analyzer (PDA) tests must be conducted on approximately 5% of the piles during installation. These tests should be performed both at the end of initial drive (EOID) of the pile and at the beginning of the restrike (BOR) of the pile to ensure that the piles reach and maintain the specified capacity. At EOID, the piles should be driven to the design depth. If piles do not reach their expected capacity at EOID, the piles will be tested at BOR after a period of 24 to 72 hours. The energy for BOR pile tests shall be determined prior to BOR pile testing.

The designer should have Case Pile Wave Analysis Program (CAPWAP) analyses performed in conjunction with PDA tests during pile installation monitoring to confirm expected axial pile capacities.

6.1.5.5 Pile Installation Monitoring

The designer should consider monitoring of the pile installation by an AECOM geotechnical inspector to verify that the piles are installed in accordance with design assumptions and the driving criteria are satisfied. For each pile, a complete driving record in terms of the number of blows per 300 mm of penetration should be recorded by the inspector and reviewed during pile installation by the designer.

6.1.6 Pile Vibration Monitoring

While driving the piles, large vibrations are created that may affect nearby structures. A comprehensive survey of the existing structures should be completed prior to pile driving to catalogue any existing damage in the adjacent structures. A structural engineer should recommend a maximum peak particle velocity to prevent additional damage caused to the nearby structures. AECOM recommends full time vibration monitoring is implemented during the installation of the piling system. Upon completion of the pile driving, the nearby structures should again be surveyed to ensure no new additional damage has resulted from the pile installation.

6.1.7 Drag Load

Consolidation settlement of the native clay layer caused by fill material may potentially induce drag load (i.e., negative skin friction on deep foundation elements. Fill materials are not expected due to the lower floor finished floor elevation (FFE) of 227.05 m ASL. These finish floor elevations result in the need for cutting material, therefore there is no drag load.

6.1.8 Lateral Earth Pressure

The lateral earth pressure on the below-grade walls of the Primary Scum Building due to soil pressure may be calculated based on the following conventional relationship, which produces a triangular pressure distribution assuming horizontal ground next to the buried wall. If the ground surface slopes significantly away from the wall (more than required for surface runoff), the design pressure should be re-evaluated.

$$P = K_o(\gamma_b D + q) + D\gamma_w$$

Where:

P = Lateral earth pressure at depth, D (kPa)
K_o = At-rest earth pressure coefficient behind the walls (from table below)
γ_b = Bulk unit weight of soil (from table below)
D = depth from ground surface to point of pressure calculation (m)
q = surface surcharge pressure, if any (kPa)
γ_w = unit weight of groundwater (9.81 kN/m^3)

Table 14: Lateral Earth Pressure Design Parameters

USCS Soil Type	Soil Unit Weight (kN/m^3)	Effective Angle of Internal Friction (Φ')	At-Rest Lateral Earth Pressure Coefficient (K_o)	Active Lateral Earth Pressure Coefficient (K_a)	Passive Lateral Earth Pressure Coefficient (K_p)
Fat Clay Fill	18	17	0.71	0.55	1.83
Fat Clay	17	20	0.66	0.49	2.04
Silt	18	24	0.59	0.42	2.37

Granular backfill is recommended between the existing soils and the walls of the structure. The granular backfill should be sufficiently compacted to minimize settlement of the backfill itself. The backfill should be compacted to a minimum of 95% of the standard proctor maximum dry density (SPMDD) within ±2% of the optimum moisture content (OMC). Placement of the backfill should be undertaken in such a manner as to prevent unbalanced forces from acting on the sides of the structure. Compaction by heavy equipment which could cause excessive lateral pressure on the walls should be avoided. All material within 1.0 m from the walls should be compacted using manually operated pad tampers. A 500 mm clay seal at the ground surface is recommended to reduce surface water infiltration. Grading should be maintained to provide positive surface drainage away from the structure.

It is not recommended to use the native clay as backfill materials immediately behind the walls, as this material is not free-draining and could contribute to buildup of hydrostatic pressure behind the walls at levels above the natural water table.

Where traffic or other live loads may trave or operate near the walls, the horizontal pressure due to the live load should be added to the lateral earth pressure.

6.2 Seismic Considerations

As per Table 6.1A of the CFEM, the site classification for seismic site response is dependent on the average properties in the top 30 m of the soil profile. Based on a soil profile having more than 3 m of high plasticity clay, a Seismic Site Class E can be assigned to the site.

The 2020 National Building Code of Canada (NBCC) Seismic Hazard Calculation for the site is provided in **Appendix E**. It includes values of spectral acceleration (for time periods of 0.2, 0.5, 1.0, 2.0, 5.0 and 10.0 seconds), peak ground acceleration, and peak ground velocity for 2%, 5%, and 10% probability of exceedance in 50 years.

6.3 Temporary Shoring

It is anticipated that temporary shoring will be used to facilitate excavation for the lower level of the Primary Scum Building. Comments regarding the design and temporary shoring system are therefore provided as follows. The design of the temporary shoring system should be carried out by a professional engineer specialized in shoring design. The shoring design should be signed, sealed and submitted for review to AECOM.

It is anticipated that the maximum excavation depth for the lower level of the Primary Scum Building will be approximately 3.75 m BGS. The depth of excavation is relatively deep; thus, shoring such as sheet pile walls and additional bracing may be required. The installation of the sheet pile walls will cause vibrations on adjacent structures, the contractor should document any cracks or settlement of existing structures to ensure no additional damage is incurred.

Groundwater elevation was recorded as high as 229.17 m ASL. It should be noted that groundwater levels observed may not be representative of stable groundwater conditions. Seasonal fluctuations due to precipitation, snow melting, drainage conditions on site and other factors may influence the groundwater levels recorded over time. Therefore, groundwater conditions at the time of construction may vary from the recorded groundwater depths above. Construction dewatering should be expected to isolate the work zone and facilitate construction in dry conditions; therefore, provisions for dewatering and groundwater control should be accounted for in the project schedule and cost.

A perimeter ditch and associated pumping and an appropriate dewatering system should be provided to intercept surface runoff and groundwater from entering the excavation. The contractor shall submit an engineered excavation plan, including dewatering measures, for engineer review. The excavation shall abide by *The Manitoba Workplace Safety and Health Act and Regulations*.

Monitoring must be carried out during the construction process and following construction to confirm that movements of the temporary shoring system are within a pre-determined acceptable range.

6.4 Foundation Concrete

Clay soils in the Winnipeg area contain sulphates that will cause deterioration of concrete. The class of exposure for concrete in contact with clay soil in Winnipeg is severe (S-2 CSA A23.1-09 Table 3). The requirements for concrete exposed to severe sulphate attack are provide in **Table 15**.

Table 15: Foundation Concrete Requirements

Parameter	Design Requirements
Class Exposure	S-2
Compressive Strength	32 MPa at 56 days
Air Content	4 to 7%
Water-to-Cement Materials Ratio	0.45 max.
Cement	Type HS or HSb

7. Quality Assurance and Quality Control

Prior to construction, it is recommended that the contractor provides an approved quality assurance and quality control program (QA/QC). This program should include but is not limited to periodic testing of granular gradation, L.A. abrasion loss, material proctors, field density tests, and PDA testing.

Upon completion of the excavation, AECOM must be present to observe the subgrade conditions and confirm the soil matches our assumptions and expectations.

8. Design Review, Construction Monitoring and Testing

AECOM should be retained to review the foundation plans and specifications for conformance with the intent of this report. During construction, it is recommended that an AECOM representative be involved with the following tasks:

- Inspection of foundation installation; and

The purpose of the foundation inspection services would be to provide AECOM the opportunity to observe the soil conditions encountered during construction, evaluate the applicability of the information presented in this report to the soil conditions encountered, and provide appropriate changes in the design or construction procedures if conditions differ from those described herein.

9. References

Canadian Commission on Building Fire Codes, (2020). National Building Code of Canada (NBCC) 2020. National Research Council of Canada 2022.

Canadian Geological Society. (2006). Canadian Foundation Engineering Manual 4th Edition.

Appendix A

Site Photos

TH24-01 – Sawcut Concrete at Testhole Location

TH24-01 – Solid Stem Auger

TH24-01 – Grouted with Flush Mount Standpipe Piezometer Installed

TH24-02 – Testhole Location

TH24-02 – Case and Core Barrels

TH24-02 – Concrete Patched

Appendix B

Testhole Location


Primary Scum Building
Proposed Testhole Location Plan @ North End Water Pollution Control Centre
Legend
Auger Refusal
Coring 10' into bedrock
TH24-01
TH24-02
Google Earth
Image © 2024 Airbus
100 m
N

Appendix C

Testhole Logs

EXPLANATION OF FIELD & LABORATORY TEST DATA

The field and laboratory test results, as shown for each hole, are described below.

1. EXPLANATION OF SOIL

Each soil stratum is classified and described noting any special conditions. The Modified Unified Classification System (MUCS) is used. The soil profile refers to the existing ground level at the time the hole was done. Where available, the ground elevation is shown. The soil symbols used are shown in detail on the soil classification chart.

1.1 Tests on Soil Samples

Laboratory and field tests are identified by the following and are on the logs:

γ_D - Dry Unit Weight. Usually expressed in kN/m^3.

γ_T - Total (moist, wet, or bulk) Unit Weight. Usually expressed in kN/m^3.

C_U - Undrained Shear Strength. Usually expressed in kPa. This value can be determined by a field vane shear test and may also be used in determining the allowable bearing capacity of the soil.

C_{PEN} - Pocket Penetrometer Reading. Usually expressed in kPa. Estimate of the undrained shear strength as determined by a pocket penetrometer.

N - Standard Penetration Test (SPT) Blow Count. The SPT is conducted in the field to assess the in-situ consistency of cohesive soils and the relative density of non-cohesive soils. The N value recorded is the number of blows from a 63.5 kg hammer free falling of 760 mm (30 in.) which is required to drive a 50 mm (2 in.) split spoon sampler 300 mm (12 in.) into the soil.

Q_U - Unconfined Compressive Strength. Usually expressed in kPa and may be used in determining allowable bearing capacity of the soil.

The following tests may also be performed on selected soil samples and the results are given on separate sheets enclosed with the logs:

- Grain Size Analysis
- Standard or Modified Proctor Compaction Test
- California Bearing Ratio Test
- Direct Shear Test
- Permeability Test
- Consolidation Test
- Triaxial Test

1.2 Natural Moisture Content

The relationship between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits for a sample should be compared to its natural moisture content and plotted on the Plasticity Chart to determine the soil classification.

Descriptive Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually in coarse-grained soils below the water table

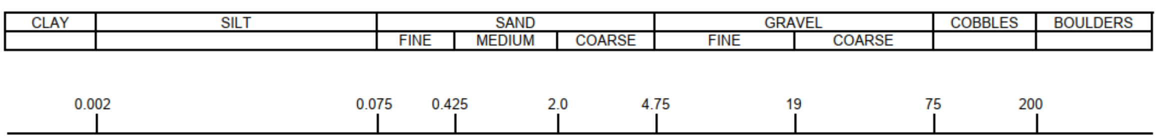
1.3 Grian Size Distrubtion

Laboratory grain size analyses provided by AECOM follow the following system. Note that, with the exception of those samples where a grain size distribution analysis has been completed, all samples have been classified by visual inspection. Visual inspection classification is not sufficient to provide exact gain sizing.

SOIL COMPONENTS

FRACTION		SIEVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	IDENTIFIER
GRAVEL	COARSE	75	19	50 – 35	AND
	FINE	19	4.75		
SAND	COARSE	4.75	2.00	35 – 20	ADJECTIVE
	MEDIUM	2.00	0.425		
	FINE	0.425	0.075	20 – 10	SOME
SILT (non-plastic) or CLAY (plastic)		0.075		10 – 1	TRACE
OVERSIZE MATERIALS					
ROUNDED OR SUB-ROUNDED COBBLES 75 mm TO 200 mm BOULDERS >200 mm			ANGULAR ROCK FRAGMENTS ROCKS > 0.75 m3 IN VOLUME		

ISSMFE / USCS SOIL CLASSIFICATION



EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

1.4 Soil Compactness and Consistency

The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by in-situ vane tests, penetrometer tests, unconfined compression tests, or similar field and laboratory analysis. Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine-grained, cohesive soils.

The standard terminology to describe cohesionless soils includes the compactness condition as determined by the Standard Penetration Test 'N' value. These approximate relationships are summarized in the following tables:

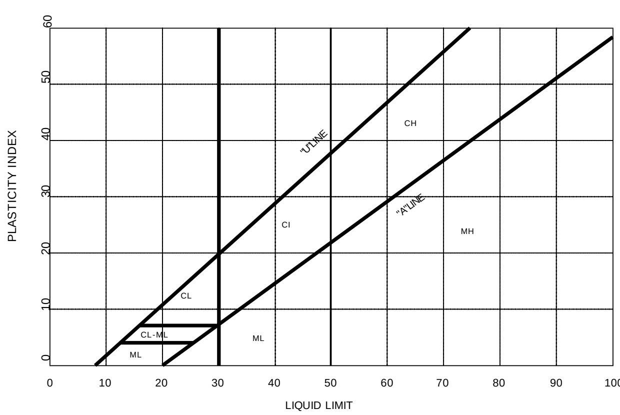
Table 1 Cohesive Soils

Consistency	SPT N (blows/0.3m)	C_u (kPa) approx.
Very Soft	<2	<12
Soft	2 - 4	12 - 25
Firm	4 - 8	25 - 50
Stiff	8 - 15	50 - 100
Very Stiff	15 - 30	100 - 200
Hard	>30	>200

Table 2 Cohesionless Soils

Compactness Condition	SPT N (blows/0.3m)
Very Loose	0 – 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	>50

MAJOR DIVISION			UCS	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4 \quad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE W_p LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE W_p MORE THAN 7
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE R NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6 \quad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE W_p LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE W_p MORE THAN 7
FINE GRAINED SOILS	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	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER 'F'. E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
	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 OF 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		
FILL			FILL	SEE REPORT DESCRIPTION		



NOTE:
1. BOUNDARY CLASSIFICATION POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%

SOIL COMPONENTS

FRACTION		SIEVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	IDENTIFIER
GRAVEL	COARSE	75	19	50 – 35	AND
	FINE	19	4.75		
SAND	COARSE	4.75	2.00	35 – 20	_____Y
	MEDIUM	2.00	0.425		
	FINE	0.425	0.075	20 – 10	SOME
SILT (non-plastic) or CLAY (plastic)		0.075		10 – 1	TRACE

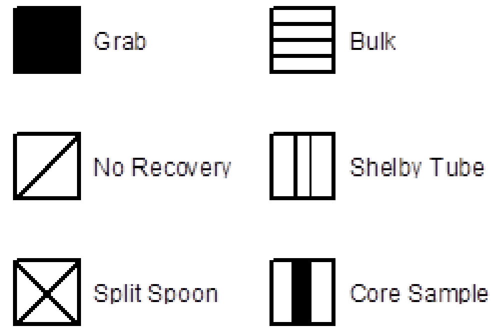
OVERSIZE MATERIALS	
ROUNDED OR SUB-ROUNDED COBBLES 75 mm TO 200 mm BOULDERS >200 mm	ANGULAR ROCK FRAGMENTS ROCKS > 0.75 m3 IN VOLUME

MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

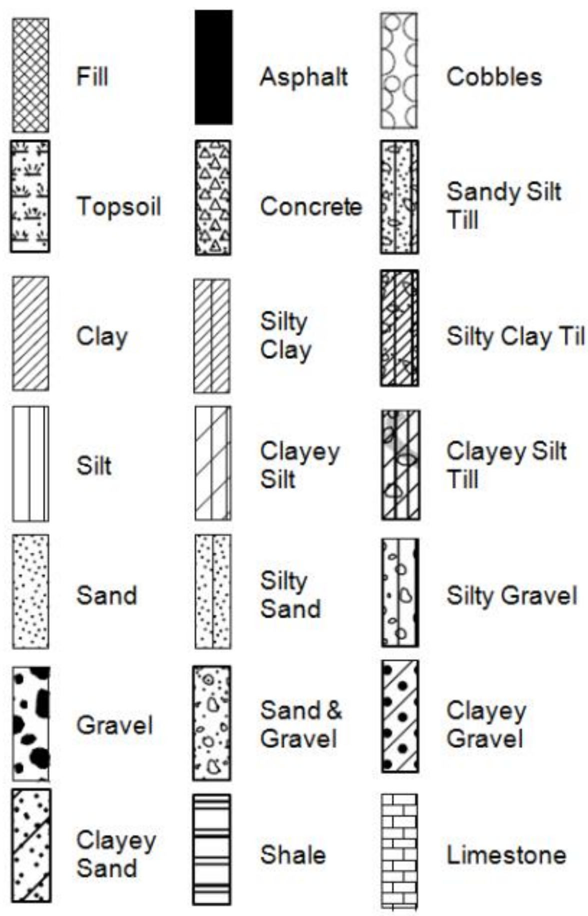
February 2022

1.5 Sample Type, Symbols and Abbreviations

The depth, type, and condition of samples are indicated on the logs by the following symbols or abbreviations:

Sample abbreviations:	Symbols:
GS: Grab Sample	
BK: Bulk Sample	
NR: No Recovery	
ST: Shelby Tube	
SS: Split Spoon	
Core: Core Samples	
FV: Field Vane	
PP: Pocket Penetrometer	
DCPT: Dynamic cone penetration test	

1.6 STRATA/Graphic Plot (Shall be Changed For Different Guidelines)



2. EXPLANATION OF ENVIROMENTAL SAMPLE

2.1 Contaminant Abbreviations

Contaminant Abbreviations	
BNAE	Base/neutral/acid extractables
BTEX	Benzene, toluene, ethylbenzene, xylenes
OCP	Organochlorine pesticides
MI	Metals and inorganics
PAH	Polycyclic aromatic hydrocarbons
PCB	Polychlorinated biphenyls
PHC	CCME petroleum hydrocarbons (fractions 1-4)
VOC	Volatile organic compounds (includes BTEX)
SO_4	Water Soluble Sulphate Content

2.2 Water Soluble Sulphate Concentration

The following table, from CSA Standard A23.1-14, indicates the requirements for concrete subjected to sulphate attack based upon the percentage of water-soluble sulphate as presented on the logs. CSA Standard A23.1-14 should be read in conjunction with the table.

Table 3 Requirements for Concrete Subjected to Sulphate Attack*

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	Performance requirements§,§§		
						Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, %		Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % †††
						At 6 months	At 12 months††	At 18 months‡‡
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS** ,HSb, HSLb*** or HSe	0.05	0.10	0.10
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS**, HSb, HSLb*** or HSe	0.05	0.10	0.10
S-3	Moderate (including seawater exposure*)	0.10–0.20	150–1500	0.20–0.60	MS, MSb, MSe, MSLb***, LH, LHb, HS**, HSb, HSLb*** or HSe	0.10		0.10

*For sea water exposure, also see Clause 4.1.1.5.
†In accordance with CSA A23.2-3B.
‡In accordance with CSA A23.2-2B.
§Where combinations of supplementary cementing materials and portland or blended hydraulic cements are to be used in the concrete mix design instead of the cementing materials listed, and provided they meet the performance requirements demonstrating equivalent performance against sulphate exposure, they shall be designated as MS equivalent (MSe) or HS equivalent (HSe) in the relevant sulphate exposures (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4).
**Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater. See Clause 4.1.1.6.3.
††The requirement for testing at 5 °C does not apply to MS, HS, MSb, HSb, and MSe and HSe combinations made without portland limestone cement.
‡‡ If the increase in expansion between 12 and 18 months exceeds 0.03%, the sulphate expansion at 24 months shall not exceed 0.10% in order for the cement to be deemed to have passed the sulphate resistance requirement.
§§For demonstrating equivalent performance, use the testing frequency in Table 1 of CSA A3004-A1 and see the applicable notes to Table A3 in A3001 with regard to re-establishing compliance if the composition of the cementing materials used to establish compliance changes.

***Where MSLb or HSLb cements are proposed for use, or where MSe or HSe combinations include Portland-limestone cement, they must also contain a minimum of 25% Type F fly ash or 40% slag or 15% metakaolin (meeting Type N pozzolan requirements) or a combination of 5% Type SF silica fume with 25% slag or a combination of 5% Type SF silica fume with 20% Type F fly ash. For some proposed MSLb, HSLb, and MSe or HSe combinations that include Portland-limestone cement, higher SCM replacement levels may be required to meet the A3004-C8 Procedure B expansion limits. Due to the 18-month test period, SCM replacements higher than the identified minimum levels should also be tested. In addition, sulphate resistance testing shall be run on MSLb and HSLb cement and MSe or HSe combinations that include Portland-limestone cement at both 23 °C and 5 °C as specified in the table.
†††If the expansion is greater than 0.05% at 6 months but less than 0.10% at 1 year, the cementing materials combination under test shall be considered to have passed.

2.3 Soil Corrosivity

The following table, from the Handbook of Corrosion Engineering (Roberge, 1999) indicates the corrosivity rating can be obtained from the soil resistivity, presented on the logs.

Table 4 Corrosivity Ratings Based on Soil Resistivity

Soil Resistivity (ohm-cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
<1,000	Extremely corrosive

3. HYDROGEOLOGICAL

The groundwater table is indicated by the equilibrium level of water in a standpipe installed in a test hole or test pit. This level is generally taken at least 24 hours after installation of the standpipe. The groundwater level is subject to seasonal variations and is usually highest in the spring. The symbol on the logs indicating the groundwater level is an inverted solid triangle (▼).

4. EXPLANATION OF ROCK

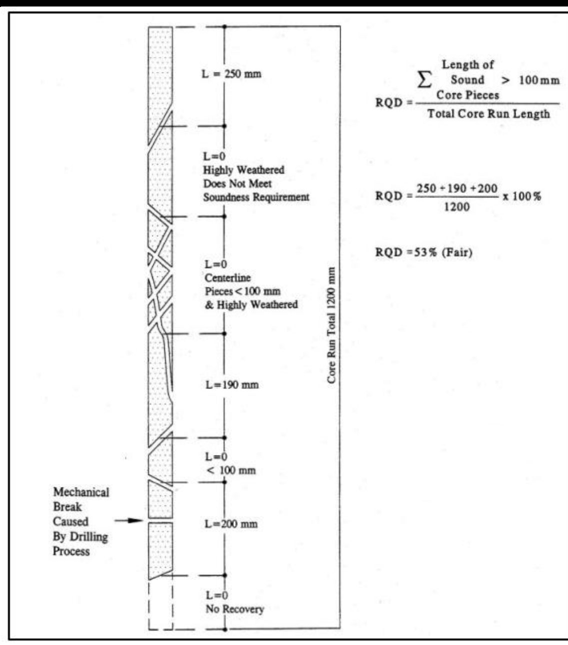
4.1 General Description and Terms

General Description of Geotechnical Unit including: Quantitative description including rock type (s), percentage of rock types, frequency and sizes of interbeds, colour, texture, weathering, strength and general joint spacing

Total Core Recovery (TCR): Total length of core recovered expressed as percentage of core run length.
Solid Core Recovery (SCR): Total length of solid full diameter core expressed as percentage of core run length.
Rock Quality Designation (RQD): Sum of lengths of solid core pieces longer than 100 mm expressed as percentage of core run length.
Fracture Index (FI): Number of fractures per meter of core.

4.2 Rock Quality Designation (RQD)

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



4.3 Classification of Strength

Grade	Description	Field identification	Approximate range of Uniaxial compression strength (MPa)
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0

R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

4.4 Classification of Weathering

Grade	Description	Field identification
W1	Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surface
W2	Slightly Weathered	Discolouration indicates weathering of rock material and discontinuity surface. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition
W3	Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
W4	Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
W5	Completely Weathered	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact. All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but soil has not been significantly transported.
W6	Residual Soil	Residual Soil

4.5 Type of discontinuity

Symbol	Description
F	Fault
J	Joint
Sh	Shear
Fo	Foliation
V	Vein
B	Bedding

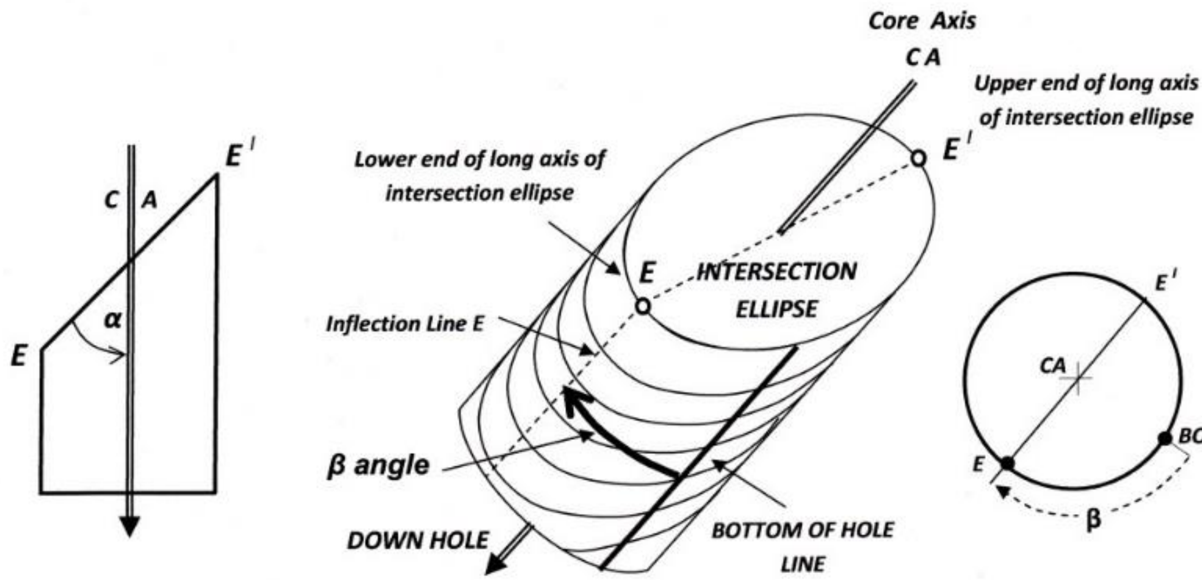
4.6 Spacing of discontinuity

Spacing Classification	Spacing width
Extremely close	<0.02m

Very close	0.02-0.06m
Close	0.06-0.2m
Moderately Close	0.2-0.6m
Wide	0.6-2.0m
Very Wide	2.0-6.0m
Extremely Wide	>6.0m

4.7 Joint Orientation

The orientation of a planar surface intersected by drill core can be defined by two angles called alpha (α) and beta (β). The definition of these angles is shown in the diagram below:



4.8 Inclination

Term	Inclination (degrees from the horizontal)
Sub-horizontal	0-5
Gently Inclined	6-15
Moderately Inclined	16-30
Steeply Inclined	31-60
Very Steeply Inclined	61-80
Sub-vertical	81-90

4.9 Stratification/foliation

Term	Spacing
Very Thickly Bedded	>2m
Thickly Bedded	600mm-2m
Medium Bedded	200mm-600mm
Thinly Bedded	60mm-200mm

Term	Spacing
Very Thinly Bedded	20mm-60mm
Laminated	6mm-20mm
Thinly Laminated	2mm-6mm
Fissile	<2mm

4.10 Grain Size

Term	Size
Very Coarse Grained	>60 mm
Coarse Grained	2mm-60mm
Medium Grained	60 microns – 2mm
Fine Grained	2 microns – 60 microns
Very Fine Grained	<2 microns

4.11 Aperture of open discontinuity

Symbol	Aperture Opening	Description	
VT	<0.1 mm	Very tight	Closed Features
T	0.1-0.25mm	Tight	
PO	0.25-0.5mm	Partly open	
O	0.5-2.5mm	Open	Gapped Features
MW	2.5-10mm	Moderately open	
W	>10mm	Wide	
VW	1-10cm	Very wide	Open Features
EW	10-100cm	Extremely wide	
C	>1m	Cavernous	

4.12 Width of filled discontinuity

Symbol	Width	Description
W	12.5-50mm	Wide
MW	2.5-12.5mm	Moderately Wide
N	1.25-2.5mm	Narrow
VN	<1.25mm	Very Narrow
T	0mm	Tight

4.13 Roughness of discontinuity

Symbol	Description
Slk	Slickenside (surface has smooth, glassy finish with visual evidence of striations)
S	Smooth (surface appears smooth and feels so to the touch)
SR	Slightly rough (asperities on the discontinuity surfaces are distinguishable and can be felt)
R	Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive)

Symbol	Description
VR	Very rough (near-vertical steps and ridges occur on the discontinuity surface)

4.14 Shape of discontinuity

Symbol	Description
Pl	Planar
St	Stepped
Un	Undulating
Ir	Irregular

4.15 Filling amount

Symbol	Description
Su	Surface Stain
Sp	Spotty
Pa	Partially Filled
Fi	Filled
No	None

4.16 Filling Type

Symbol	Term	Hard/Soft
Ab	Albite	Hard
Ah	Anhydrite	Hard
Bt	Biotite	Soft
Bn	Bornite	Hard
Ca	Calcite	Hard
Cb	Carbonate	Hard
Ch	Chlorite	Soft
Cpy	Chalcopyrite	Hard
Cy	Clay	Soft
Do	Dolomite	Hard
Ep	Epidote	Hard
Fd	Feldspar	Hard
FeOx	Iron Oxide	Hard
Go	Gouge	Soft
Gr	Graphite	Soft
Gy	Gypsum	Soft
He	Hematite	Hard
Ka	Kaolinite	Soft
Kf	K-feldspar	Hard

Symbol	Term	Hard/Soft
Lm	Limonite/FeOx	Soft
Ms	Muscovite	Soft
Mt	Magnetite	Hard
Py	Pyrite	Hard
Qz	Quartz	Hard
Rb	Rubble	Hard
Sa	Sand	Hard
Se	Sericite/Illite	Soft
Si	Silt	Hard
Sm	Smectite	Soft
Su	Sulphide	Hard
Ta	Talc	Soft
UH	Unknown Hard	Hard
US	Unknown Soft	Soft
OTH - see comments		

PROJECT: NEWPCC Primary Scum Building	CLIENT: City of Winnipeg	TESTHOLE NO: TH24-01
LOCATION: UTM: 14U, 5535035 m N, 635596.323 m E		PROJECT NO.: 60709390
CONTRACTOR: Paddock Drilling	METHOD: Solid Stem Auger	ELEVATION (m): 230.89

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

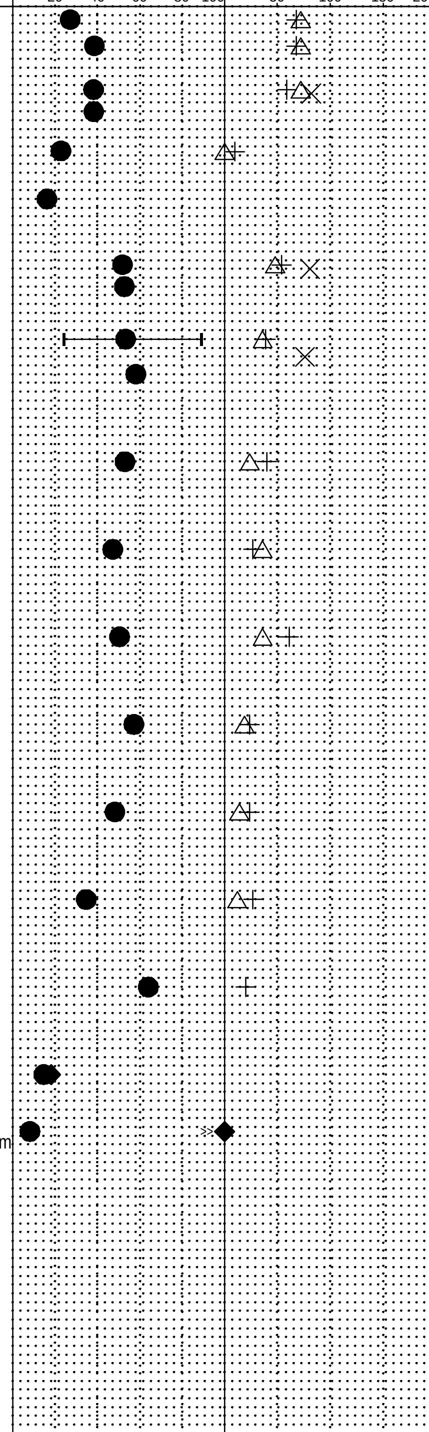
BACKFILL TYPE: BENTONITE, GRAVEL, SLOUGH, GROUT, CUTTINGS, SAND

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
0	CONC	Concrete - 0.15 m thick	G1, G2		
1	FILL	FILL - Clay - Black, stiff, moist - High plasticity	G3, T4		T4: Bulk Unit Weight (kN/m^3) = 18.4
2	ML	very soft to soft tan SILT (ML) - Moist - Low plasticity	G5, T6		
4	CH	firm to stiff grey fat CLAY (CH) - Moist - High plasticity - soft below 12.50 m	G7, T8, G9, T10, G11, G12, G13, G14, G15, G16, G17		T8: Bulk Unit Weight (kN/m^3) = 16.8 G9: LL = 89, PL = 24; Gravel (%) = 0.0, Sand (%) = 0.5, Silt (%) = 22.4, Clay (%) = 77.0 T10: Bulk Unit Weight (kN/m^3) = 16.9
18	CL-ML	compact to very dense tan sandy silty CLAY (CL-ML) TILL - moist, compact, no plasticity	S18 S19	18 50/ 152mm	

END OF TESTHOLE
- Testhole was terminated at a depth of 19.66 m at auger refusal on suspected bedrock
- Groundwater level was observed at a final depth of 12.80 m upon completion of drilling
- Soil sloughing was observed in the SILT (ML) at a depth of 2.29 m
- Testhole backfilled with soil cuttings and bentoite

PENETRATION TESTS: Becker; Dynamic Cone; SPT (Standard Pen Test) (Blows/300mm) 0 20 40 60 80 100; Total Unit Wt (kN/m^3) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: Torvane; QU/2; Lab Vane; Pocket Pen.; Field Vane (kPa) 50 100 150 200



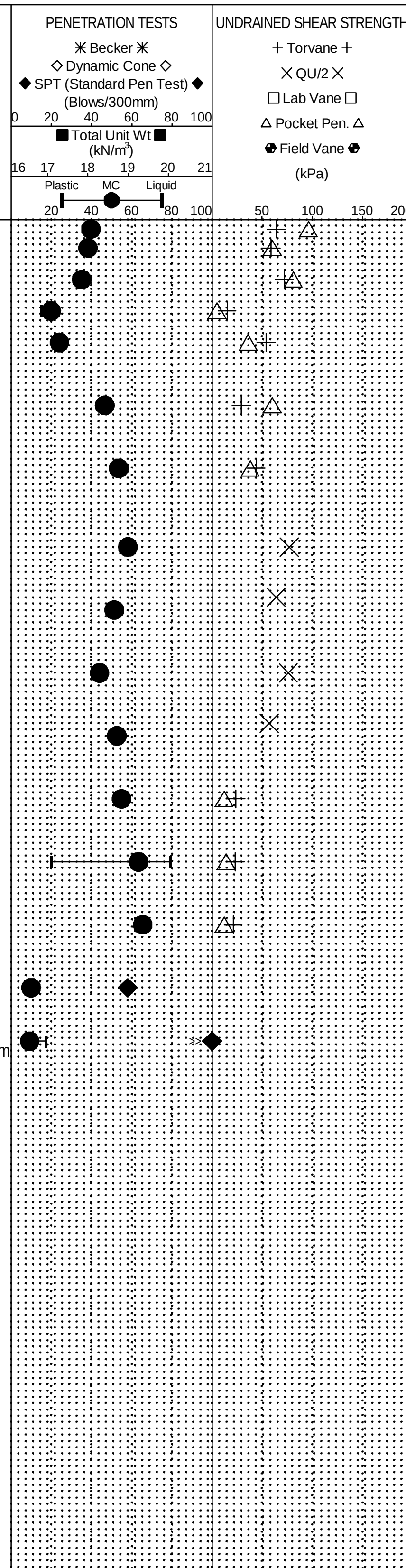
LOGGED BY: GA	COMPLETION DEPTH: 19.66 m
REVIEWED BY: GL	COMPLETION DATE: 24-7-11
PROJECT ENGINEER:	Page 1 of 1

AECOM

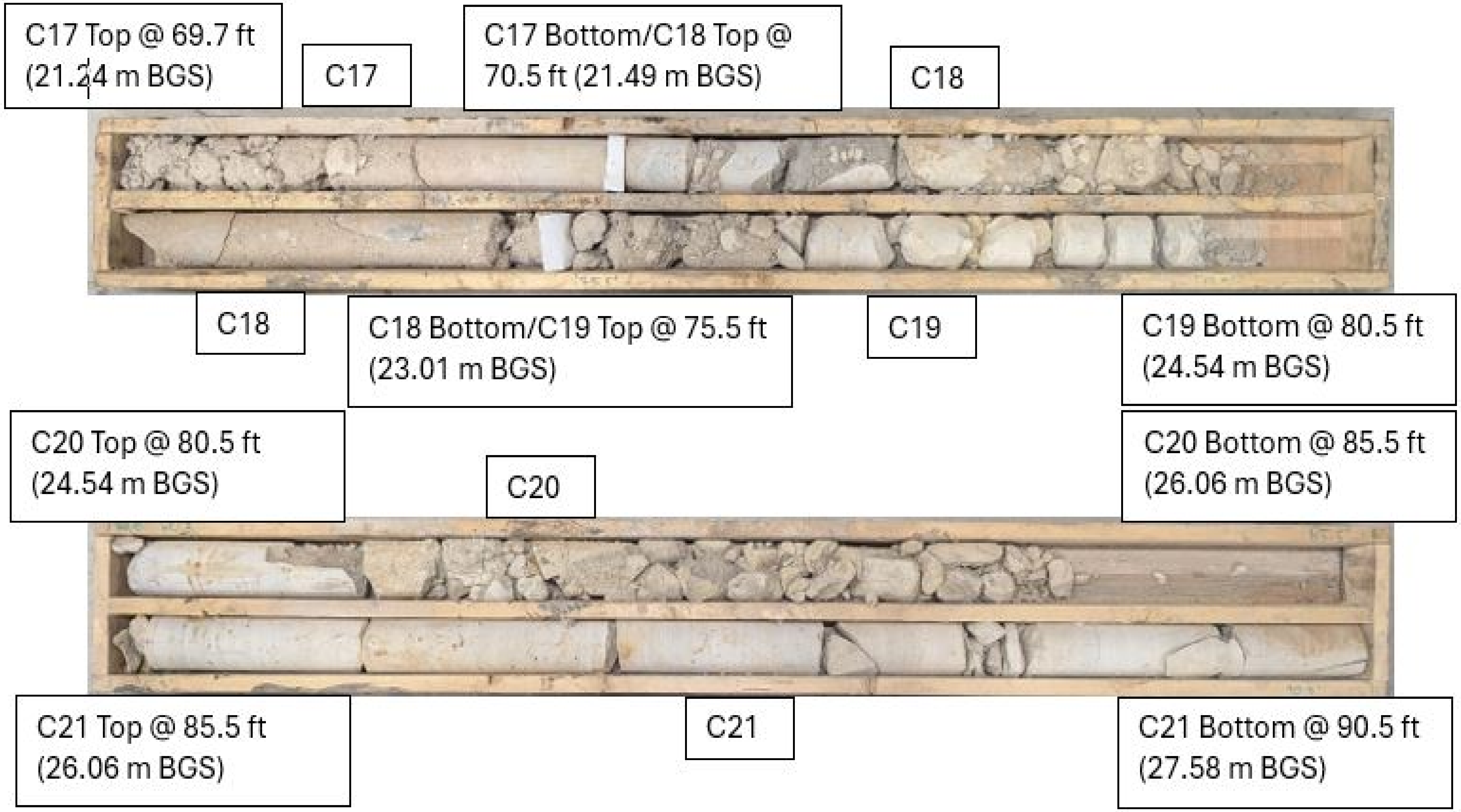
PROJECT: NEWPCC Primary Scum Building	CLIENT: City of Winnipeg	TESTHOLE NO: TH24-02
LOCATION: UTM: 14U, 5535019.81 m N, 635588.986 m E		PROJECT NO.: 60709390
CONTRACTOR: Paddock Drilling	METHOD: Solid Stem Auger	ELEVATION (m): 230.80

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

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
0	CONC	Concrete - 0.15 m thick	G1		
1	FILL	FILL - Clay - Black, stiff, moist - High plasticity	G2, G3		
2	ML	very soft to soft tan SILT (ML) - Moist - Low plasticity	G4		G4: LL = 23, PL = 15; Gravel (%) = 0.4, Sand (%) = 5.4, Silt (%) = 81.0, Clay (%) = 13.1
3	CL	very soft to firm grey lean CLAY (CL) - Moist - Low plasticity	G5		
6	CH	soft to firm grey fat CLAY (CH) - Moist - High plasticity	G6, G7		
8			T8		T8: Bulk Unit Weight (kN/m^3) = 16.9
9.5			T9		T9: Bulk Unit Weight (kN/m^3) = 16.7
11			T10		T10: Bulk Unit Weight (kN/m^3) = 17.6
12.5			T11		T11: Bulk Unit Weight (kN/m^3) = 17.1
14		- soft below 14.02 m	G12		
15.5			G13		G13: LL = 79, PL = 20; Gravel (%) = 1.0, Sand (%) = 4.6, Silt (%) = 22.7, Clay (%) = 71.6
17			G14		
18.5	CL-ML	dense to very dense tan sandy silty CLAY (CL-ML) TILL - moist, compact - low plasticity	S15	58	
20			S16	50/ 152mm	S16: LL = 17, PL = 10; Gravel (%) = 4.8, Sand (%) = 33.9, Silt (%) = 46.3, Clay (%) = 15.0
21.5			C17, C18		
24		- water no longer returning to surface during coring	C19		
25	BR	Mottled Dolomitic Limestone (Red River Formation, Selkirk Member)	C20, C21		
28		END OF TESTHOLE - Testhole terminated at a depth of 27.58 m in bedrock - Moderate groundwater seepage was observed at a depth of 2.29 m - Groundwater level was observed at a depth of 6.71 m upon completion of drilling - Soil sloughing was ob			



LOGGED BY: GA	COMPLETION DEPTH: 18.90 m
REVIEWED BY: GL	COMPLETION DATE: 24-7-10
PROJECT ENGINEER:	Page 1 of 1


C17 Top @ 69.7 ft (21.24 m BGS)
C17
C17 Bottom/C18 Top @ 70.5 ft (21.49 m BGS)
C18
C18
C18 Bottom/C19 Top @ 75.5 ft (23.01 m BGS)
C19
C19 Bottom @ 80.5 ft (24.54 m BGS)
C20 Top @ 80.5 ft (24.54 m BGS)
C20
C20 Bottom @ 85.5 ft (26.06 m BGS)
C21 Top @ 85.5 ft (26.06 m BGS)
C21
C21 Bottom @ 90.5 ft (27.58 m BGS)

Appendix D

Laboratory Results

AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Memorandum

To	Colton Wooster	Page	1
CC			
Subject	NEWPCC Primary Scum Building – Test Results		
From	German Leal		
Date	August 7, 2024	Project Number	60661262

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Twenty-four (24) Moisture Content Determination Test.
- Four (4) Atterberg Limits (3 Points) Test.
- Four (4) Grain Size Distribution (Hydrometer method) Test.
- Eight (8) Unconfined Compressive Strength Test.

If you have any questions, please contact the undersigned.

Prepared by:

Boughton, Lee

Digitally signed by Boughton, Lee
DN: cn=Boughton, Lee, ou=CAWPG1, email=Lee.Boughton@aecom.com
Date: 2024.08.07 15:24:25 -05'00'

Lee Boughton
Laboratory Manager

Reviewed by:

Leal, German

Digitally signed by Leal, German
DN: cn=Leal, German, ou=CAWPG1, email=German.Leal@aecom.com
Date: 2024.08.07 15:53:46 -05'00'

German Leal, M.Eng., P.Eng.
Discipline Lead, Geotechnical

Att.

AECOM

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	AECOM
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	Winnipeg, Manitoba	Sample Date:	24-Jul-24
Sample Depth :	Varies	Lab Technician:	LBoughton
Sample Number:	Varies	Date Tested:	24-Jul-24

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	Moisture Content (%)
TH24-01	G1	0.15 - 0.30 m	**27.1%**
	G2	0.61 - 0.76 m	**38.5%**
	G3	1.37 - 1.52 m	**38.1%**
	T4	1.52 - 2.13 m	**38.2%**
	G5	2.90 - 3.05 m	**22.7%**
	T6	3.05 - 3.66 m	**16.1%**
	G7	4.42 - 4.57 m	**51.8%**
	T8	4.57 - 5.18 m	**52.7%**
	G9	5.49 - 6.10 m	**53.3%**
	T10	6.10 - 6.71 m	**58.1%**
	G11	7.62 - 8.23 m	**53.0%**
	G12	9.14 - 9.75 m	**47.2%**
	G13	10.67 - 11.28 m	**50.4%**
	G14	12.19 - 12.80 m	**57.2%**
	G15	13.72 - 14.33 m	**48.2%**
	G16	15.24 - 15.85 m	**34.7%**
	G17	16.76 - 17.37 m	**64.0%**
	S18	18.29 - 18.90 m	**14.6%**
	S19	19.51 - 19.66 m	**8.1%**
			-
TH24-02	G1	0.15 - 0.30 m	**39.7%**
	G2	0.61 - 0.76 m	**38.2%**
	G3	1.37 - 1.52 m	**35.1%**
	G4	2.13 - 2.29 m	**19.9%**
	G5	2.90 - 3.05 m	**24.0%**
	G6	4.42 - 4.57 m	**46.6%**
	G7	5.94 - 6.10 m	**53.5%**
	T8	7.62 - 8.23 m	**58.1%**
	T9	9.14 - 9.75 m	**51.3%**
	T10	10.67 - 11.28 m	**44.0%**
	T11	12.19 - 12.80 m	**52.6%**
	G12	13.72 - 14.33 m	**54.9%**
	G13	15.24 - 15.85 m	**63.4%**
	G14	16.76 - 17.37 m	**65.5%**
	S15	18.29 - 18.90 m	**9.9%**
	S16	19.81 - 19.96 m	**9.2%**

Location	Sample	Depth (m)	Moisture Content (%)

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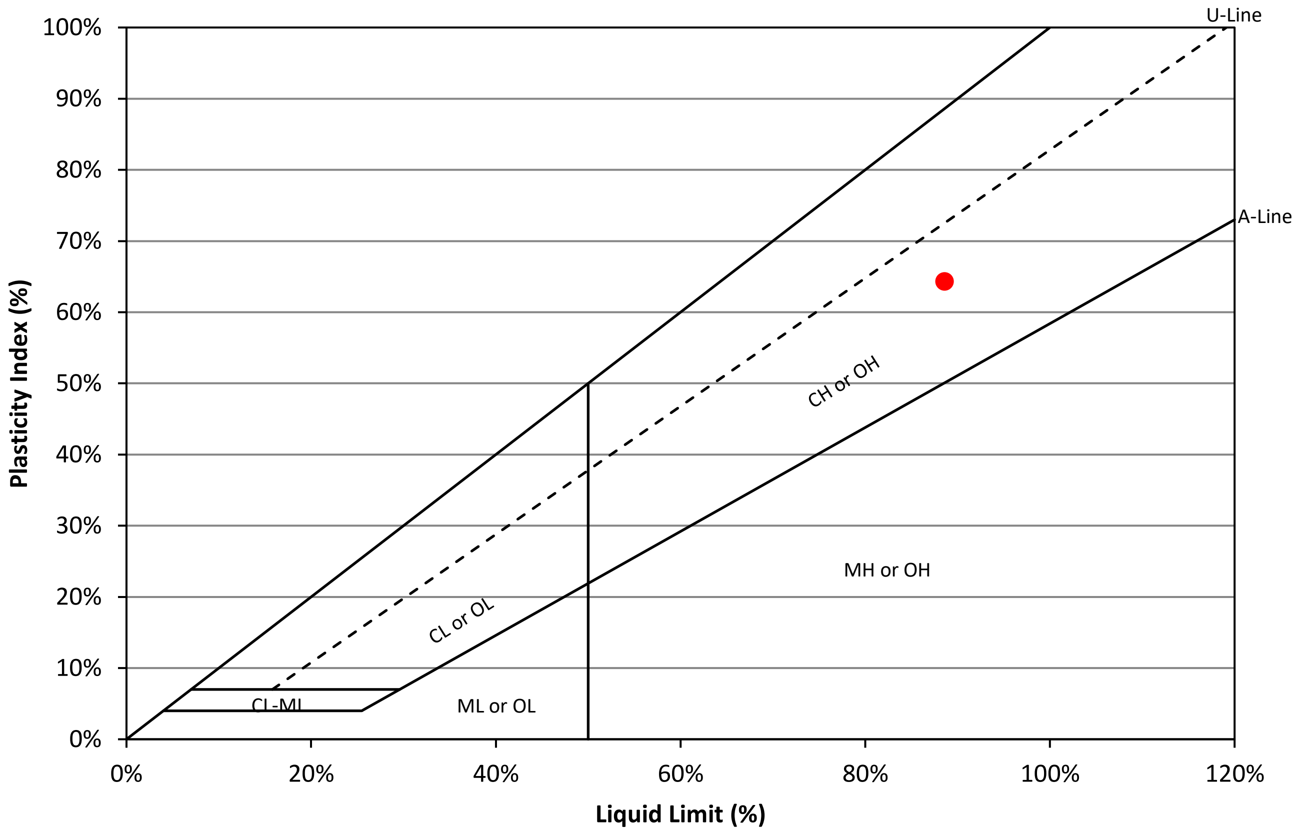
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	July 24, 2024
Sample Depth:	5.49 - 6.10 m	Lab Technician:	LBoughton
Sample Number:	G9	Date Tested:	August 6, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	30	23
Wet Sample (g)	11.9	12.4	12.8
Dry Sample (g)	6.5	6.6	6.8
Water Content (%)	82.9%	86.8%	89.5%

Plastic Limit		
Trial	1	2
Wet Sample (g)	5.0	6.2
Dry Sample (g)	4.0	5.0
Water Content (%)	24.3%	24.3%



Liquid Limit:	89	Plastic Limit:	24	Plasticity Index:	65

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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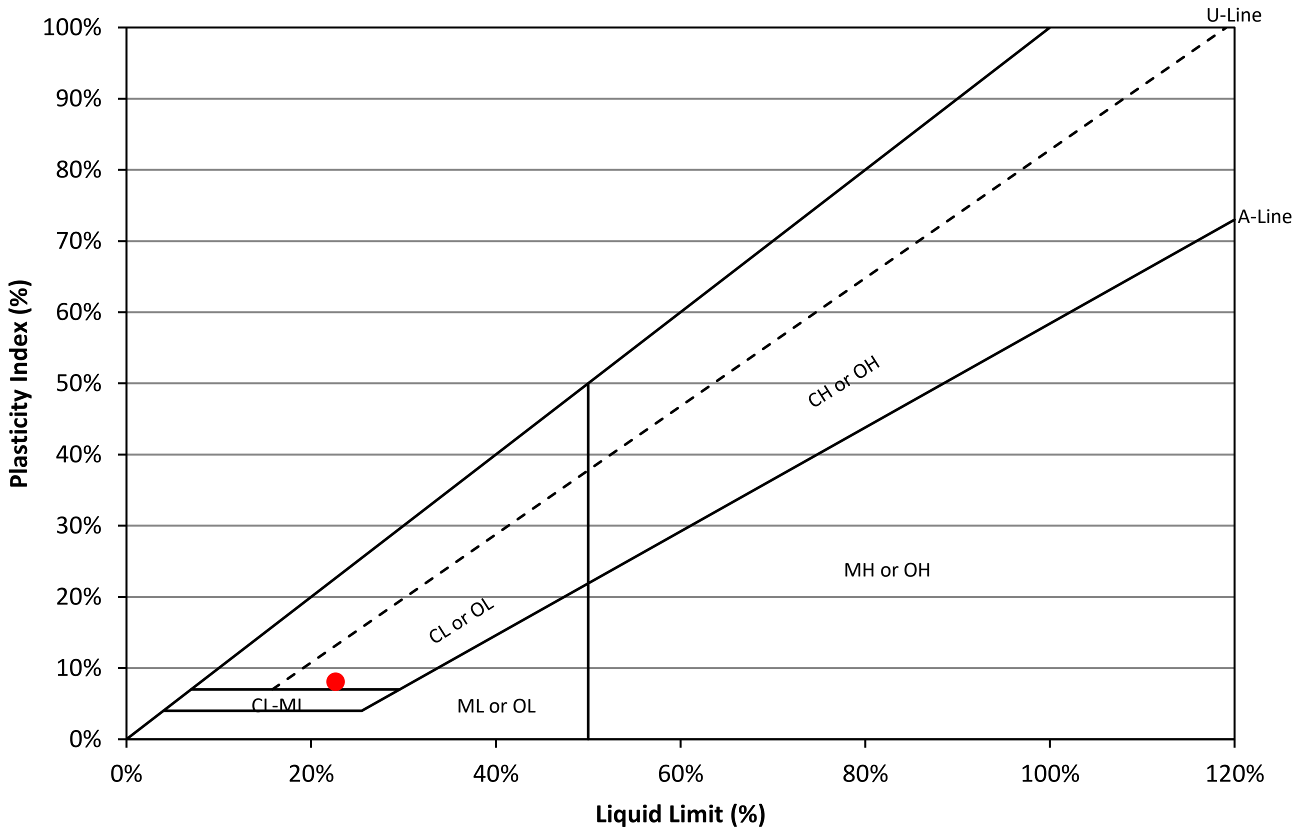
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	July 24, 2024
Sample Depth:	2.13 - 2.29 m	Lab Technician:	LBoughton
Sample Number:	G4	Date Tested:	August 6, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	31	24	20
Wet Sample (g)	14.8	12.6	13.3
Dry Sample (g)	12.1	10.2	10.8
Water Content (%)	22.2%	22.6%	23.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.9	5.5
Dry Sample (g)	6.9	4.8
Water Content (%)	14.7%	14.4%



Liquid Limit:	**23**	**Plastic Limit:**	**15**	**Plasticity Index:**	**8**

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Laboratory Manager

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Geotechnical Discipline Lead

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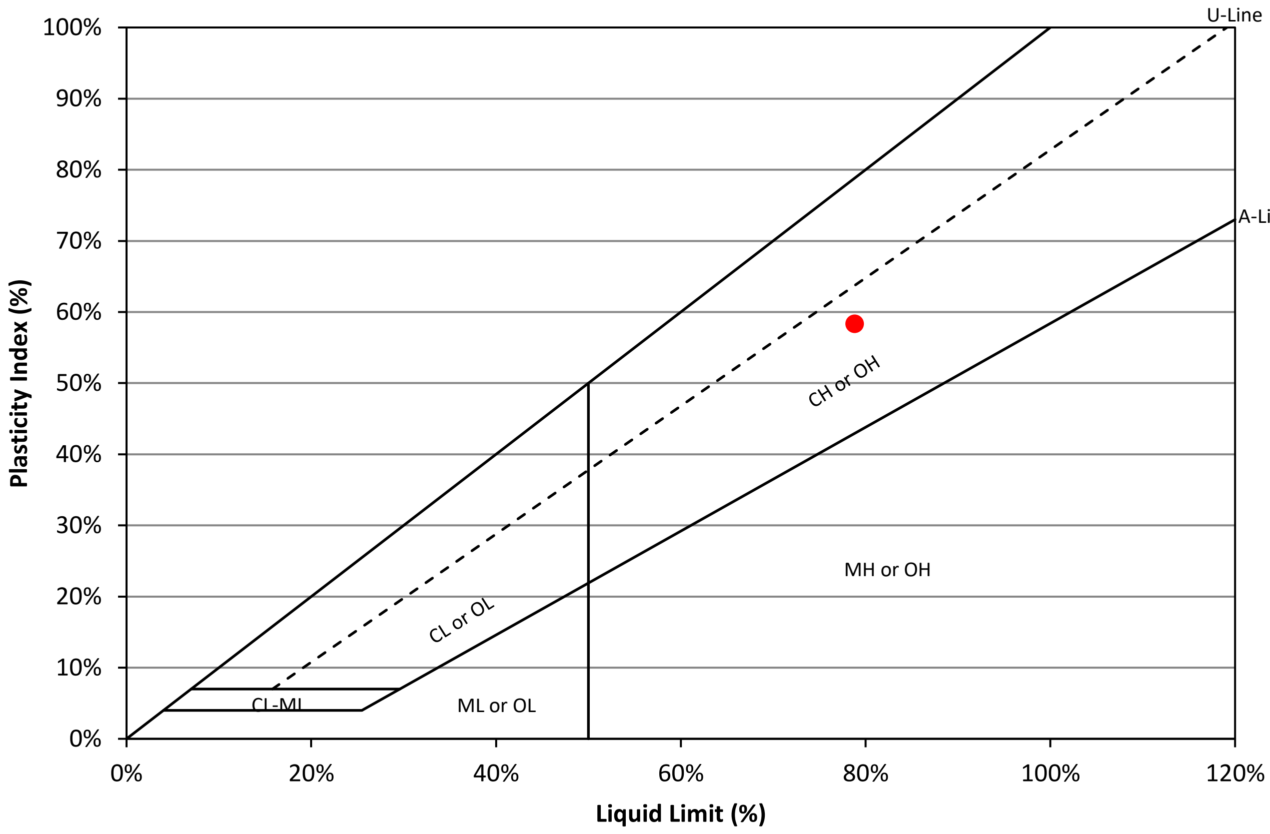
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	July 24, 2024
Sample Depth:	15.24 - 15.85 m	Lab Technician:	LBoughton
Sample Number:	S13	Date Tested:	August 6, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	25	21
Wet Sample (g)	14.1	11.7	11.2
Dry Sample (g)	8.1	6.6	6.1
Water Content (%)	73.1%	78.1%	82.4%

Plastic Limit		
Trial	1	2
Wet Sample (g)	4.9	5.5
Dry Sample (g)	4.1	4.6
Water Content (%)	20.4%	20.5%



Liquid Limit:	79	Plastic Limit:	20	Plasticity Index:	59

Reviewed by: Lee Boughton
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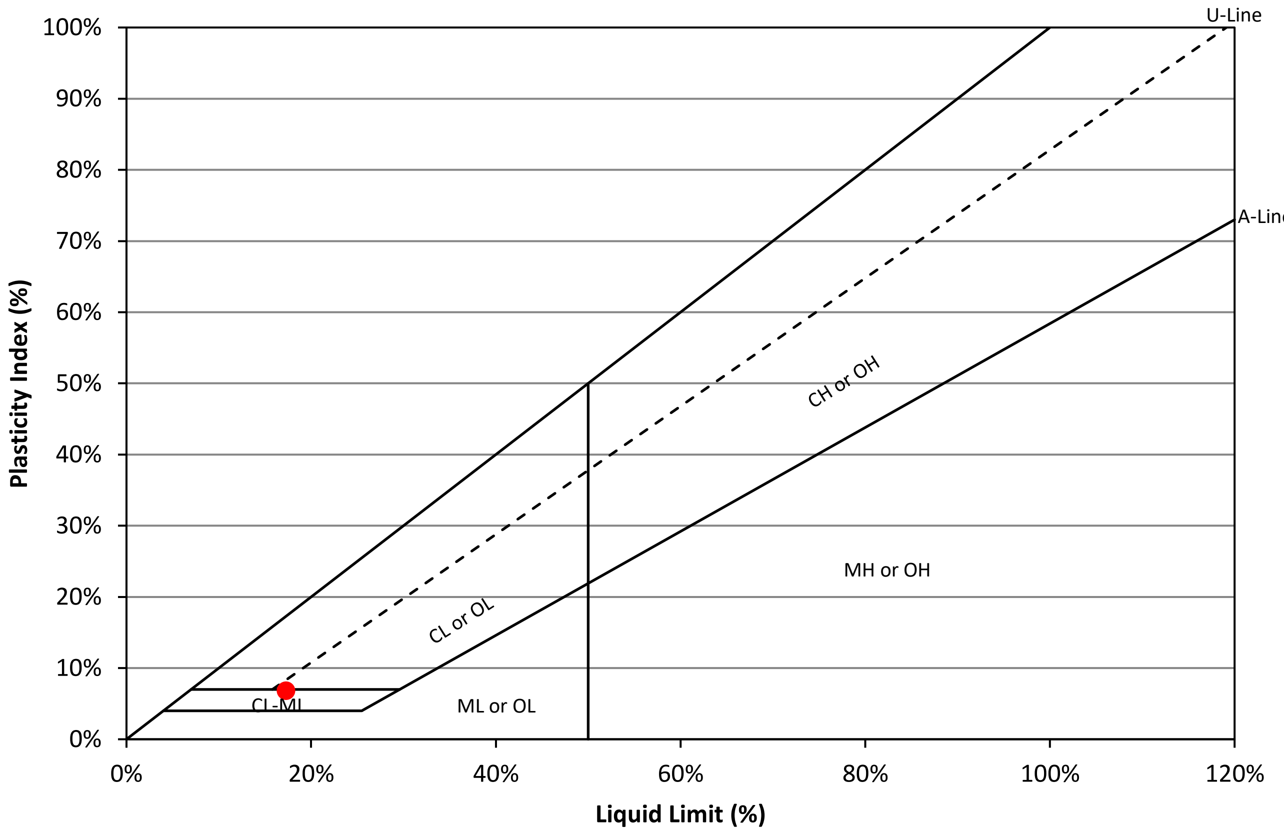
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	July 24, 2024
Sample Depth:	19.81 - 19.96 m	Lab Technician:	LBoughton
Sample Number:	S16	Date Tested:	August 6, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	33	23	18
Wet Sample (g)	14.7	13.7	12.5
Dry Sample (g)	12.7	11.6	10.6
Water Content (%)	16.3%	17.7%	18.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	5.7	6.0
Dry Sample (g)	5.1	5.5
Water Content (%)	10.8%	10.2%



Liquid Limit:	**17**	**Plastic Limit:**	**10**	**Plasticity Index:**	**7**

Reviewed by: Lee Boughton
Laboratory Manager

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Geotechnical Discipline Lead

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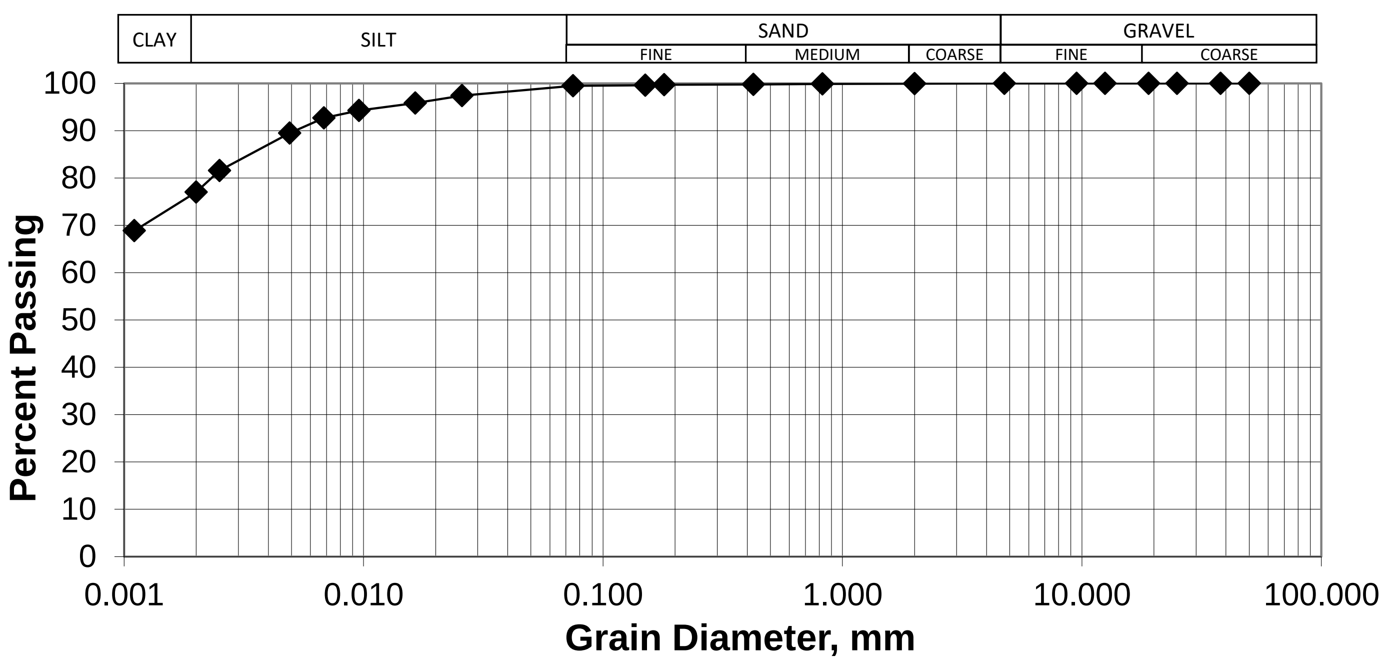
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	24-Jul-24
Sample Depth :	5.49 - 6.10 m	Lab Technician:	LBoughton
Sample Number:	G9	Date Tested:	31-Jul-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	99.5
38.0	100.0	2.00	100.0	0.0258	97.5
25.0	100.0	0.825	99.9	0.0164	95.9
19.0	100.0	0.425	99.8	0.0096	94.3
12.5	100.0	0.18	99.7	0.0068	92.7
9.5	100.0	0.15	99.6	0.0049	89.5
4.75	100.0	0.075	99.5	0.0025	81.6
				0.0020	77.0
				0.0011	68.9

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**22.4%**
Sand	**0.5%**	**Clay**	**77.0%**

Reviewed by: Lee Boughton, Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng., Geotechnical Discipline Lead

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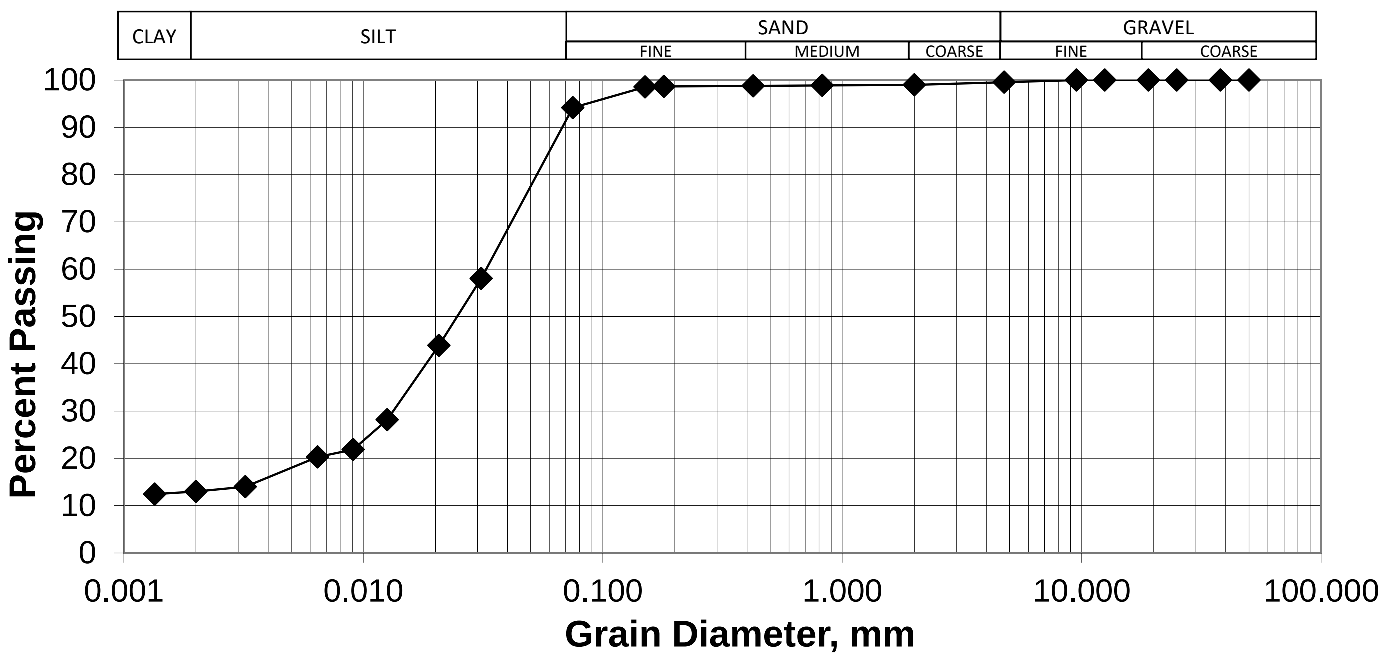
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	24-Jul-24
Sample Depth :	2.13 - 2.29 m	Lab Technician:	LBoughton
Sample Number:	G4	Date Tested:	31-Jul-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	99.6	0.0750	94.2
38.0	100.0	2.00	99.0	0.0310	58.6
25.0	100.0	0.825	98.9	0.0207	44.3
19.0	100.0	0.425	98.8	0.0126	28.5
12.5	100.0	0.18	98.7	0.0091	22.1
9.5	100.0	0.15	98.5	0.0064	20.5
4.75	99.6	0.075	94.2	0.0032	14.2
				0.0020	13.1
				0.0013	12.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.4%**	**Silt**	**81.0%**
Sand	**5.4%**	**Clay**	**13.1%**

Reviewed by: Lee Boughton, Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng., Geotechnical Discipline Lead

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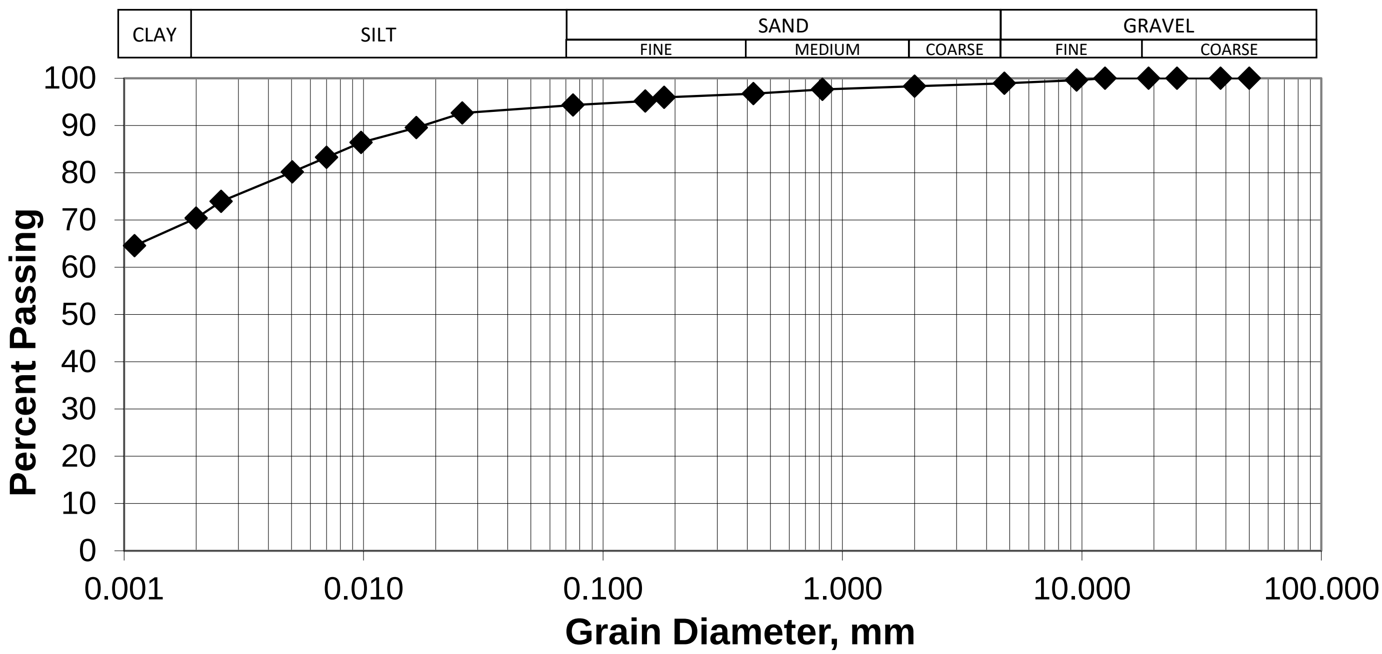
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	24-Jul-24
Sample Depth :	15.24 - 15.85 m	Lab Technician:	LBoughton
Sample Number:	S13	Date Tested:	31-Jul-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	99.0	0.0750	94.3
38.0	100.0	2.00	98.3	0.0258	94.2
25.0	100.0	0.825	97.7	0.0166	91.0
19.0	100.0	0.425	96.8	0.0098	87.9
12.5	100.0	0.18	96.0	0.0070	84.7
9.5	99.6	0.15	95.2	0.0050	81.5
4.75	99.0	0.075	94.3	0.0025	75.2
				0.0020	71.6
				0.0011	65.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**1.0%**	**Silt**	**22.7%**
Sand	**4.6%**	**Clay**	**71.6%**

Reviewed by: Lee Boughton, Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng., Geotechnical Discipline Lead

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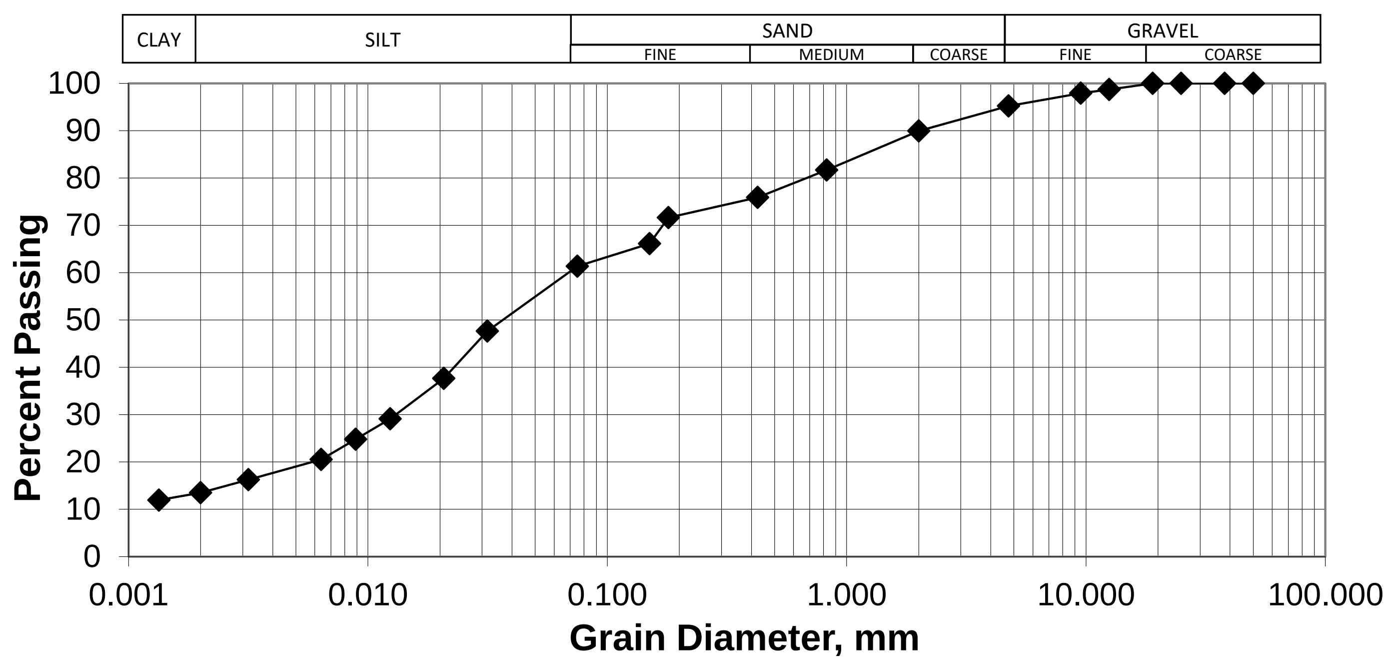
Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Supplier/Location:	Winnipeg, Manitoba
Client:	City of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	24-Jul-24
Sample Depth :	19.81 - 19.96 m	Lab Technician:	LBoughton
Sample Number:	S16	Date Tested:	31-Jul-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	95.2	0.0750	61.4
38.0	100.0	2.00	89.9	0.0315	53.0
25.0	100.0	0.825	81.7	0.0208	41.9
19.0	100.0	0.425	75.9	0.0124	32.4
12.5	98.7	0.18	71.6	0.0089	27.6
9.5	97.9	0.15	66.1	0.0064	22.8
4.75	95.2	0.075	61.4	0.0032	18.1
				0.0020	15.0
				0.0013	13.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**4.8%**	**Silt**	**46.3%**
Sand	**33.9%**	**Clay**	**15.0%**

Reviewed by: Lee Boughton, Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng., Geotechnical Discipline Lead

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Phone: 204 477 5381

Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	1.52 - 2.13 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	July 31, 2024
Sample Number:	T4	Tested By:	LBoughton

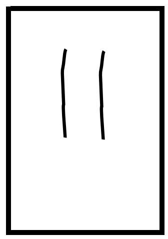
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, High Plasticity, slickensided

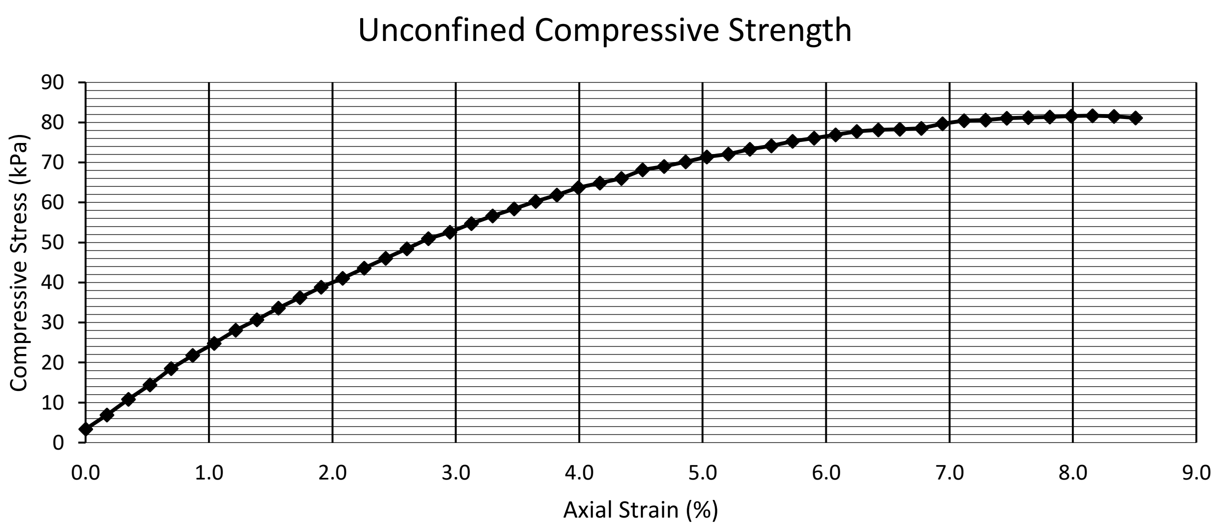
Average Diameter (cm):	7.20
Average Length (cm):	14.40
Length/Diameter Ratio:	2.00
Moisture content (%):	38.2
Bulk Density (g/cm³):	1.875
Bulk Unit Weight (kN/m³):	18.4
Bulk Unit Weight (pcf):	117.1
Dry Unit Weight (kN/m³):	13.30

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	63.8
Pocket Pen.	Undrained Shear Strength (kPa)	51.9

UCS	Unconfined compressive strength (kPa)	81.70	Undrained Shear Strength (kPa)	40.85
	Unconfined compressive strength (ksf)	1.706	Undrained Shear Strength (ksf)	0.853
	Avg. Rate of Strain to Failure (%/min):	1.04	Strain at Failure (%):	8.16



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	3.05 - 3.66 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	July 31, 2024
Sample Number:	T6	Tested By:	LBoughton

Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	SAND - brown, , moist, some silt, trace gravel, trace clay, Non Plastic, homogeneous

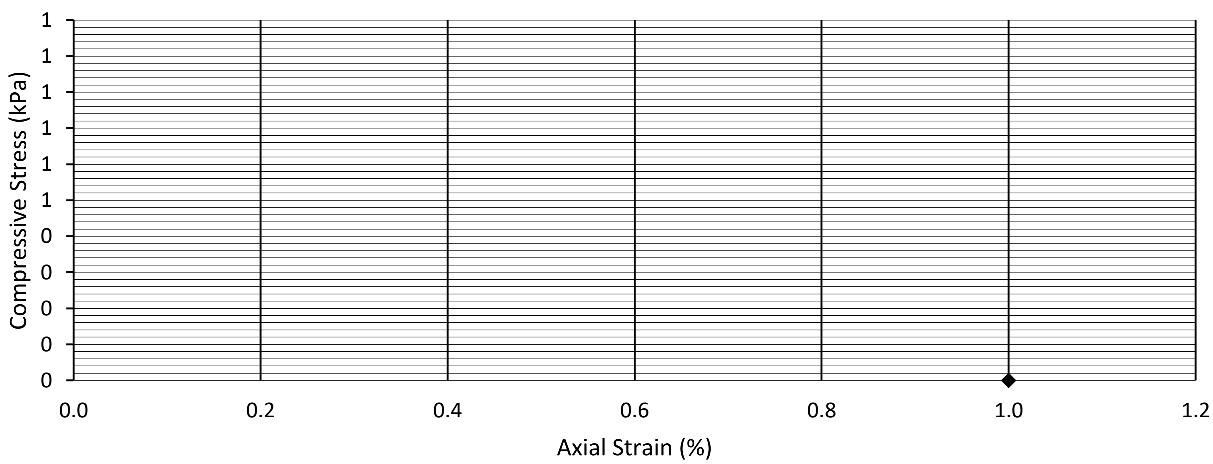
Average Diameter (cm):	
Average Length (cm):	
Length/Diameter Ratio:	
Moisture content (%):	16.1
Bulk Density (g/cm³):	
Bulk Unit Weight (kN/m³):	
Bulk Unit Weight (pcf):	
Dry Unit Weight (kN/m³):	

FAILURE SKETCH

Torvane	Undrained Shear Strength (kPa)	
Pocket Pen.	Undrained Shear Strength (kPa)	

UCS	Unconfined compressive strength (kPa)		Undrained Shear Strength (kPa)	
	Unconfined compressive strength (ksf)		Undrained Shear Strength (ksf)	
	Avg. Rate of Strain to Failure (%/min):		Strain at Failure (%):	#######

Unconfined Compressive Strength



Comments:
Sample to soft for unconfined compressive test

Reviewed by:	Lee Boughton Laboratory Manager	Approved by:	German Leal, M.Eng., P.Eng. Geotechnical Discipline Lead

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Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	4.57 - 5.18 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	July 31, 2024
Sample Number:	T8	Tested By:	LBoughton

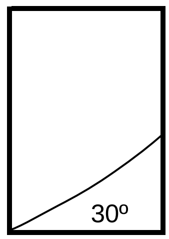
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, High Plasticity, slickensided

Average Diameter (cm):	7.20
Average Length (cm):	15.00
Length/Diameter Ratio:	2.08
Moisture content (%):	52.7
Bulk Density (g/cm³):	1.712
Bulk Unit Weight (kN/m³):	16.8
Bulk Unit Weight (pcf):	106.9
Dry Unit Weight (kN/m³):	11.00

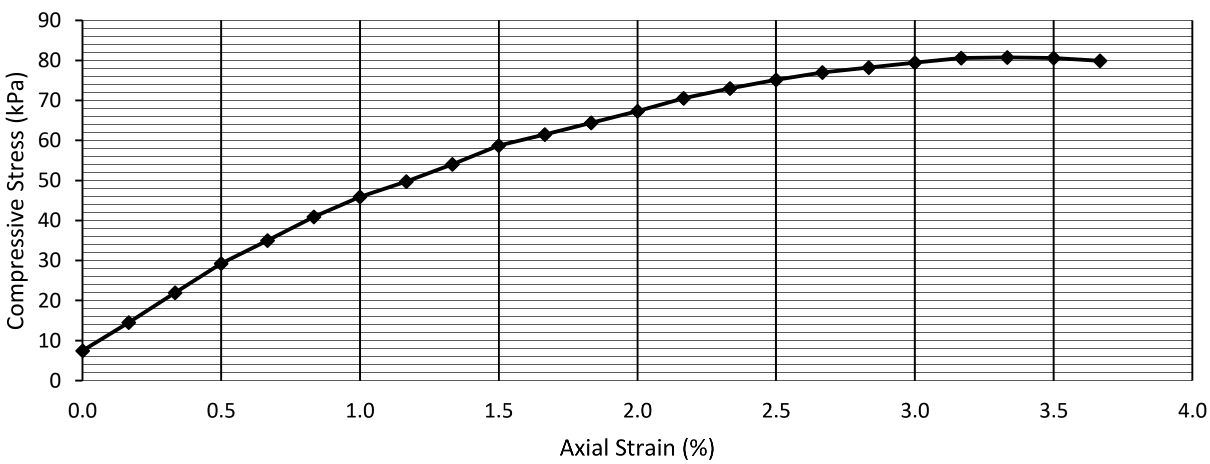
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	57.9
Pocket Pen.	Undrained Shear Strength (kPa)	55.9

UCS	Unconfined compressive strength (kPa)	80.75	Undrained Shear Strength (kPa)	40.37
	Unconfined compressive strength (ksf)	1.686	Undrained Shear Strength (ksf)	0.843
	Avg. Rate of Strain to Failure (%/min):	1.00	Strain at Failure (%):	3.33

Unconfined Compressive Strength



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	6.10 - 6.71 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	July 31, 2024
Sample Number:	T10	Tested By:	LBoughton

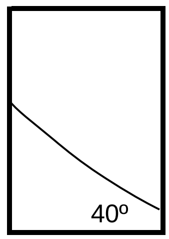
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, High Plasticity, slickensided

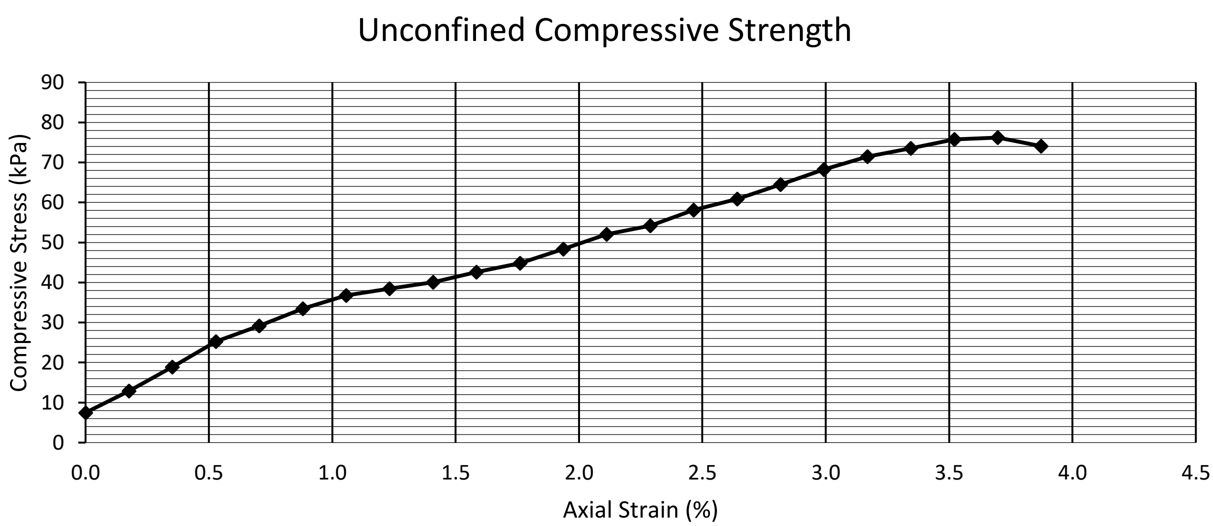
Average Diameter (cm):	7.20
Average Length (cm):	14.20
Length/Diameter Ratio:	1.97
Moisture content (%):	58.1
Bulk Density (g/cm³):	1.721
Bulk Unit Weight (kN/m³):	16.9
Bulk Unit Weight (pcf):	107.5
Dry Unit Weight (kN/m³):	10.68

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	66.7
Pocket Pen.	Undrained Shear Strength (kPa)	63.8

UCS	Unconfined compressive strength (kPa)	76.21	Undrained Shear Strength (kPa)	38.10
	Unconfined compressive strength (ksf)	1.592	Undrained Shear Strength (ksf)	0.796
	Avg. Rate of Strain to Failure (%/min):	1.06	Strain at Failure (%):	3.70



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Phone: 204 477 5381

Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	7.62 - 8.23 m	Submitted By:	GAcurin
Sample Location:	TH24-02	Date Tested:	July 31, 2024
Sample Number:	T8	Tested By:	LBoughton

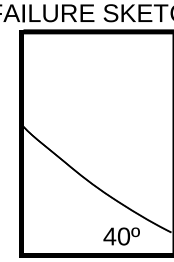
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, firm, moist, silty, High Plasticity, homogeneous

Average Diameter (cm):	7.10
Average Length (cm):	15.10
Length/Diameter Ratio:	2.13
Moisture content (%):	58.1
Bulk Density (g/cm³):	1.723
Bulk Unit Weight (kN/m³):	16.9
Bulk Unit Weight (pcf):	107.6
Dry Unit Weight (kN/m³):	10.69

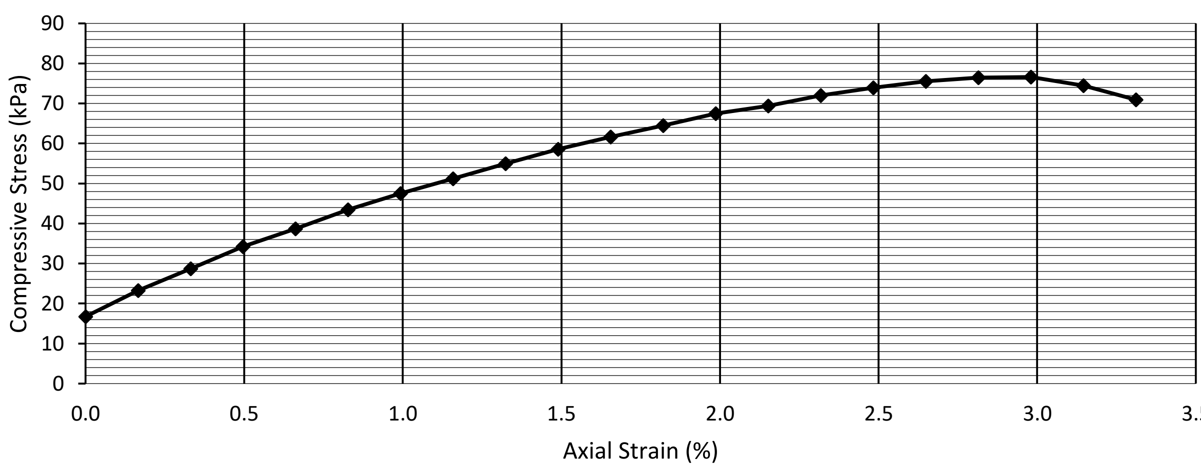
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	45.1
Pocket Pen.	Undrained Shear Strength (kPa)	44.7

UCS	Unconfined compressive strength (kPa)	76.60	Undrained Shear Strength (kPa)	38.30
	Unconfined compressive strength (ksf)	1.600	Undrained Shear Strength (ksf)	0.800
	Avg. Rate of Strain to Failure (%/min):	0.99	Strain at Failure (%):	2.98

Unconfined Compressive Strength



Comments:

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	NEWPCC - Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	9.14 - 9.75 m	Submitted By:	GAcurin
Sample Location:	TH24-02	Date Tested:	July 31, 2024
Sample Number:	T9	Tested By:	LBoughton

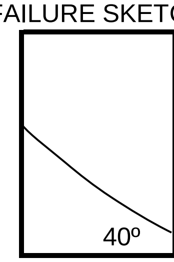
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, firm, moist, silty, High Plasticity, slickensided

Average Diameter (cm):	7.20
Average Length (cm):	14.90
Length/Diameter Ratio:	2.07
Moisture content (%):	51.3
Bulk Density (g/cm³):	1.702
Bulk Unit Weight (kN/m³):	16.7
Bulk Unit Weight (pcf):	106.3
Dry Unit Weight (kN/m³):	11.03

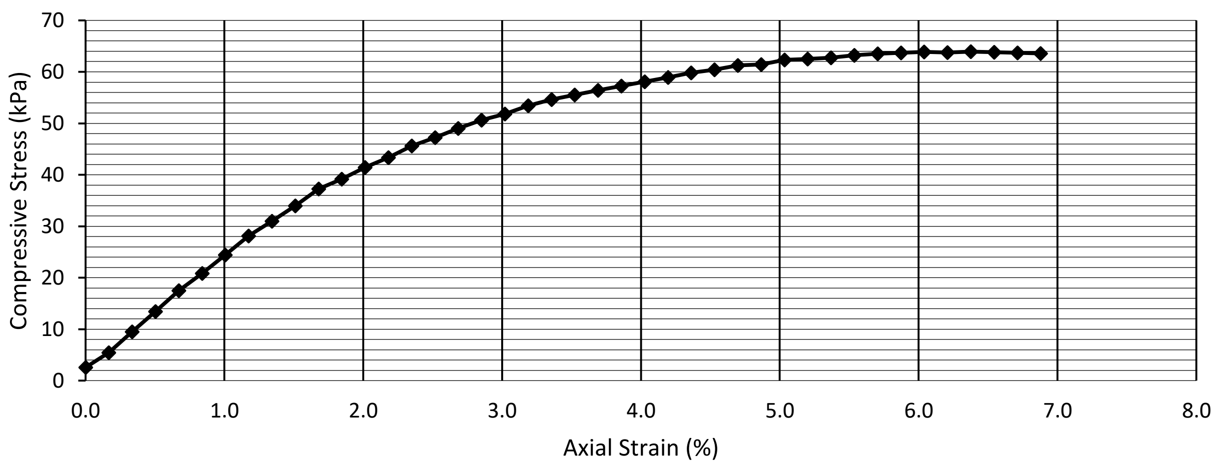
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	40.2
Pocket Pen.	Undrained Shear Strength (kPa)	40.7

UCS	Unconfined compressive strength (kPa)	63.93	Undrained Shear Strength (kPa)	31.96
	Unconfined compressive strength (ksf)	1.335	Undrained Shear Strength (ksf)	0.668
	Avg. Rate of Strain to Failure (%/min):	1.01	Strain at Failure (%):	6.38

Unconfined Compressive Strength



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	10.67 - 11.28 m	Submitted By:	GAcurin
Sample Location:	TH24-02	Date Tested:	July 31, 2024
Sample Number:	T10	Tested By:	LBoughton

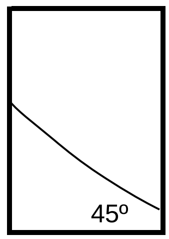
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, firm, moist, silty, High Plasticity, slickensided

Average Diameter (cm):	7.20
Average Length (cm):	15.10
Length/Diameter Ratio:	2.10
Moisture content (%):	44.0
Bulk Density (g/cm³):	1.793
Bulk Unit Weight (kN/m³):	17.6
Bulk Unit Weight (pcf):	111.9
Dry Unit Weight (kN/m³):	12.21

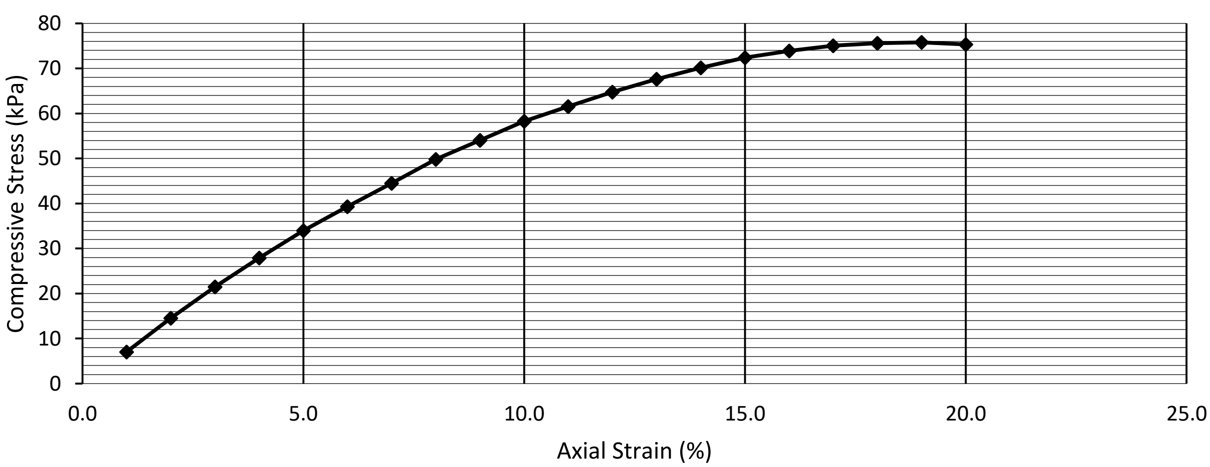
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	50.0
Pocket Pen.	Undrained Shear Strength (kPa)	44.7

UCS	Unconfined compressive strength (kPa)	75.78	Undrained Shear Strength (kPa)	37.89
	Unconfined compressive strength (ksf)	1.583	Undrained Shear Strength (ksf)	0.791
	Avg. Rate of Strain to Failure (%/min):	0.99	Strain at Failure (%):	2.98

Unconfined Compressive Strength



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by:	Lee Boughton	Approved by:	German Leal, M.Eng., P.Eng.
	Laboratory Manager		Geotechnical Discipline Lead

AECOM

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	NEWPCC Primary Scum Building		
Project Number:	60661262	Date Sampled:	July 24, 2024
Client:	City of Winnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, Manitoba	Date Received:	July 24, 2024
Sample Depth (m):	12.19 - 12.80 m	Submitted By:	GAcurin
Sample Location:	TH24-02	Date Tested:	July 31, 2024
Sample Number:	T11	Tested By:	LBoughton

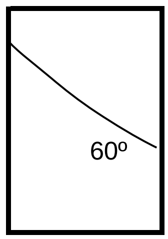
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, firm, moist, silty, High Plasticity, slickensided

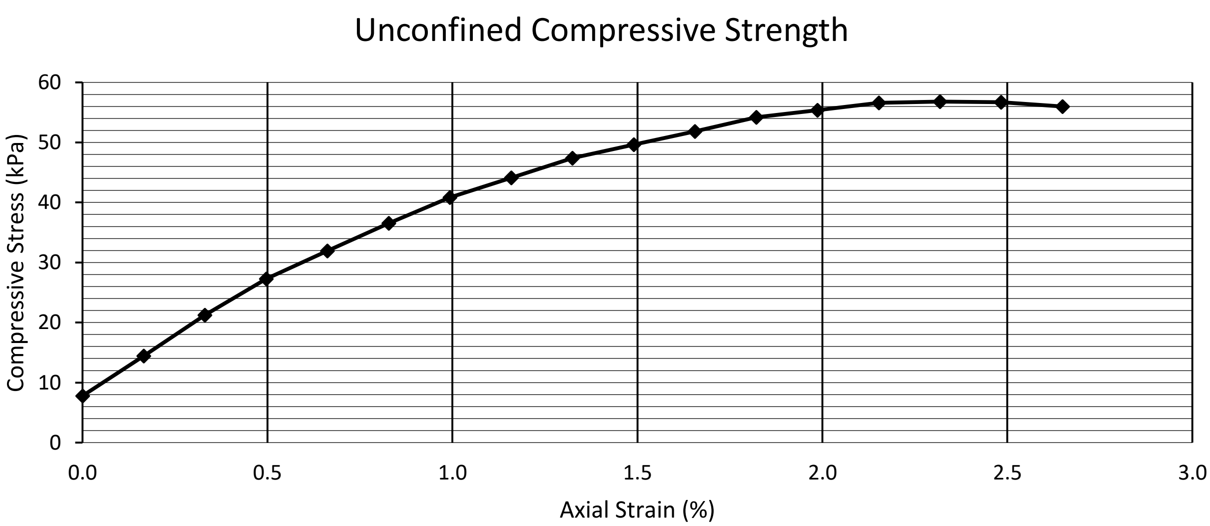
Average Diameter (cm):	7.20
Average Length (cm):	15.10
Length/Diameter Ratio:	2.10
Moisture content (%):	52.6
Bulk Density (g/cm³):	1.746
Bulk Unit Weight (kN/m³):	17.1
Bulk Unit Weight (pcf):	109.0
Dry Unit Weight (kN/m³):	11.22

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	44.1
Pocket Pen.	Undrained Shear Strength (kPa)	41.5

UCS	Unconfined compressive strength (kPa)	56.80	Undrained Shear Strength (kPa)	28.40
	Unconfined compressive strength (ksf)	1.186	Undrained Shear Strength (ksf)	0.593
	Avg. Rate of Strain to Failure (%/min):	0.99	Strain at Failure (%):	2.32



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by:	Lee Boughton Laboratory Manager	Approved by:	German Leal, M.Eng., P.Eng. Geotechnical Discipline Lead

ENG-TECH CONSULTING LIMITED

420 Turenne Street
Winnipeg, Manitoba
R2J 3W8
engtech@mymts.net
www.eng-tech.ca

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01

Ref. No.: 24-27-1-4,5,6,7

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60661262 - PRIMARY SCUM BUILDING

Submitted By: Client
Date Cored: Jul 10/24
Received By: ENG-TECH (Jessica Bauer)
Core Conditioning: As received moisture condition
Specimen Temperature: 24.0°C (room temperature)

Date Received: Aug 6/24
Tested By: ENG-TECH (Kevin Dowbeta)
Method: ASTM D2938-95

Core No.	Client I.D.	Test Hole Location / Core Depth (m)	Length		Average Diameter (mm)	Rate of Loading (kN/s)	Compressive Strength (MPa)	Date Tested (m/d/y)
			Cored (mm)	Tested (mm)				
1	C21	TH24-02 26.11 - 26.37	220	147.25	63.00	1.0	134	Aug 7/24
2	C21	TH24-02 26.37 - 26.66	281	155.00	63.00	0.9	117	Aug 7/24
3	C21	TH24-02 26.67 - 26.92	270	155.50	63.00	0.9	115	Aug 7/24
4	C21	TH24-02 27.17 - 27.39	224	135.75	63.00	0.9	103	Aug 7/24

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.
*Denotes core Length/Diameter ratio not between 2.0 and 2.5.

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

Enclosure: Unconfined Compressive Strength Of Intact Rock Core Specimen Reports Ref. No.'s 24-27-1-4,5,6 and 7

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420 Turenne Street
Winnipeg, Manitoba
R2J 3W8
engtech@mymts.net
www.eng-tech.ca

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

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99 Commerce Drive
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R3P 1J9

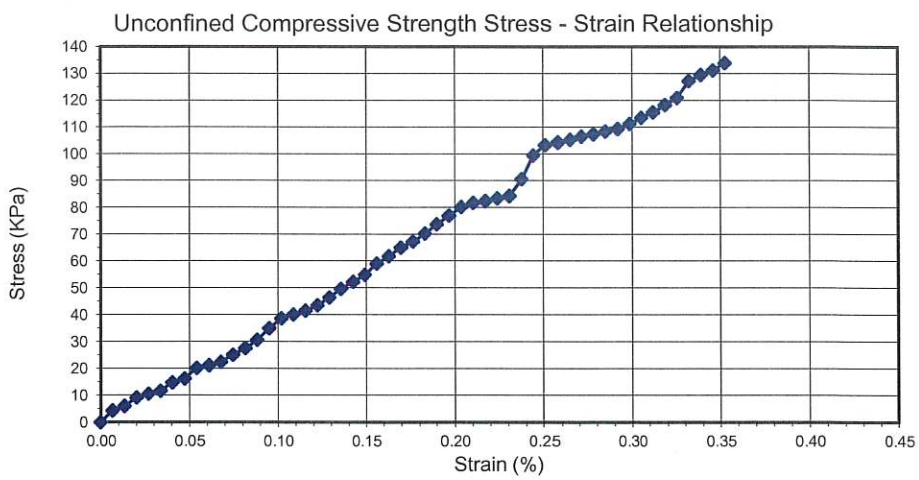
File No.: 24-027-01

Ref. No.: 24-27-1-4

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60661262 - PRIMARY SCUM BUILDING

Client I.D.	C21		
Test Hole/Depth	TH24-02, 26.11 - 26.37 meters	Submitted By:	Client
Date Cored:	Jul 10/24	Date Tested:	Aug 7/24
Date Received:	Aug 6/24	Tested By:	ENG-TECH (Kevin Dowbeta)
Compression Machine Model:	Soil Test CT-710	Method:	ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	24°C
Average Length of Specimen:	147.25 mm	Average Diameter of Specimen:	63.00 mm
Load Rate: 1.0 kN/s	Maximum Load: 417.2 kN	Compressive Strength:	134 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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420 Turenne Street
Winnipeg, Manitoba
R2J 3W8
engtech@mymts.net
www.eng-tech.ca

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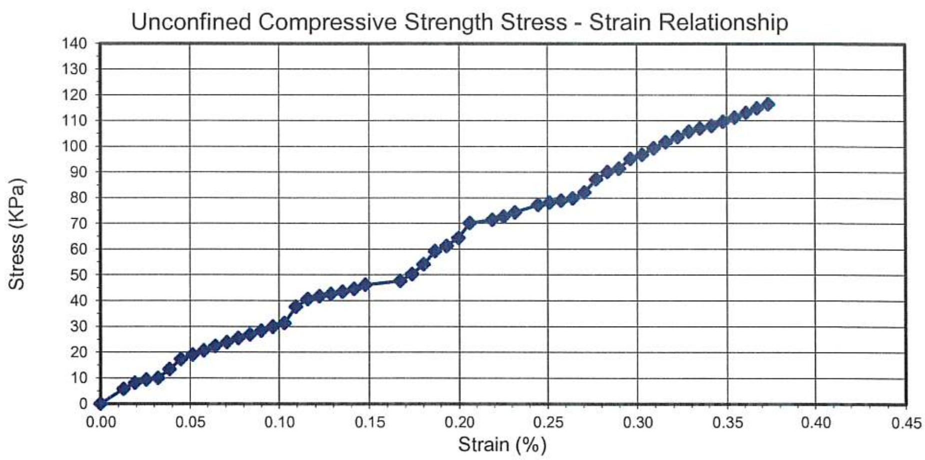
File No.: 24-027-01

Ref. No.: 24-27-1-5

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60661262 - PRIMARY SCUM BUILDING

Client I.D.	C21		
Test Hole/Depth:	TH24-02, 26.37 - 26.66 meters	Submitted By:	Client
Date Cored:	Jul 10/24	Date Tested:	Aug 7/24
Date Received:	Aug 6/24	Tested By:	ENG-TECH (Kevin Dowbeta)
Compression Machine Model:	Soil Test CT-710	Method:	ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	24°C
Average Length of Specimen:	155.50 mm	Average Diameter of Specimen:	63.00 mm
Load Rate: 0.9 kN/s	Maximum Load: 363.2 kN	Compressive Strength:	117 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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years of innovation

420 Turenne Street
Winnipeg, Manitoba
R2J 3W8
engtech@mymts.net
www.eng-tech.ca

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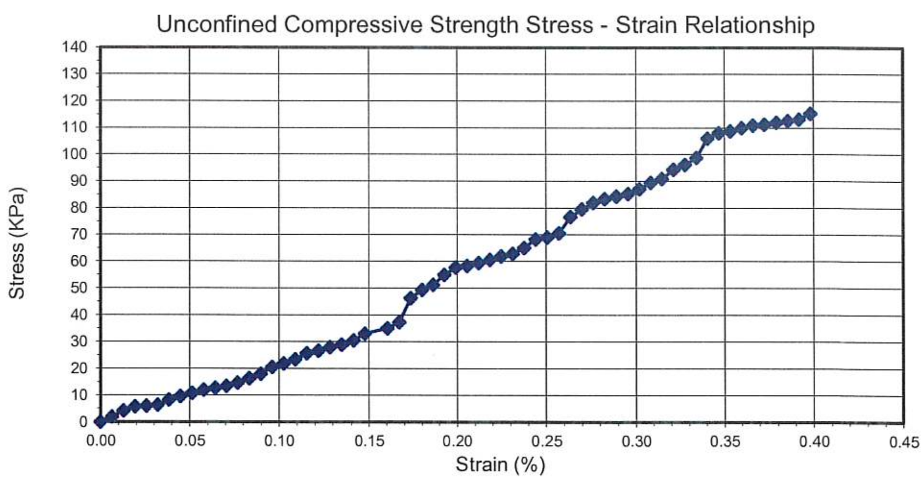
File No.: 24-027-01

Ref. No.: 24-27-1-6

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60661262 - PRIMARY SCUM BUILDING

Client I.D.	C21		
Test Hole/Depth:	TH24-02, 26.67 - 26.92 meters	Submitted By:	Client
Date Cored:	Jul 10/24	Date Tested:	Aug 7/24
Date Received:	Aug 6/24	Tested By:	ENG-TECH (Kevin Dowbeta)
Compression Machine Model:	Soil Test CT-710	Method:	ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	24°C
Average Length of Specimen:	155.50 mm	Average Diameter of Specimen:	63.00 mm
Load Rate: 0.9 kN/s	Maximum Load: 359.7 kN	Compressive Strength:	115 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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years of innovation

420 Turenne Street
Winnipeg, Manitoba
R2J 3W8
engtech@mymts.net
www.eng-tech.ca

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99 Commerce Drive
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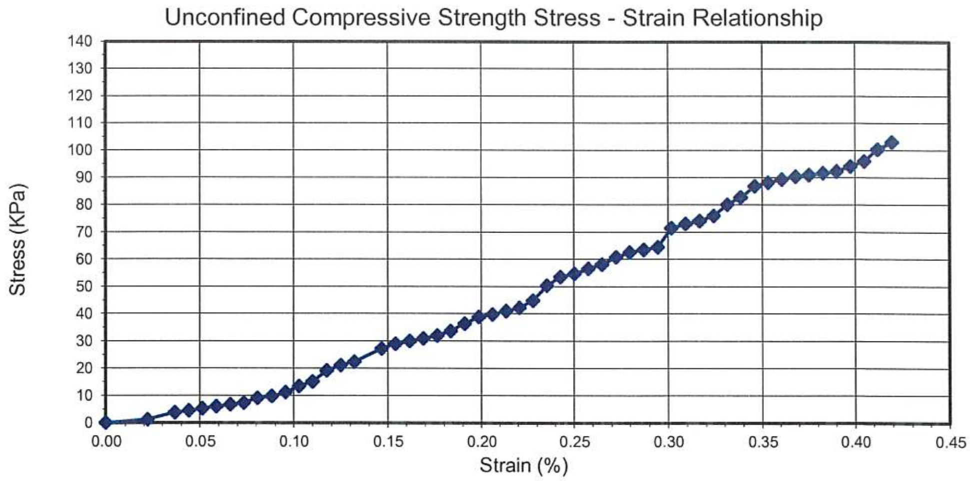
File No.: 24-027-01

Ref. No.: 24-27-1-7

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60661262 - PRIMARY SCUM BUILDING

Client I.D.	C21		
Test Hole/Depth:	TH24-02, 27.17 - 27.39 meters	Submitted By:	Client
Date Cored:	Jul 10/24	Date Tested:	Aug 7/24
Date Received:	Aug 6/24	Tested By:	ENG-TECH (Kevin Dowbeta)
Compression Machine Model:	Soil Test CT-710	Method:	ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	24°C
Average Length of Specimen:	135.75 mm	Average Diameter of Specimen:	63.00 mm
Load Rate: 0.9 kN/s	Maximum Load: 321.3 kN	Compressive Strength:	103 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

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Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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Appendix E

Seismic Hazard Values

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_E
Latitude (°)	49.952
Longitude (°)	-97.107

Please select one of the tabs below.

NBC 2020 | Additional Values | Plots | API | Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.113	0.107	0.055	0.0216	0.00434	0.00126	0.0679	0.0544

The log-log interpolated 2%/50 year $S_a(4.0, X_E)$ value is : **0.0064**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.0591	0.0565	0.028	0.0104	0.00193	0.000552	0.0339	0.027

The log-log interpolated 5%/50 year $S_a(4.0, X_E)$ value is : **0.0029**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.0334	0.0317	0.0149	0.00517	0.000881	0.000242	0.0184	0.0142

The log-log interpolated 10%/50 year $S_a(4.0, X_E)$ value is : **0.0014**

Download CSV

Go back to the seismic hazard calculator form

Date modified: 2021-04-06

Error! Reference source not found.

Colton Wooster
Geotechnical EIT
M: 204-583-8797
E: colton.wooster@aecom.com

German Leal
Discipline Lead, Geotechnical
M: 204-928-8479
E: german.leal@aecom.com

AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, MB R3P 0Y7
Canada

T: 204.477.5381
F: 431.800.1210
www.aecom.com