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AECOM

MacLean Regional Pumping Station Cooling Upgrades Geotechnical Report (Revised)

Prepared for:
Brad Peterson, P.Eng., LEED AP
Operations Manager, Buildings + Places, Winnipeg
AECOM
99 Commerce Drive, Winnipeg, MB
R3P 0Y7

Project Number: 0013 040 00

Date: February 2, 2022

TREK GEOTECHNICAL Quality Engineering | Valued Relationships

February 2, 2022

Our File No. 0013 040 00

Brad Peterson, P.Eng., LEED AP
Operations Manager, Buildings + Places, Winnipeg
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99 Commerce Drive, Winnipeg, MB
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RE: MacLean Regional Pumping Station Cooling Upgrades – Geotechnical Report (Revised)

TREK Geotechnical Inc. is pleased to submit our revised report for the geotechnical investigation for the above noted project.

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Ryan Belbas, M.Sc., P.Eng.
Senior Geotechnical Engineer

Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	RVR	November 12, 2022	Final Report
1	NM	February 2, 2022	Revision 1

Authorization Signatures

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TREK GEOTECHNICAL INC.
No. 4877 Date: Feb. 2/22

Brent Hay for:

Kent Bannister, M.Sc., P.Eng.
Senior Geotechnical Engineer

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1.0 Introduction

This report summarizes the results of a geotechnical investigation completed by TREK Geotechnical Inc. (TREK) for the proposed cooling system upgrades at the MacLean Regional Pumping Station (RPS) located at 875 Lagimodiere Blvd. in Winnipeg, Manitoba. The terms of reference for the investigation are included in our proposal to Brad Peterson of AECOM dated April 28, 2021. The scope of work includes a sub-surface investigation, laboratory testing and provision of foundation recommendations.

2.0 Background

The City of Winnipeg's regional water distribution system consists of three regional pumping stations (MacLean RPS, McPhillips RPS, and Hurst RPS) and two booster pumping stations (Deacon BPS and Taché BPS). The Deacon BPS station pumps the treated water from the Winnipeg Drinking Water Treatment Plant in Dugald, MB to the three RPS reservoirs located within Winnipeg. The RPSs and BPSs are critical infrastructure for the City and the failure of any of the pumping systems at these facilities has the potential to disrupt the City's residential, commercial, industrial, and fire protection water supplies. The MacLean, McPhillips, and Hurst RPSs as well as the Deacon BPS require cooling upgrades which will consist of exterior new air-cooled chillers and condensers. Based on equipment drawings provided by AECOM, the chiller and condenser are relatively light. The operating weight of the chiller is 33 kN (7,400 lbs) and the condenser is 32 kN (7,150 lbs). This report provides geotechnical recommendations for new foundations for the cooling equipment at the MacLean RPS which was constructed in 1964. The existing chiller is supported by a grade-supported mat foundation which is the preferred foundation alternative to support the new chillers and condensers.

3.0 Field Program

3.1 Sub-surface Investigation

A sub-surface investigation was completed at the MacLean RPS on October 5th, 2021 under the supervision of TREK personnel to determine the soil stratigraphy and groundwater conditions at the site. Two test holes (TH21-01A and -01B) were drilled and sampled to a depth of 6.7 m and 12.2 m below ground surface, respectively, within the vicinity of the coolant upgrades, at the locations shown on Figure 01. Power auger refusal occurred in TH21-01A at 6.7 m depth on an unknown obstruction, the drill rig was subsequently moved 3.2 m east and 2.6 m north to re-drill (TH21-01B). The test hole locations were selected to avoid critical buried infrastructure based on discussion during an on-site meeting with Jeff Brooks, Project Manager for the City of Winnipeg on October 1, 2021. The test holes were drilled by Paddock Drilling Ltd. using a Ranger 24 track-mounted drill rig equipped with 125 mm solid stem augers. The test holes were backfilled with auger cuttings and bentonite chips.

Sub-surface soils encountered during drilling were visually classified based on the Unified Soil Classification System (USCS). Disturbed (auger cutting) samples were taken at regular intervals and relatively undisturbed (Shelby tube) samples were collected at select depths. All samples retrieved during drilling were transported to TREK's testing laboratory in Winnipeg, Manitoba. Laboratory testing consisted of water content determination on all samples as well as bulk unit weight measurements and unconfined compression tests on select Shelby tube samples.

The test hole locations were determined by measuring offsets to the existing RPS building. The test hole elevations were surveyed using a rod and level relative to a temporary benchmark assigned an arbitrary elevation of 100.0 m. The temporary benchmark selected for this project was the top of the concrete slab located near the exit on the east side of the RPS building; its location is shown on Figure 01. The test hole elevations and offsets from the RPS building are provided on the test hole logs. The test hole logs also include a description of the soil units encountered and other pertinent information such as groundwater and sloughing conditions, and a summary of the laboratory testing results. Laboratory test results are included in Appendix A.

3.1.1 *Soil Stratigraphy*

A brief description of the soil units encountered during drilling is provided below. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the attached test hole log.

The stratigraphy at the test hole locations generally consists of 25 mm of clay topsoil, overlying clay fill and native silty clay which was present to the depth of exploration (12.2 m below grade). The clay fill is 6.1 m thick in TH21-01A and 6.7 m thick in TH21-01B. It is of high plasticity, moist, and stiff to very stiff becoming stiff with depth. The native silty clay is of high plasticity, moist and firm to stiff becoming soft to firm with depth. A 0.6 m layer of sand fill was encountered in TH21-01A at 5.5 to 6.1 m depth. Glacial till deposits are anticipated to be situated approximately 12 to 15 m below ground surface based on published geology maps of Winnipeg.

3.1.2 *Groundwater Conditions*

Seepage and sloughing were not observed during drilling. Squeezing of the test holes was observed within the clay fill between 3.0 and 6.1 m depth. Both test holes were open and dry to their full depth immediately after drilling and removal of the augers.

The groundwater observations made during drilling are short-term and should not be considered reflective of (static) groundwater levels at the site which would require monitoring over an extended period to determine. It is important to recognize that groundwater conditions may vary seasonally, annually, or as a result of construction activities.

4.0 Foundation Recommendations

Cast-in-place concrete (CIPC) friction piles are considered to be the most suitable foundation to support the proposed cooling equipment based on the observed sub-surface and anticipated loading conditions. Shallow foundations are also a suitable foundation alternative provided seasonal movements associated with freeze/thaw and moisture and volume changes of the underlying clay fill soils can be tolerated. Design and construction parameters for CIPC friction piles and grade-supported mat foundations are provided in this section and are based on Limit States Design in accordance with National Building Code of Canada (NBCC 2010).

TREK anticipates that the clay fill at the site was placed during original construction of the RPS in 1964 and, as such, consolidation settlement of the fill and the underlying native clay is assumed to be complete. In this regard, movements of a new mat foundation will likely be associated with seasonal volumetric changes within the bearing soils as described in more detail below.

4.1 Limit States Design

Limit States Design recommendations for shallow and deep foundations in accordance with the National Building Code of Canada (NBCC, 2010) are provided below. Limit States Design requires consideration of distinct loading scenarios comparing the structural loads to the foundation bearing capacity using resistance and load factors that are based on reliability criteria. Two general design scenarios are evaluated corresponding to the serviceability and ultimate capacity requirements.

The **Ultimate Limit State (ULS)** is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation units. The ULS foundation bearing capacity is obtained by multiplying the nominal (ultimate) bearing capacity by a resistance factor (reduction factor), which is then compared to the factored (increased) structural loads. The ULS bearing capacity must be greater or equal to the maximum factored load to provide an adequate margin of safety. Table 1 summarizes the resistance factors that can be used for the design of shallow and deep foundations as per the NBCC (2010) depending upon the method of analysis and verification testing completed during construction.

The **Service Limit State (SLS)** is concerned with limiting deformation or settlement of the foundation under service loading conditions such that the integrity of the structure will not be impacted. The Service Limit State should generally be analysed by calculating the settlement resulting from applied service loads and comparing this to the settlement tolerance of the structure. However, the settlement tolerance of the structure is typically not yet defined at the preliminary design stage. As such, SLS bearing capacities are often provided that are developed on the basis of limiting settlement to 25 mm or less. A more detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust the SLS capacity if a more stringent settlement tolerance is required or if large groups of piles are used.

Table 1. ULS Resistance Factors for Foundations (NBCC, 2010)

Resistance to Vertical Loads for Shallow Foundations (Analysis Methods)	Resistance Factor
Semi-empirical analysis using laboratory and in-situ test data	0.5
Bearing Resistance to Axial Load for Deep Foundations (Analysis Methods)	**Resistance Factor**
Semi-empirical analysis using laboratory and *in-situ* test data	0.4
Analysis using static loading test results	0.6
Uplift resistance by semi-empirical analysis.	0.3
Uplift resistance using loading test results.	0.4

4.2 Mat Foundations

Grade-supported mat foundations (mats) are a suitable foundation provided seasonal movements associated with freeze/thaw and moisture and volume changes of the underlying clay fill soils can be tolerated. A piled foundation will be required if seasonal movements cannot be tolerated. Mats bearing on stiff clay fill can be designed based on a Factored ULS bearing resistance of 125 kPa and a SLS bearing resistance of 85 kPa. The SLS bearing resistances are based on limiting settlement to 25 mm or less and the factored ULS bearing resistances were calculated using a resistance factor of 0.5.

Shallow foundations are subject to vertical movements associated with moisture and volume changes of the underlying clay fill soil. Although difficult to predict, these movements (total and differential) could be in the order of 50 mm or more. In this regard, flexible pipe connections will likely be required to accommodate these movements. It should be understood that seasonal movements are independent of displacement required to mobilize bearing capacity.

The clay fill soils at the site are also highly frost susceptible, which refers to the propensity of the soil to grow ice lenses and heave during freezing. Insulation such as Styrofoam Highload could be incorporated into the design of foundations to provide frost protection to an equivalent depth of 2.4 m for protection against seasonal frost related (i.e. freeze/thaw) movements. An insulation manufacturer or supplier should be contacted to verify the insulation design.

<u>Additional Design Recommendations:</u>

1. Mats should be designed by a structural engineer to resist axial, lateral, and bending loads from the structure as well as forces induced from seasonal movements (i.e. shrinkage/swelling and frost-related movements) of the bearing soils.

<u>Additional Construction Recommendations:</u>

1. Organics, debris, and all other deleterious materials should be removed such that the bearing surfaces consist of stiff clay fill.
2. Excavations for mats should be completed by an excavator equipped with a smooth-bladed bucket operating from the edge of the excavation.
3. After excavation, the clay fill bearing surface should be scarified, moisture conditioned and compacted to a minimum of 95% of the Standard Proctor Maximum Dry Density (SPMDD).
4. The bearing surfaces should be protected from freezing, drying, and inundation with water at all times. If any of these conditions occur, the disturbed material should be removed in its entirety and the clay fill bearing surface should be recompacted to 95% of the SPMDD. If groundwater seepage is encountered, it should be controlled and removed from the bearing surface, such that concrete is placed under dry conditions.
5. Final bearing surfaces should be inspected and documented by TREK prior to concrete placement to verify the adequacy of the bearing surface and proper installation of the foundation.

4.3 Friction Piles

Cast-in-place concrete friction piles will derive a majority of their resistance in shaft friction (adhesion) with a relatively small contribution from end bearing. Table 2 provides the recommended axial (compressive and uplift) unit resistances for shaft adhesion and end bearing. Piles designed based on the SLS resistances provided in Table 2 are expected to exhibit less than 10 mm of settlement at the pile toe. Elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement.

Table 2. ULS and SLS Resistances for Friction Piles

Pile Depth Below Existing Site Grade (m approx.)	SLS Unit Resistance (kPa)	Factored ULS Unit Resistance (kPa)		
		Compression $\phi = 0.4$		Uplift $\phi = 0.3$
		Shaft Adhesion	End Bearing(1)	Shaft Adhesion
0 to 2.4	-	-	-	-
2.4 to 8.5	12	14	-	11
8.5 to 12	7	8	55	6

1. *For piles with a diameter of less than 1.0 m. If larger pile diameters are required TREK should be contacted to provide revised end bearing values.*

Additional Design Recommendations:

1. The weight of the embedded portion of the pile may be neglected.
2. Piles should be designed with a maximum depth of 12 m below existing site grade to avoid penetration into the underlying silt till and to protect against heaving at the base of the pile shaft. In the event the silt till is encountered at shallower depths, the pile design may have to be re-evaluated by the structural engineer.
3. Piles should have a minimum spacing of 3 pile diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
4. Piles require steel reinforcement designed by a structural engineer for the anticipated axial (compression and tension), lateral and bending loads induced from the structure. Piles subject to frost jacking forces should be reinforced for their entire length.

Additional Installation Recommendations:

1. Temporary steel casings (sleeves) should be available and used if sloughing of the pile hole occurs and/or to control groundwater seepage, if encountered. Care should be taken in removing sleeves to prevent sloughing (necking) of the shaft walls and a reduction in the cross-sectional area of the pile.
2. Concrete should be placed in one continuous operation immediately after the completion of drilling the pile hole to avoid potential construction problems such as sloughing or caving of the pile hole and groundwater seepage. Concrete placed by free-fall methods should be poured under dry conditions. If groundwater is encountered, it should be controlled or removed. If water cannot be controlled or removed, the concrete should be placed using tremie methods.
3. Concrete placed by free-fall methods should be directed through the middle of the pile shaft and steel reinforcing cage to prevent striking of the drilled shaft walls to protect against soil contamination of the concrete.

4.4 Adfreeze Effects

Concrete piles, pile caps, grade beams and walls subjected to freezing conditions should be designed to resist ad-freeze and uplift forces related to frost action acting along the vertical face of the member within the depth of frost penetration (2.4 m). In this regard, concrete piles, pile caps, grade beams and walls may be subject to an ad-freeze bond stress of 65 kPa within the depth of frost penetration. Ad-freeze forces will be resisted by structural dead loads and uplift resistance provided by the length of the pile below the depth of frost penetration. The following design recommendations apply to piles subject to ad-freeze forces:

1. An ad-freeze bond stress of 65 kPa within the depth of frost penetration (2.4 m).
2. A load factor (α) of 1.2 may be used in the calculation of ad-freezing forces.
3. A reduction factor of 0.8 may be used in calculation of the geotechnical resistance for the factored ULS condition with an ultimate (nominal) uplift resistance of 35 kPa between 2.4 and 8.5 m depth and 20 kPa below.
4. Resistance to ad-freezing within the depth of frost penetration (2.4 m) should be neglected.
5. Structural dead loads should be added to the resistance.
6. The calculated geotechnical resistance plus the structural dead loads must be greater than the factored ad-freezing forces.
7. Piles subject to ad-freezing forces should be a minimum of 8.0 m or as calculated by the method above, whichever is greater.
8. Measures such as flat lying rigid polystyrene insulation could be considered to reduce frost penetration depths and thereby ad-freezing and uplift forces.

4.5 Negative Skin Friction and Dragload

It is possible that the existing fill at the site is still undergoing settlement from its original placement which could result in development of negative skin friction along pile shafts causing dragload and excessive settlement (downdrag) of new CIPC friction piles. However, TREK suspects that the existing fill has been in place since construction of the RPS in 1964 and, therefore, the risk of additional settlement of the fill and pile downdrag is considered to be low and pile settlement is expected to be less than 25 mm. Should this not be the case, TREK should be contacted to evaluate the potential effects of negative skin friction.

4.6 Foundation Concrete

All foundation concrete should be designed by a structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads from the structure. Based on local experience gathered through previous work in Winnipeg, the degree of exposure for concrete subjected to sulphate attack is classified as severe according to Table 3, CSA A23.1-14 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56-day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-14 for concrete with severe sulphate exposure (S2).

Concrete that may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-14.

4.7 Pile Caps and Grade Beams

A minimum void of 150 mm should be provided underneath all pile caps and grade beams to accommodate volumetric changes in the underlying sub-grade soils (i.e. swelling, shrinkage, and thermal expansion and contraction in unheated areas). Void forms should be used under pile caps and grade beams and should be capable of deforming a minimum of 150 mm without transferring any stress to the structure. Excavations for pile caps and grade beams should be backfilled with non-frost susceptible granular fill compacted to a minimum of 95% of the Standard Proctor Maximum Dry Density (SPMDD).

4.8 Foundation Inspection Requirements

In accordance with Section *4.2.2.3 Field Review* of the NBCC (2010) states that the designer or other suitably qualified person shall carry out a field review on:

a) continuous basis during:

   i. the construction of all deep foundation units with all pertinent information recorded for each *foundation unit*,

   iii. during the placement of engineered fills that are to be used to support the *foundation units*,

b) on an as-required basis for the construction of shallow foundation units and in excavating, dewatering and other related works.

In consideration of the above and relative to this particular project, TREK is familiar with the geotechnical conditions and the basis for the foundation recommendations and can provide any design modifications deemed to be necessary should unexpected sub-surface conditions be encountered. TREK, as the geotechnical engineer of record, should be retained to observe the installation of any foundation elements.

5.0 Temporary Excavations

Excavations must be carried out in compliance with the appropriate regulations under the Manitoba Workplace Safety and Health Act. Any open-cut excavation greater than 3 m deep must be designed and sealed by a professional engineer and reviewed by the geotechnical engineer of record (TREK). If space is limited or the stability of adjacent structures may be endangered by an excavation, a shoring system may be required to prevent damage to, or movement of, any part of adjacent structures, and the creation of a hazard to workers and the public.

Excavation stability is the responsibility of the Contractor for the duration of construction. Excavations should be monitored regularly and flattened as necessary to maintain stability recognizing that excavation stability is time and weather dependent. Excavated slopes should be covered with polyethylene sheets to prevent wetting and drying.

Stockpiles of excavated material and heavy equipment should be kept away from the edge of any excavation by a distance equal to or greater than the depth of excavation. Dewatering measures should be completed as necessary to maintain a dry excavation and permit proper completion of the work. If seepage is encountered, it should be collected and pumped out of the excavation. If saturated silts or sands are encountered, shoring or slope flattening may be required. To prevent wet silts and sands from entering the excavation, gravel buttressing could be used in conjunction with sump pits for dewatering. Surface water should be diverted away from the excavation and the excavation should be backfilled as soon as possible following construction.

6.0 Site Drainage

Drainage adjacent to the slab should promote runoff away from the structure. A minimum gradient of about 2% should be used for both landscaped and paved areas and maintained throughout the life of the structures.

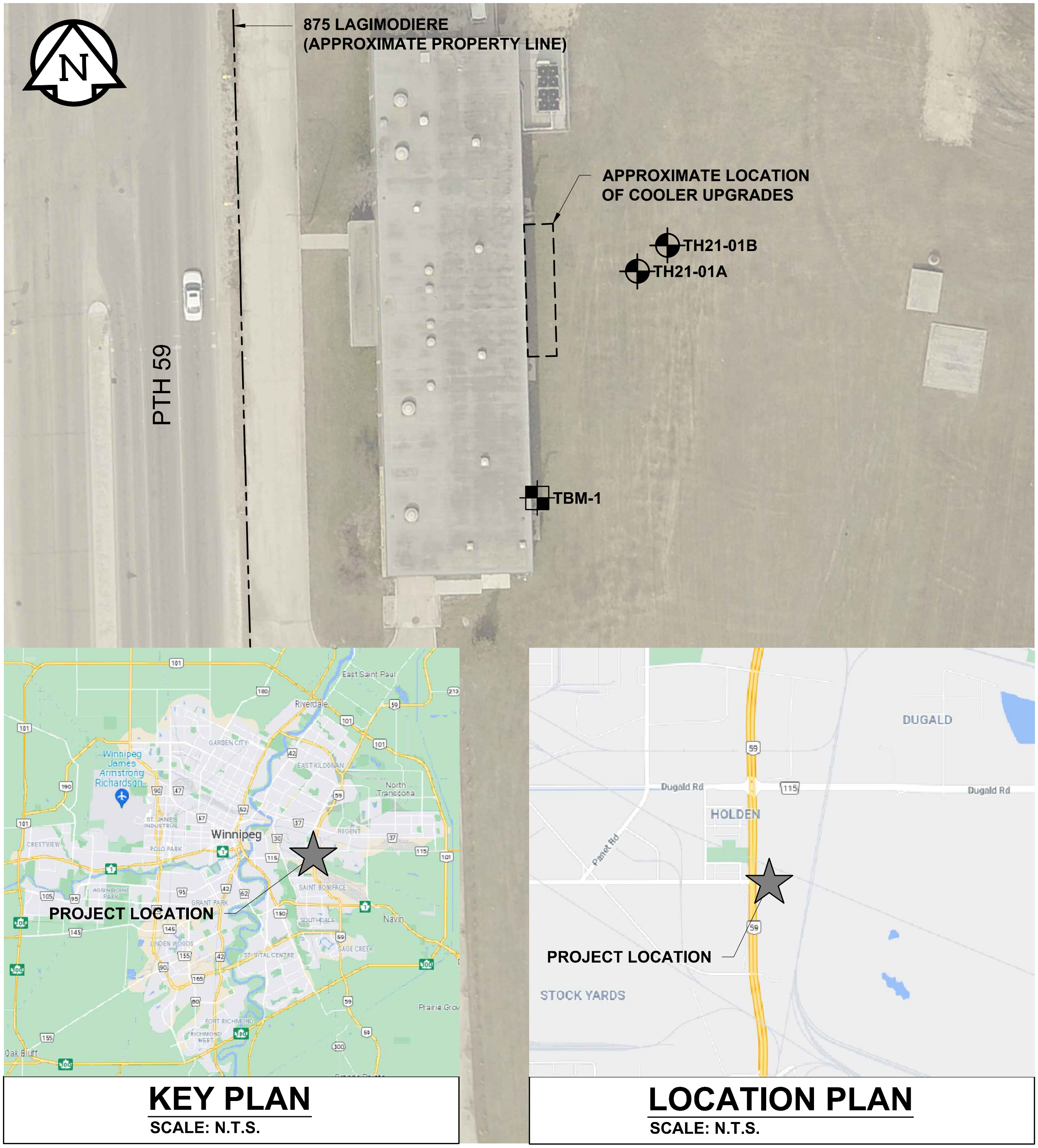
7.0 Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of AECOM (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Figure



LEGEND:

- TEST HOLE (TREK, 2021)
- TEMPORARY BENCHMARK TBM1

NOTES:

1. AERIAL IMAGERY FROM CITY OF WINNIPEG, (2016).
2. TEMPORARY BENCHMARK WAS ESTABLISHED ON TOP OF THE EXTERIOR CONCRETE SLAB AT THE EAST DOORWAY.

0 5 10 15 20 25 m

SCALE = 1 : 600 (216 mm x 279 mm)

Figure 01
Test Hole Location Plan

Test Hole Logs

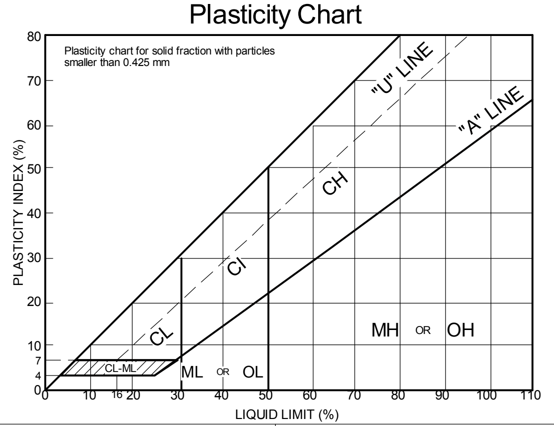
EXPLANATION OF FIELD AND LABORATORY TESTING

GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. Descriptions on these test hole logs apply only at the specific test hole locations and at the time the test holes were drilled. Variability of soil and groundwater conditions may exist between test hole locations.
3. When the following classification terms are used in this report or test hole logs, the primary and secondary soil fractions may be visually estimated.

Major Divisions			USCS Classification	Symbols	Typical Names	Laboratory Classification Criteria	
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent...... GW, GP, SW, SP; More than 12 percent...... GM, GC, SM, SC; 6 to 12 percent...... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			GP		Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW
		Gravel with fines (Appreciable amount of fines)	GM		Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4 / Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC		Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7
	Sands (More than half of coarse fraction is smaller than 4.75 mm)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3
			SP		Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW
		Sands with fines (Appreciable amount of fines)	SM		Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4 / Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart	
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			OL		Organic silts and organic silty clays of low plasticity		
	Silts and Clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or distomaceous fine sandy or silty soils, organic silts		
			CH		Inorganic clays of high plasticity, fat clays		
			OH		Organic clays of medium to high plasticity, organic silts		
	Highly Organic Soils		Pt		Peat and other highly organic soils	Von Post Classification Limit	Strong colour or odour, and often fibrous texture

Plasticity Chart



Particle Size			
Material		ASTM Sieve sizes	mm
Sand	Coarse	#10 to #4	2.00 to 4.75
	Medium	#40 to #10	0.425 to 2.00
	Fine	#200 to #40	0.075 to 0.425
Silt or Clay		< #200	< 0.075
Boulders		> 12 in.	> 300
Cobbles		3 in. to 12 in.	75 to 300
Gravel	Coarse	3/4 in. to 3 in.	19 to 75
	Fine	#4 to 3/4 in.	4.75 to 19

* Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols. For example; GW-GC, well-graded gravel-sand mixture with clay binder.

Other Symbol Types

	Asphalt		Bedrock (undifferentiated)		Cobbles
	Concrete		Limestone Bedrock		Boulders and Cobbles
	Fill		Cemented Shale		Silt Till
			Non-Cemented Shale		Clay Till

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

TERM	EXAMPLES	PERCENTAGE
and	and CLAY	35 to 50 percent
"y" or "ey"	clayey, silty	20 to 35 percent
some	some silt	10 to 20 percent
trace	trace gravel	1 to 10 percent

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

The Standard Penetration Test blow count (N) of a non-cohesive soil can be related to compactness condition as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

The Standard Penetration Test blow count (N) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very soft	< 2
Soft	2 to 4
Firm	4 to 8
Stiff	8 to 15
Very stiff	15 to 30
Hard	> 30

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	Undrained Shear Strength (kPa)
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

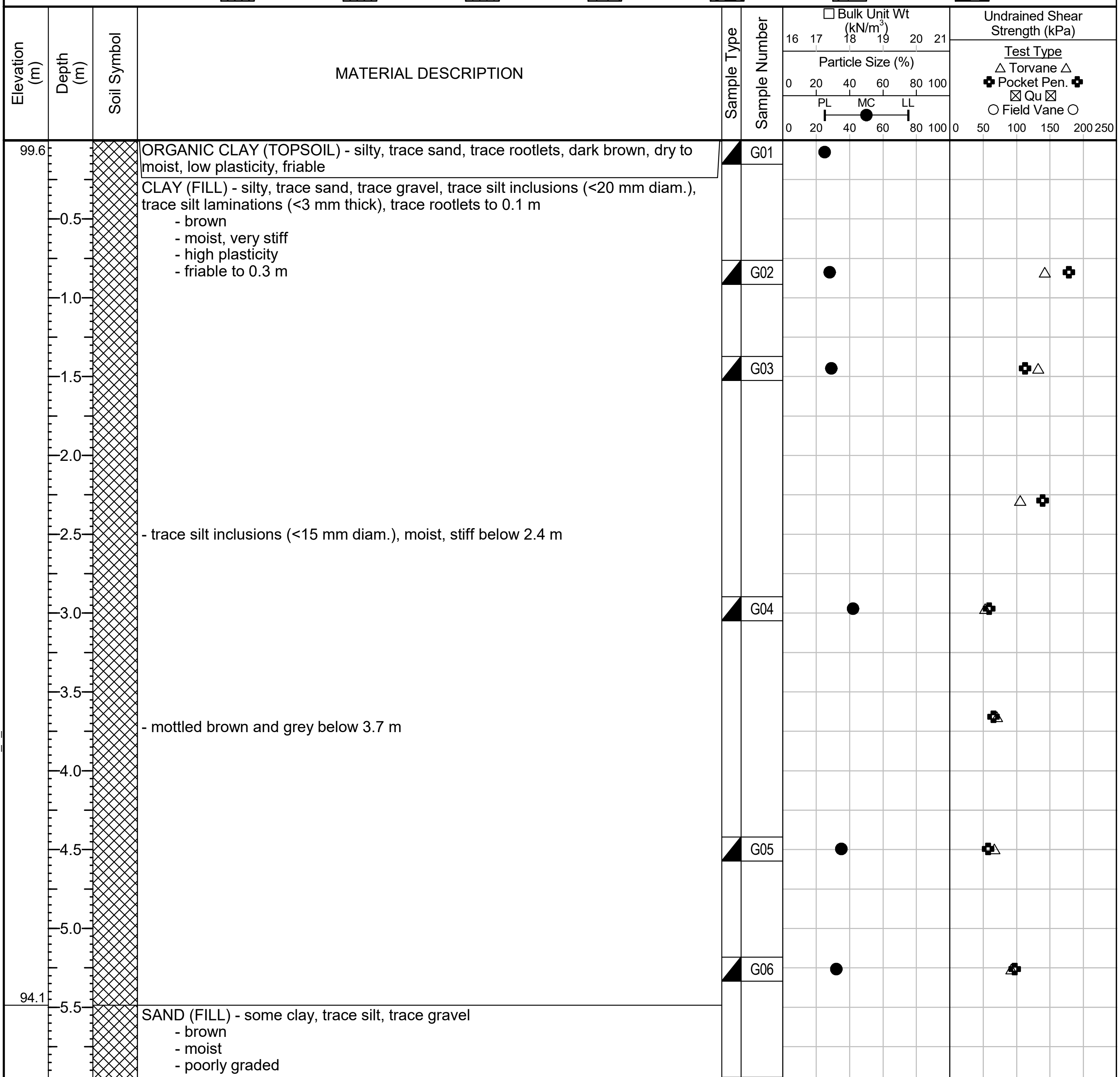
Sub-Surface Log

TREK GEOTECHNICAL

Client:	AECOM	**Project Number:**	0013 040 00
Project Name:	Winnipeg RPS Cooling Upgrades	**Location:**	875 Lagimodier Blvd; 24.7m S and 11.7m E of NE corner of
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	99.60 m Existing Ground (local datum)
Method:	125 mm Solid Stem Augers Ranger 24 Track Mount	**Date Drilled:**	October 5, 2021

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) / SPT | Split Barrel (SB) / LPT | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders



Logged By: Reinhardt Van Rensburg	**Reviewed By:** Kent Bannister	**Project Engineer:** Ryan Belbas

Sub-Surface Log

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	Bulk Unit Wt (kN/m³) / Particle Size (%) / PL MC LL	Undrained Shear Strength (kPa)
93.5					G07		
	6.5		CLAY (FILL) - silty, trace sand, trace silt inclusions (<5 mm diam.) - dark brown - moist, firm to stiff - high plasticity		G08		
92.9							

Test Type: △ Torvane △, ✚ Pocket Pen. ✚, ⊠ Qu ⊠, ○ Field Vane ○

END OF TEST HOLE AT 6.7 m IN CLAY

Notes:

1. Power auger refusal encountered at 6.7 m depth.
2. Seepage or sloughing not observed.
3. Squeezing observed in clay (fill) between 3.0 and 6.1 m depth during drilling.
4. Test hole open to 6.7 m depth and dry immediately after drilling.
5. Test hole backfilled with auger cuttings to 3.0 m depth and bentonite chips to surface.

Logged By: Reinhardt Van Rensburg **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

TREK GEOTECHNICAL

Sub-Surface Log

Client:	AECOM	**Project Number:**	0013 040 00
Project Name:	Winnipeg RPS Cooling Upgrades	**Location:**	875 Lagimodier Blvd; 22.1m S and 14.9m E of NE corner of
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	99.56 m Existing Ground (local datum)
Method:	125 mm Solid Stem Augers Ranger 24 Track Mount	**Date Drilled:**	October 5, 2021

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) / SPT | Split Barrel (SB) / LPT | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

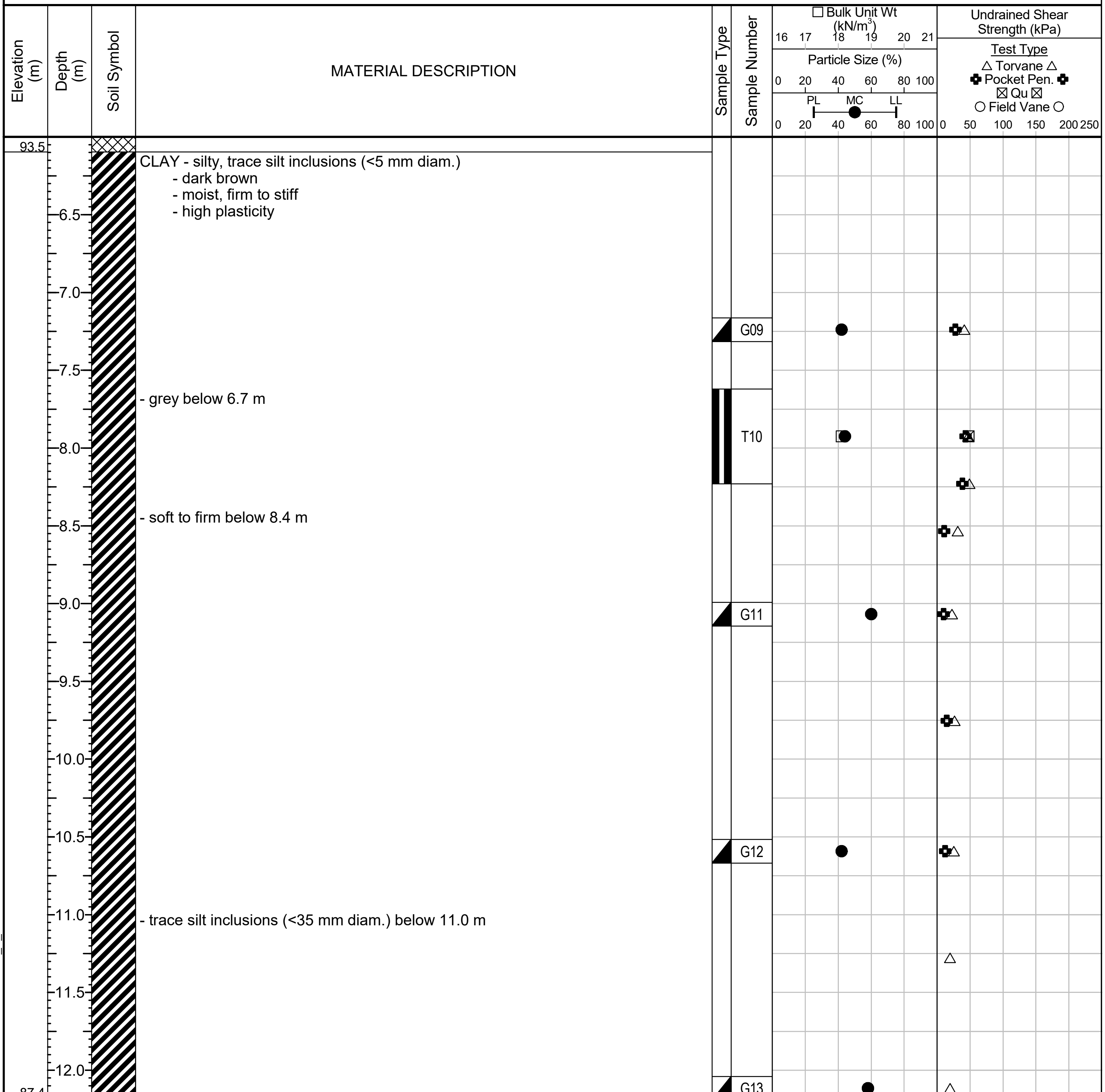
Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 / Particle Size (%) 0 20 40 60 80 100 / PL MC LL 0 20 40 60 80 100	Undrained Shear Strength (kPa) Test Type: △ Torvane △ ✚ Pocket Pen. ✚ ⊠ Qu ⊠ ○ Field Vane ○ 0 50 100 150 200 250
99.5			ORGANIC CLAY (TOPSOIL) - silty, trace sand, trace rootlets, dark brown, dry to moist, low plasticity, friable				
			CLAY (FILL) - silty, trace sand, trace gravel, trace silt inclusions (<20 mm diam.), trace silt laminations (<3 mm thick), trace rootlets to 0.1 m - brown - moist, very stiff - high plasticity - friable to 0.3 m				
	0.5						
	1.0						
	1.5						
	2.0						
	2.5		- trace silt inclusions (<15 mm diam.), moist, stiff below 2.4 m				
	3.0						
	3.5		- mottled brown and grey below 3.7 m				
	4.0						
	4.5						
	5.0						
	5.5						

Logged By: Reinhardt Van Rensburg **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

GEOTECHNICAL

Sub-Surface Log

Test Hole TH21-01B (MacLean RPS)



END OF TEST HOLE AT 12.2 m IN CLAY

Notes:

1. Seepage or sloughing not observed.
2. Squeezing observed in clay (fill) between 3.0 and 6.1 m depth during drilling.
3. Test hole open to 12.2 m depth and dry immediately after drilling.
4. Test hole backfilled with auger cuttings to 3.0 m depth and bentonite chips to surface.

Logged By: Reinhardt Van Rensburg **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

Appendix A

Laboratory Testing

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Winnipeg, MB R3H 0L3
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Moisture Content Report
ASTM D2216-10

Project No. 0013-040-00
Client AECOM
Project Winnipeg RPS- MacLean RPS

Sample Date 05-Oct-21
Test Date 07-Oct-21
Technician DJ

Test Hole	TH21-01	TH21-01	TH21-01	TH21-01	TH21-01	TH21-01
Depth (m)	0.0 - 0.2	0.8 - 0.9	1.4 - 1.5	2.9 - 3.0	4.4 - 4.6	5.2 - 5.3
Sample #	G01	G02	G03	G04	G05	G06
Tare ID	AB67	K33	Z68	P14	F88	D12
Mass of tare	7.0	8.6	8.6	8.7	8.4	8.4
Mass wet + tare	211.9	243.7	275.6	269.8	223.2	265.2
Mass dry + tare	170.8	192.6	215.4	192.9	167.3	203.3
Mass water	41.1	51.1	60.2	76.9	55.9	61.9
Mass dry soil	163.8	184.0	206.8	184.2	158.9	194.9
Moisture %	25.1%	27.8%	29.1%	41.7%	35.2%	31.8%

Test Hole	TH21-01	TH21-01	TH21-01	TH21-01	TH21-01	TH21-01
Depth (m)	5.9 - 6.1	6.4 - 6.6	7.2 - 7.3	9.0 - 9.1	10.5 - 10.7	12.0 - 12.2
Sample #	G07	G08	G09	G11	G12	G13
Tare ID	N75	E121	W91	E33	E69	H67
Mass of tare	8.6	8.4	8.6	8.5	8.7	8.7
Mass wet + tare	210.7	231.7	234.1	236.7	264.4	245.0
Mass dry + tare	168.6	169.9	167.3	151.3	188.5	158.2
Mass water	42.1	61.8	66.8	85.4	75.9	86.8
Mass dry soil	160.0	161.5	158.7	142.8	179.8	149.5
Moisture %	26.3%	38.3%	42.1%	59.8%	42.2%	58.1%

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Shelby Tube Visual

Project No.	0013-040-00
Client	AECOM
Project	Winnipeg RPS- MacLean RPS
Test Hole	TH21-01
Sample #	T10
Depth (m)	7.6 - 8.2
Sample Date	05-Oct-21
Test Date	09-Oct-21
Technician	DJ

Tube Extraction

Recovery (mm) 550

Bottom - 8.2 m … **Top - 7.7 m**

	8.22 m	8.12 m	7.94 m	7.77 m
Toss	Moisture Content PP/TV Visual	Keep	Qu Bulk	Toss
30 mm	100 mm	180 mm	170 mm	70 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<10.0 mm diam.)	
trace gravel (<10.0 mm diam.)	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.48
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	47.1

Pocket Penetrometer

Reading	**1**	0.90
	2	0.90
	3	0.85
	Average	0.88
Undrained Shear Strength (kPa)		43.3

Moisture Content

Tare ID	N04
Mass tare (g)	8.6
Mass wet + tare (g)	267
Mass dry + tare (g)	188.6
Moisture %	43.6%

Unit Weight

Bulk Weight (g)		1139.0
Length (mm)	**1**	149.83
	2	149.67
	3	149.20
	4	149.76
Average Length (m)		0.150
Diam. (mm)	**1**	71.95
	2	72.58
	3	72.87
	4	72.60
Average Diameter (m)		0.073
Volume (m^3)		6.18E-04
Bulk Unit Weight (kN/m^3)		18.1
Bulk Unit Weight (pcf)		115.1
Dry Unit Weight (kN/m^3)		12.6
Dry Unit Weight (pcf)		80.2

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Unconfined Compressive Strength
ASTM D2166

Project No.	0013-040-00
Client	AECOM
Project	Winnipeg RPS- MacLean RPS

Test Hole	TH21-01
Sample #	T10
Depth (m)	7.6 - 8.2
Sample Date	5-Oct-21
Test Date	9-Oct-21
Technician	DJ

Unconfined Strength

	kPa	ksf
Max q_u	95.7	2.0
Max S_u	47.8	1.0

Specimen Data

Description CLAY - silty, trace silt inclusions (<10.0 mm diam.), trace gravel (<10.0 mm diam.), dark grey, moist, firm, high plasticity

Length	149.6	(mm)	**Moisture %**	44%	
Diameter	72.5	(mm)	**Bulk Unit Wt.**	18.1	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	12.6	(kN/m^3)
Initial Area	0.00413	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

trace silt inclu	**Undrained Shear Strength**	
trace gravel (<	**kPa**	**ksf**
0.48	47.1	0.98
Vane Size		
m		

Pocket Penetrometer

Reading		**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	0.90	44.1	0.92
	0.90	44.1	0.92
	0.85	41.7	0.87
Average	**0.88**	**43.3**	**0.90**

Failure Geometry

Sketch:

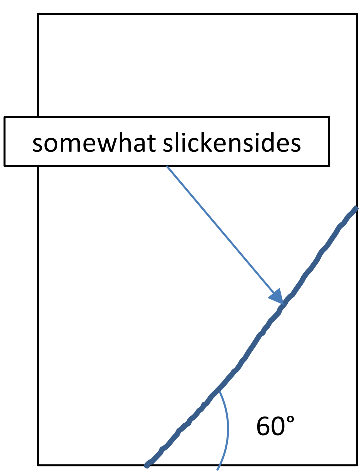
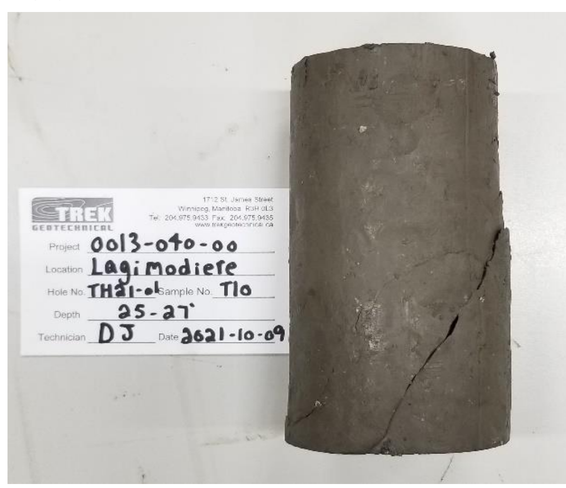


Photo:



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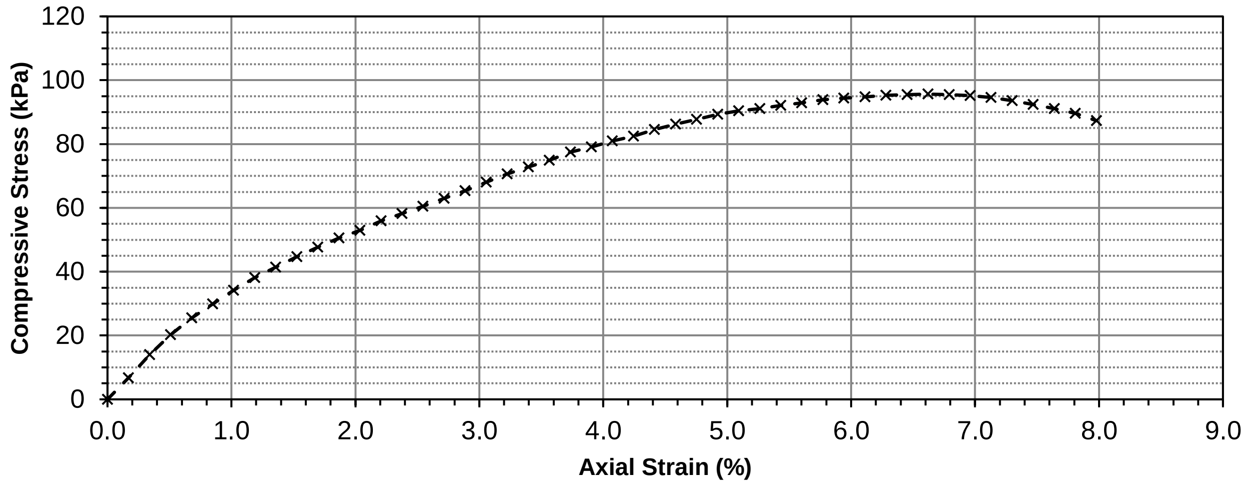
Unconfined Compressive Strength
ASTM D2166

Project No. 0013-040-00
Client AECOM
Project Winnipeg RPS- MacLean RPS

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0.07	0.0000	0.00	0.004128	0.0	0.00	0.00
10	0.62	0.2540	0.17	0.004135	27.7	6.70	3.35
20	1.22	0.5080	0.34	0.004142	58.0	13.99	7.00
30	1.74	0.7620	0.51	0.004149	84.2	20.29	10.14
40	2.17	1.0160	0.68	0.004156	105.8	25.47	12.73
50	2.54	1.2700	0.85	0.004164	124.5	29.90	14.95
60	2.90	1.5240	1.02	0.004171	142.6	34.20	17.10
70	3.23	1.7780	1.19	0.004178	159.3	38.12	19.06
80	3.51	2.0320	1.36	0.004185	173.4	41.43	20.71
90	3.79	2.2860	1.53	0.004192	187.5	44.72	22.36
100	4.04	2.5400	1.70	0.004200	200.1	47.65	23.82
110	4.29	2.7940	1.87	0.004207	212.7	50.56	25.28
120	4.50	3.0480	2.04	0.004214	223.3	52.99	26.49
130	4.75	3.3020	2.21	0.004221	235.9	55.88	27.94
140	4.95	3.5560	2.38	0.004229	246.0	58.17	29.08
150	5.15	3.8100	2.55	0.004236	256.0	60.44	30.22
160	5.37	4.0640	2.72	0.004244	267.1	62.95	31.48
170	5.58	4.3180	2.89	0.004251	277.7	65.33	32.67
180	5.82	4.5720	3.06	0.004258	289.8	68.06	34.03
190	6.05	4.8260	3.23	0.004266	301.4	70.66	35.33
200	6.24	5.0800	3.40	0.004273	311.0	72.77	36.39
210	6.43	5.3340	3.57	0.004281	320.6	74.88	37.44
220	6.66	5.5880	3.73	0.004288	332.2	77.45	38.73
230	6.81	5.8420	3.90	0.004296	339.7	79.08	39.54

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Unconfined Compressive Strength
ASTM D2166

Project No. 0013-040-00
Client AECOM
Project Winnipeg RPS- MacLean RPS

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	6.98	6.0960	4.07	0.004304	348.3	80.93	40.46
250	7.12	6.3500	4.24	0.004311	355.3	82.42	41.21
260	7.31	6.6040	4.41	0.004319	364.9	84.49	42.25
270	7.47	6.8580	4.58	0.004327	373.0	86.21	43.10
280	7.61	7.1120	4.75	0.004334	380.0	87.68	43.84
290	7.76	7.3660	4.92	0.004342	387.6	89.27	44.63
300	7.87	7.6200	5.09	0.004350	393.1	90.38	45.19
310	7.95	7.8740	5.26	0.004358	397.2	91.15	45.57
320	8.05	8.1280	5.43	0.004365	402.2	92.14	46.07
330	8.13	8.3820	5.60	0.004373	406.2	92.89	46.45
340	8.23	8.6360	5.77	0.004381	411.3	93.88	46.94
350	8.29	8.8900	5.94	0.004389	414.3	94.40	47.20
360	8.34	9.1440	6.11	0.004397	416.8	94.80	47.40
370	8.40	9.3980	6.28	0.004405	419.9	95.31	47.66
380	8.43	9.6520	6.45	0.004413	421.4	95.48	47.74
390	8.46	9.9060	6.62	0.004421	422.9	95.65	47.83
400	8.46	10.1600	6.79	0.004429	422.9	95.48	47.74
410	8.45	10.4140	6.96	0.004437	422.4	95.19	47.60
420	8.41	10.6680	7.13	0.004445	420.4	94.57	47.28
430	8.34	10.9220	7.30	0.004453	416.8	93.60	46.80
440	8.25	11.1760	7.47	0.004462	412.3	92.41	46.21
450	8.15	11.4300	7.64	0.004470	407.3	91.11	45.56
460	8.03	11.6840	7.81	0.004478	401.2	89.60	44.80
470	7.84	11.9380	7.98	0.004486	391.6	87.30	43.65