Appendix A
Geotechnical Investigation Report

UMA | AECOM

UMA Engineering Ltd.
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Memorandum

Date:	September 14, 2007
To:	Blair Moore, UMA
From:	Faris Khalil, UMA
Subject:	Perimeter Road Sewage Pumping Station (PRPS) Reliability Upgrade Geotechnical Investigation Report
Project Number:	D265-202-02 (4.6.1)

Distribution: Barry Biswanger

INTRODUCTION

This memorandum summarizes the results of the geotecnhical investigation and provides foundation and excavation recommnedations for the proposed PRPS upgrade.

GEOTECHNICAL INVESTIGATION

On April 16 and 18, 2007, three test holes (07-01 to 07-03) were drilled to assess the soil and groundwater conditions at the proposed pumping station stair well locations as shown on Test Hole Location Plan, Figure 01. Drilling of test holes was completed by Paddock Drilling Ltd. using a truck mounted Acker MP5-T and SS2 drill rig equipped with 125 mm diameter solid stem augers. Test Hole (TH) 07-01 was drilled near the south side of the existing building and TH 07-02 and 07-03 were drilled near the northeast and northwest corners of the exisitng building, respectively. TH 07-01 was advanced into the till to a depth of 15.2 m below the existing ground surface. Test Holes 07-02 and 07-03 were advanced into the till to the depth of auger refusal at 22.1 and 18.4 m below the existing ground surface, respectively.

The soils observed during drilling of the test holes were visually classified on site by Mr. Stephen Peters, E.I.T. of UMA Engineering. Disturbed soil samples of auger cuttings were collected at regular intervals in each test hole. Standpipe piezometers fitted with Casagrande tips were installed in each test hole to facilitate measurement of groundwater levels in the till. Piezometer construction details are shown on the test hole logs in Appendix A.

Laboratory testing was conducted at UMA's Material Laboratory in Winnipeg and consisted of moisture content determination, plasticity tests (Atterberg limits), undrained shear strength, and grain size analysis.

A detailed test hole log has been prepared for each test hole to record the description and the relative position of the various soil strata, location of samples obtained, field and laboratory test results, and other pertinent information. Groundwater levels observed after drilling are also recorded on the test hole logs. The test hole logs and laboratory results are provided in Appendix A.

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SUBSURFACE CONDITIONS

The general soil profile in descending order is:

- Topsoil
- Sand Fill
- Clay Fill
- Clay
- Till

These soil units are described separately as follows:

Topsoil

Topsoil 0.2 m thick was encountered at the ground surface at TH 07-01. The topsoil consists of black silty clay and contains trace organics.

Sand Fill

Sand fill about 0.6 m thick was encountered at the ground surface at TH 07-02 and 07-03. The sand is brown and moist and contains some gravel and trace silt.

Clay Fill

Medium to highly plastic grey and brown silty clay was encountered below the topsoil and sand fill. The material is believed to be the backfill material used in the original construction in 1963/1964. The clay fill is generally firm to stiff, moist and contains trace organics, trace silt seams and trace gypsum inclusions (sulphates). The moisture content of the clay ranges between 39 and 22 percent and decreases with depth. At the time of the drilling the clay was frozen to a depth of about 1.8 m below existing ground surface. Undrained shear strengths as determined by unconfined compression test increase with depth in a range from 30 to 70 Kpa. The results of unconfined compression test were reasonably consistent with torvane and pocket pentrometer results. Field moisture contents are close to the plastic limit indicating the stiff nature of the material.

Clay

Low to medium plastic silty clay was encountered in TH07-02 underlying the clay fill at about 12 to 13m below the existing ground surface. The clay is grey, stiff to very stiff with average moisture contents of 13 percent.

Till

Till was encountered below the fill and clay in all the test holes at depths from 7.0 to 19.5m. In TH 07-01 the till consists of an upper unit of silt till about 3 m thick and lower clay till unit which extends to the bottom of the test holes. Clay till was encountered at the other two test holes. The silt till is brown, moist to dry and low to non plastic. The clay till is red, moist to dry and medium plastic. The till contains some sand (10 to 20%), trace gravel and trace gypsum. Moisture contents range from 10 to 25 percent. Sloughing was observed from the till in TH07-01 at a depth of 14.6m below the existing ground surface.

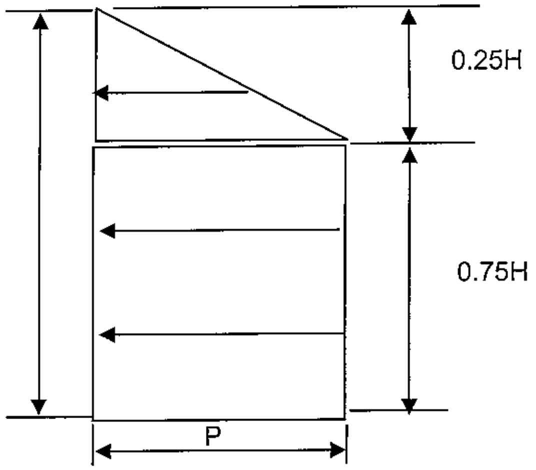
Groundwater

Seepage was not noted in the test holes during drilling. Standpipe piezometers fitted with Casagrande tips were installed in each test hole at completion of drilling to facilitate measurement of groundwater levels in till. The groundwater level was measured at 5.3 to 5.8 m below the ground surface on May 3, 2007. The measured groundwater levels were lower than would be regionally expected and most likely were affected by groundwater dewatering (pumping) for the construction at the West End Water Pollution Control Centre.

EXCAVATIONS

It is understood from the available reports that an open excavation with cut slopes of 1H: 1 V to a depth of up to 12m below the ground surface was used for construction of the existing building in the early 1960's. The excavation was backfilled using the excavated material once the underground structure had been completed. The location of the proposed stair wells is most likely within the limits of the original excavation and backfill. Considering the required excavation depth, and the fact that the excavation will be within previously backfilled material and the vicinity of the excavation from the existing building, a shored or partially shored excavation is considered to be the preferred excavation support method. A full depth unsupported open excavation is considered unsuitable and no further discussion is provided in this regard.

Cantilevered shoring in Winnipeg clays is limited to depths of about 4m. Beyond this depth, the shoring will have to be braced or tied back. In this regard, the earth pressure distribution shown in Figure 02 should be used for the design of shoring. Shoring is usually designed to keep movements around the perimeter of the excavation within acceptable limits. Avoidance of ground movements entirely is not possible. The amount of movement that will occur cannot be accurately predicted mainly because the movements are more a function of excavation procedures and workmanship than they are of theoretical considerations. Settlements of the ground surface adjacent to braced excavation are often estimated using the design chart developed by Peck (1969) as shown in Figure 03. It is recommended that the boundary between Zone I and II be used to estimate vertical ground movements at the site. It should be recognized that the predicted ground movements are associated with standard soldier piles and lagging or sheet piles with cross bracing or tie back anchors, assuming they are installed with a normal quality of workmanship.



P= γH – 4 m S_u *not less than* 0.3 γH,
where
γ = 18 kPa , m= 1.0, S_u= 40 kPa
H= depth of excavation in meter

Figure 02: Earth Pressure Distribution for Shored Excavations

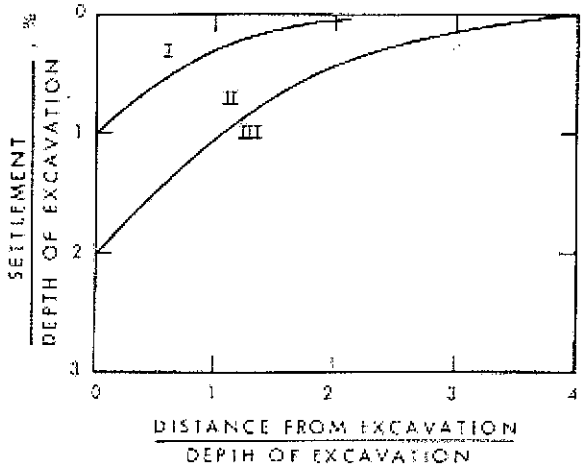


Figure 03: Ground Settlement estimate adjacent to excavation

If used, unsupported excavation shall be limited to the top 3m of the clay backfill and can be cut with back slopes not steeper than (1.5 H: 1 V). If soft zones or perched groundwater are encountered, flatter slopes may be required. The toe of the cut slope should be at least half the depth of the shored excavation from the shoring face.

A perimeter ditch should be provided to intercept surface runoff and/or any groundwater from entering the excavation.

The shallow clay/till contact encountered in TH 07-01 and the presence of stiff to very stiff clay in TH07-02 and 07-03 may impose installation difficulties for a sheet pile shoring system. Driving of soldier piles into the stiff clay and till units to the required embedment depth may generate ground vibration level intolerable to the existing building. Another possibility associated with the soldier pile driving is the potential for premature refusal before the required embedment depth is reached. Sacrificial soldier piles can be installed in pre-drilled holes to the required embedment depth and backfilled with normal strength concrete below the excavation line. Good contact between the lagging and retained soil should be maintained throughout the construction period. Free draining sand should be used to fill the voids behind the lagging. The soldier piles may be integrated structurally with the permanent stair well walls.

The potential of base instability and associated ground displacement adjacent to the excavation must be recognized in the design of deep excavations. The factor of safety against base instability should be determined using the equation:

$$F_{sb} = (N_b\, S_u) / \sigma_z$$

Where:

F_{sb} = Factor of Safety with respect to base instability
N_b = stability factor depending on the geometry of the excavation
S_u = Undrained shear strength of the clay below base level
σ_z = Total overburden pressure at base level

A minimum factor of safety of 1.50 is recommended for design purposes. For the proposed maximum excavation depth the factor of safety against base instability exceeds the design objective of 1.50, provided no surcharge is allowed within a distance equal to half the depth of the excavation from the shoring face. The factor of safety calculation was made based on undrained shear strength of 40 to 50 kpa and unit weight of 18 kN/m^3 for the clay. All excavations should be completed in accordance with Manitoba Workplace Health and Safety Regulations.

BASE HEAVE

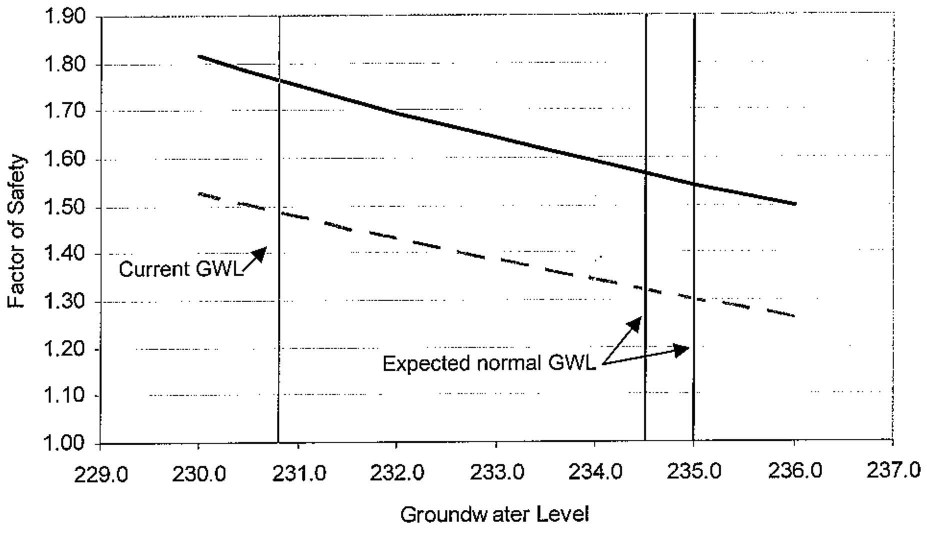
Where impervious layer(s) are underlain by a pervious layer under artesian pressure, the potential for base heave must be taken into consideration for both short and long term. Heaving of the impervious layer(s) and any infrastructure located within it can occur if the groundwater pressure at the contact is sufficient to overcome its weight.

The factor of safety (FS) against base heave is expressed as the ratio of the total stress (weight of soil) at the base of the impervious layer(s) to the groundwater pressure acting on the base. In this regard, a minimum factor of safety of 1.5 and 1.30 are recommended for long and short term design objectives. Long term conditions pertain to base heave potential for the completed structure and short term condition pertains to conditions during construction.

A site specific hydrogeological investigation was undertaken by UMA to assist in the assessment for the potential of seepage and base heave. The results of the investigation is presented in UMA report " Perimeter Road Pumping Station - 6281 Wilkies Avenue,Winnipeg,MB- Hydrogeological Assessment Report" date September 14, 2007. Based on the hydrogeological assessment report, the impervious layers (clay, silt and clay till and the Amaranth Formation of shales /siltstones) extend to a depth of 36.6m below the existing ground surface. Limestone bedrock with fractured and water bearing zones under artesian pressure underlies these layers. The hydrogeological assessment report indicated that while the current measured groundwater in bedrock aquifer is at about elevation 230.0 the regional groundwater level is likely to stabilize at elevation 234.5 to 235.0, once pumping operation at the West End Water Pollution Control Centre is terminated. The factor of safety against base heave corresponding to groundwater level in the bedrock aquifer for each design case was estimated using an average soil unit weight of 18 kN/m^3, the results are shown on Figure 04. For the current and expected normal groundwater levels the factor of safety exceeds or meets the design objective as shown in Figure 04.

Groundwater depressurization of the limestone aquifer will likely not be required, however groundwater monitoring is recommended before and during construction to confirm that groundwater levels do not exceed the assumed values in determination of the factor of safety against base heave.

Figure 04 : Factor of Safety Against Base heave



During construction, the potential for groundwater flow along existing vertical fractures in the clay cannot be ruled out even if the base of the excavation does not heave. Should such fractures exist, groundwater flow could occur into the excavation. In this circumstance, groundwater flow may start slowly and gradually increase over time due to erosion along the fractures. Should it occurs, it is expected that the seepage be at a rate which can be handled by conventional construction dewatering equipment.

FOUNDATIONS

North Side Stairwell

A floating foundation can be used to support the north side stairwell. Foundations placed at depths where the structural weight equals the weight of displaced soil usually assures adequate bearing capacity and only recompression settlement. Where the structural loads are less than the weight of the excavated soil the foundation is considered to be overcompensated. The depth of the proposed stairwells at the north end of the building is about 13 m below the existing ground and the unloading due to excavation is estimated to be 234 kPa. The pressure from the structure (dead load only) on the foundation soil is estimated to be 79 kpa, thus the foundation is overcompensated. At the foundation level there will be a maximum unloading of 155 kPa.

Soil displacement and upward rebound at the foundation level is expected as a result for stress relief due to excavation unloading. The rebound movement is excpected to be partially restrained by the weight of the structure, integrity of the stair well with the existing building and side friction along the stair well walls and backfill. Theoretically the rebound will continue up to a point where the stress at the foundation level is equal to the in-situ overburden pressure before the excavation. In this regard, the base of the stair well should be designed to resist a uniform upward pressure of 130 kpa. The influence of the material surrounding the excavation on the distribution of stresses and displacementes within the medium below the excavation made the prediction of soil rebound a complex soil-structure interaction case. However, it is expected that upward movement in the order of 25 mm may occur during mobilization of sufficient resistance to restrain further soil rebound. The following recommendations should be incorporated in the design and construction of the foundation:

1. Footing should not be placed on loose, disturbed or fill soil.

2. The foundation should be suitably designed to act as a rigid foundation.

3. Care should be taken during excavation to ensure that the bearing surface is not disturbed or subjected to freezing, water inundation or excessive drying. All loosened or disturbed soils should be removed from the final bearing surface.

4. Care should be taken during excavation to limit the loss of ground from underneath the footings of the existing adjacent building.

5. Once the bearing surface has been suitably prepared, it should be evaluated by qualified geotechnical personnel to verify the suitability of the proposed bearing soils, confirm that the soils are uniform, not affected by frost or disturbance and to confirm that the soils encountered are consistent with the conditions noted in this report.

6. As soon as possible, following approval of the bearing surface by qualified geotechnical personnel, the steel reinforcing should be placed and concrete shall be poured.

South Side Stairwell

The depth of the proposed stair well on the south side is about 3.8m below the ground surface. The logs of TH 07-01 shows that clay fill extends up to 7.0m below the exisitng ground surface. The fill is not considered to be competent bearing strata and the stair well structre should not be founded on this unit. The use of driven piles to support the structure may adversly impact the exisitng building and utilities and it is not recommended without proper vibration monitoring. Cast-in-place concrete friction piles can be used to support the structure. An allowable unit skin friction resistance of 15 kpa can be used to estimate the pile's load carrying capacity. No skin friction resistance shall be accounted for the length of the pile within the fill (i.e., about 3.2 m below the underside of the stair well or 7.0 m below the ground surface). Further design and construction recommendations for cast-in-place concrete frictions piles are summarized below:

1. The contribution from end-bearing should be ignored.
2. The piles should be spaced a minimum of three pile diameters, measured center to center.
3. The weight of the embedded portion of the pile may be neglected in the design.
4. All piles should be provided with adequate steel reinforcement.
5. Concrete should be placed as soon as practical following the drilling of each pile.
6. Seepage and sloughing can be expected in pile holes, particularly during wetter times of the year. As such, steel sleeves should be made available on site and utilized as required during construction to maintain the pile holes in a clean dry state.
7. Void space shall be provided under the base slab of the stairwell to avoid pressure from soil rebound.

EARTH PRESSURE

Permanent stair well walls should be designed to resist at-rest lateral earth pressure derived on the basis of the following conventional relationship which produces triangular pressure distribution:

$$P=K_o\gamma D$$

Where:

P= Lateral earth pressure at depth D (kPa)
K_o= At-rest earth pressure coefficient = 0.70
γ = Soil/Backfill unit weight =18 (kPa)
D = depth from ground surface to point of pressure calculation (m)

Below the groundwater table, the hydrostatic water pressure must be added and the submerged or buoyant weight of soil/backfill can be used. In this regard, long term groundwater table can be assumed at 2.0 m below the existing ground surface.

Where required, backfill between the stair well walls and the excavated faces should consist of granular material. The backfill should be sufficiently compacted only to minimize settlement of the backfill itself. A clay cap of 1 m minimum thickness should be used at the ground surface to protect against ingress of surface water to the footing level.

Foundation Concrete

The degree of exposure of concrete in contact with soils to sulphate attack is classified in CSA-A23.1-M2004 (Concrete Materials and Methods of Concrete Construction) as moderate, severe or very severe. Based on significant data gathered through previous work in the Winnipeg area and in accordance with the Manitoba Building Code, the degree of exposure for soils in Winnipeg is commonly classified as severe. Accordingly, all concrete in contact with the soils should be made with sulphate resistant cement (CSA Type 50) in accordance with CSA-A23.1-M2004.

CONSTRUCTION TESTING AND MONITORING

The engineering design recommendations presented within this memorandum are based on the assumption that an adequate level of geotechnical monitoring will be provided during construction and that qualified contractors experienced in foundations and excavations will carry out the construction. An adequate level of geotechnical monitoring is considered to be regular monitoring of footing construction procedures and compaction testing for earthworks related to shallow foundations and excavations.

If the conditions encountered during construction are different from the conditions reported in this report or the conditions upon which the recommendations in this report are made, our office shall be contacted so that our recommendations can be reviewed and modified if required.

Respectfully submitted,

UMA Engineering Ltd.

Faris Khalil, M.Sc., P.Eng.
Senior Geotechnical Engineer
Earth and Water

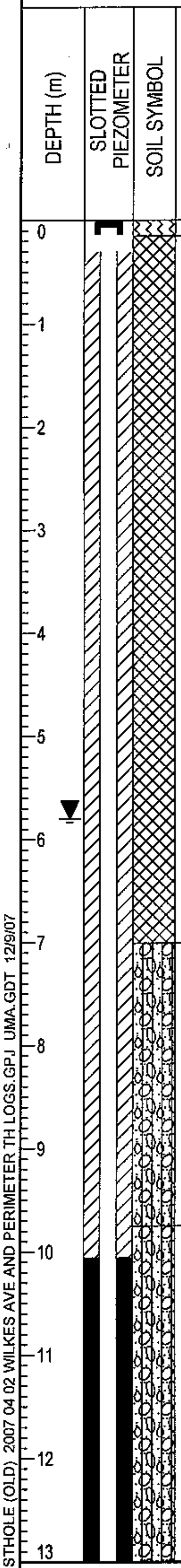
Reviewed by

Ken Skaftfeld, P.Eng.
Senior Geotechnical Engineer
Earth and Water

Appendix A
Test Holes Log

PROJECT: PRPS Reliability - PRPS Upgrades	CLIENT: City of Winnipeg	TESTHOLE NO: **TH07-1**
LOCATION: South Side of Wilkes Pumping Station N 5,522,058.9 E 620,794.5		PROJECT NO.: D265 202 02
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker SS2, 125 mm dia. SSA	ELEVATION (m): 238.353

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▨ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND



SOIL DESCRIPTION

TOPSOIL - trace organics (rootlets), black

CLAY (Fill) - silty
- light brown
- stiff, moist
- medium to high plasticity

- trace sand, trace gravel, firm below 1.5 m

- brown, firm to stiff, trace silt pockets below 3.0 m

- trace till inclusions

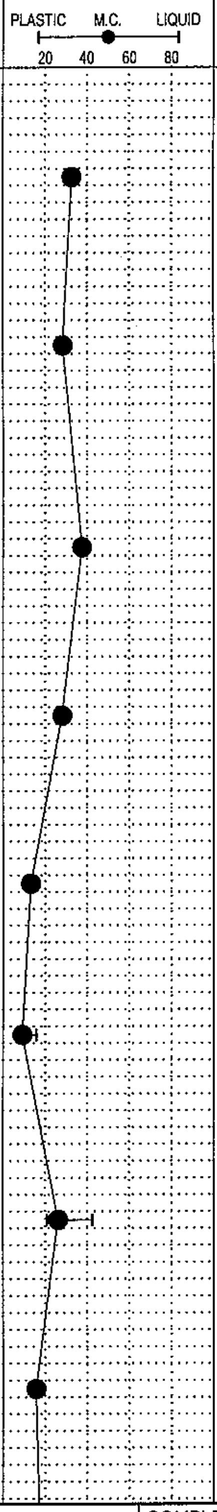
SILT (Till) - some clay, some sand, trace gravel
- brown
- moist
- low to non plastic

- dry below 8.2 m

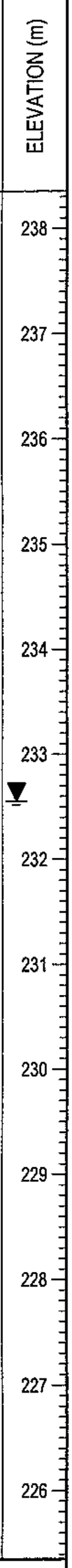
CLAY (Till) - silty, some sand, trace gravel
- red
- dry to moist
- trace of gypsum, white fine grained
- medium plasticity

- light brown/red, dry below 12.8 m

SAMPLE TYPE	SAMPLE #
■	G129
■	G130
■	G131
■	G132
■	G133
■	G134
■	G135
■	G136



COMMENTS



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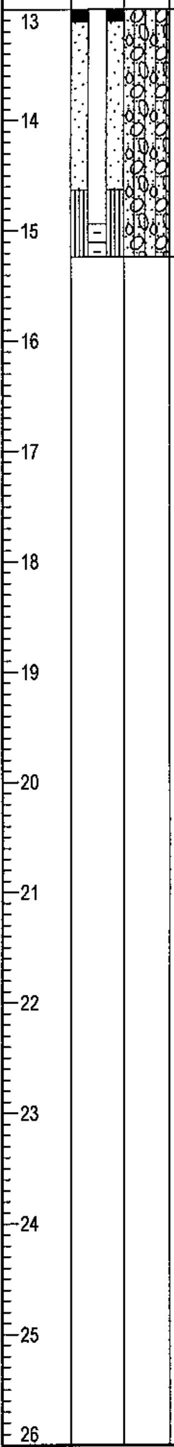
LOGGED BY: S. Peters	COMPLETION DEPTH: 15.24 m
REVIEWED BY:	COMPLETION DATE: 18/4/07
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

PROJECT: PRPS Reliability - PRPS Upgrades	CLIENT: City of Winnipeg	TESTHOLE NO: **TH07-1**
LOCATION: South Side of Wilkes Pumping Station N 5,522,058.9 E 620,794.5		PROJECT NO.: D265 202 02
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker SS2, 125 mm dia. SSA	ELEVATION (m): 238.353

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▨ NO RECOVERY	▌CORE
BACKFILL TYPE	■ BENTONITE	⁖ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁖ SAND

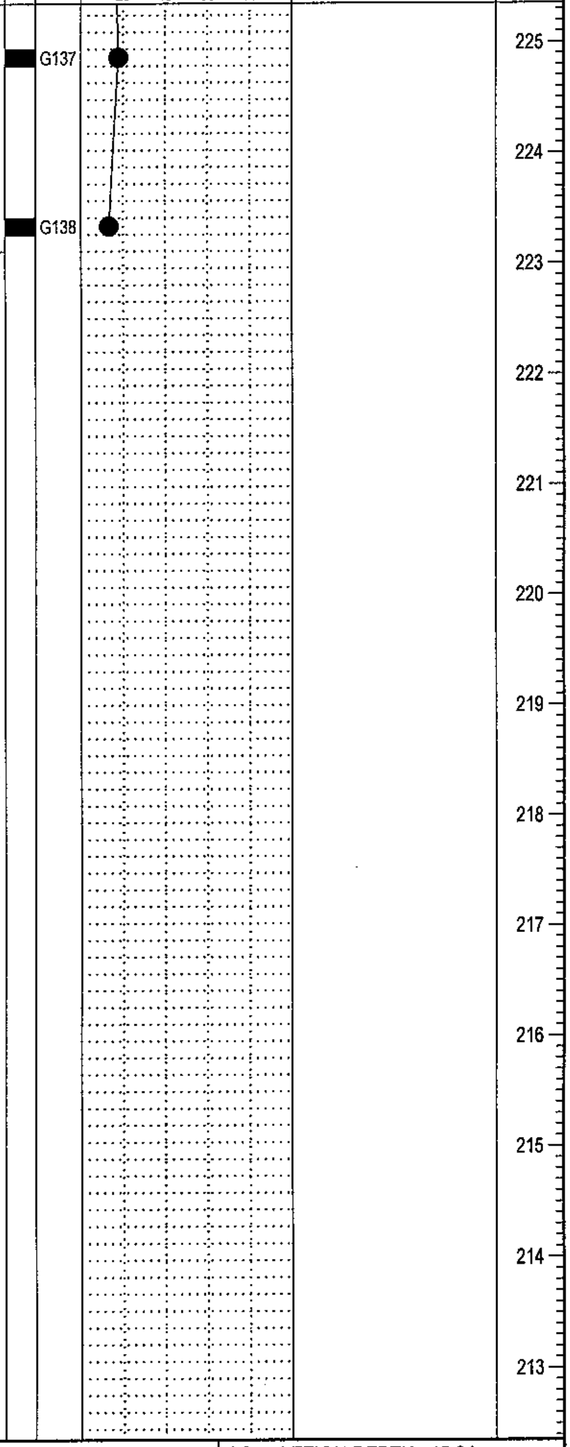
DEPTH (m)	SLOTTED PIEZOMETER	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	PLASTIC M.C. LIQUID 20 40 60 80	COMMENTS	ELEVATION (m)



- dry to moist below 13.7 m

END OF TEST HOLE AT 15.2 m

Notes:
1. Slougthing at 14.6m.
2. No seepage observed.
3. Installed standpipe at 15.2 m, with flush mount cover.
4. Backfilled test hole with sand to 13.1 m, bentonite chips to 10.1 m and auger cuttings to surface.
5. Water level at 5.80 m below ground surface on May 1, 2007.



LOGGED BY: S. Peters	COMPLETION DEPTH: 15.24 m
REVIEWED BY:	COMPLETION DATE: 18/4/07
PROJECT ENGINEER: Faris Khalil	Page 2 of 2

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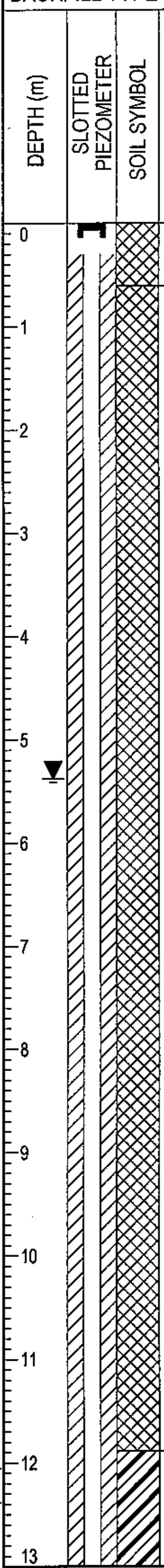
PROJECT: PRPS Reliability - PRPS Upgrades	CLIENT: City of Winnipeg	TESTHOLE NO: **TH07-2**
LOCATION: North East corner of Wilkes Pumping Station N 5,522,087.2 E 620,804.7		PROJECT NO.: D265 202 02
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker MP5-T, 125 mm dia. SSA	ELEVATION (m): 238.305

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◫ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



SOIL DESCRIPTION

SAND (Fill) - some gravel, trace silt
- brown, moist

CLAY (Fill) - silty, trace gravel
- black
- frozen to ~1.8 m
- low to medium plasticity
- organic odour

- firm to stiff, moist, below 1.8 m

- some silt inclusions, trace organics, grey, medium to high plasticity, organic odour below 2.4 m

- firm below 4.6 m

- trace sand, dry to moist, silt seams (~1mm thick)

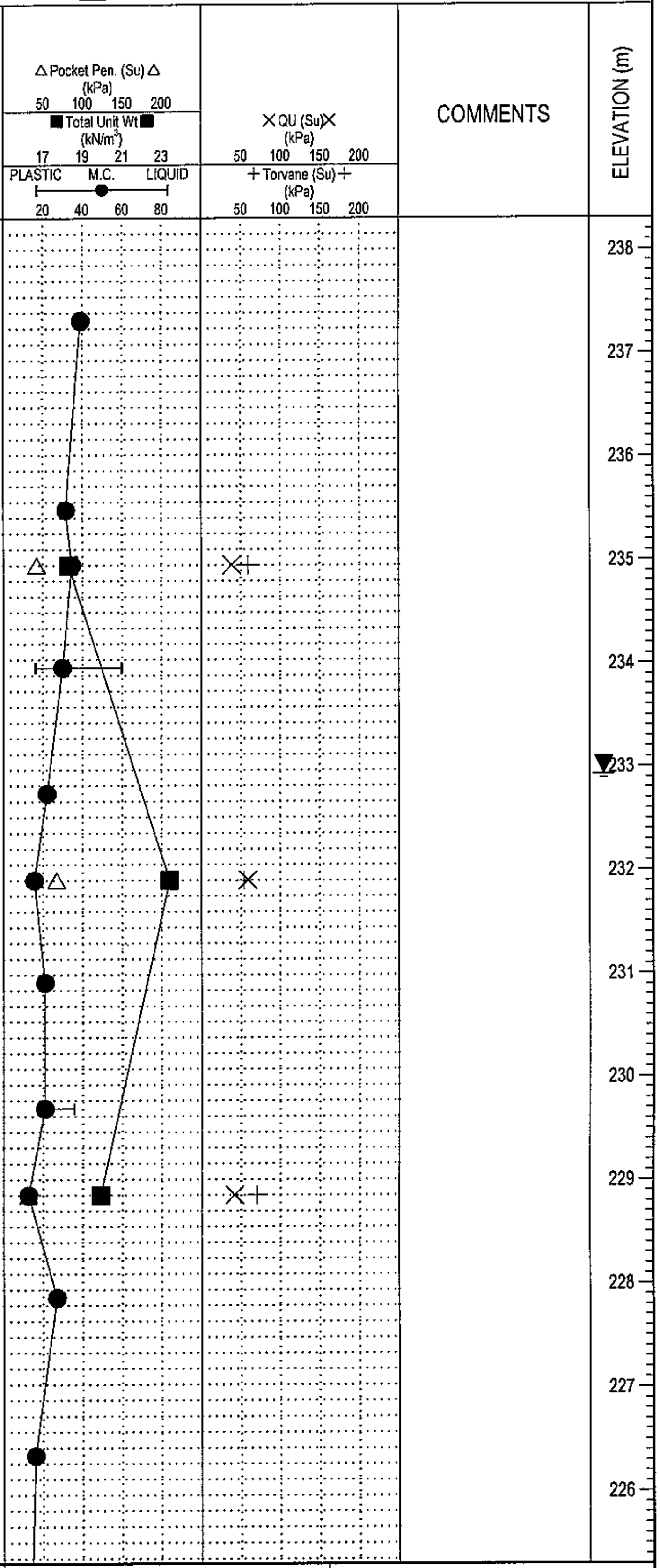
- trace organics, grey to black, some red, firm to stiff, medium to high plasticity
- organic odour

- red below 7.9 m
- trace gypsum, white fine grained

- stiff to very stiff below 11.0 m
- dry to moist

CLAY - silty, trace sand, trace fine gravel
- brown
- stiff to very stiff, dry
- low to medium plasticity

SAMPLE TYPE	SAMPLE #
■	G113
■	G114
▥	T115
■	G116
■	G117
▥	T118
■	G119
■	G120
▥	T121
■	G122
■	G123



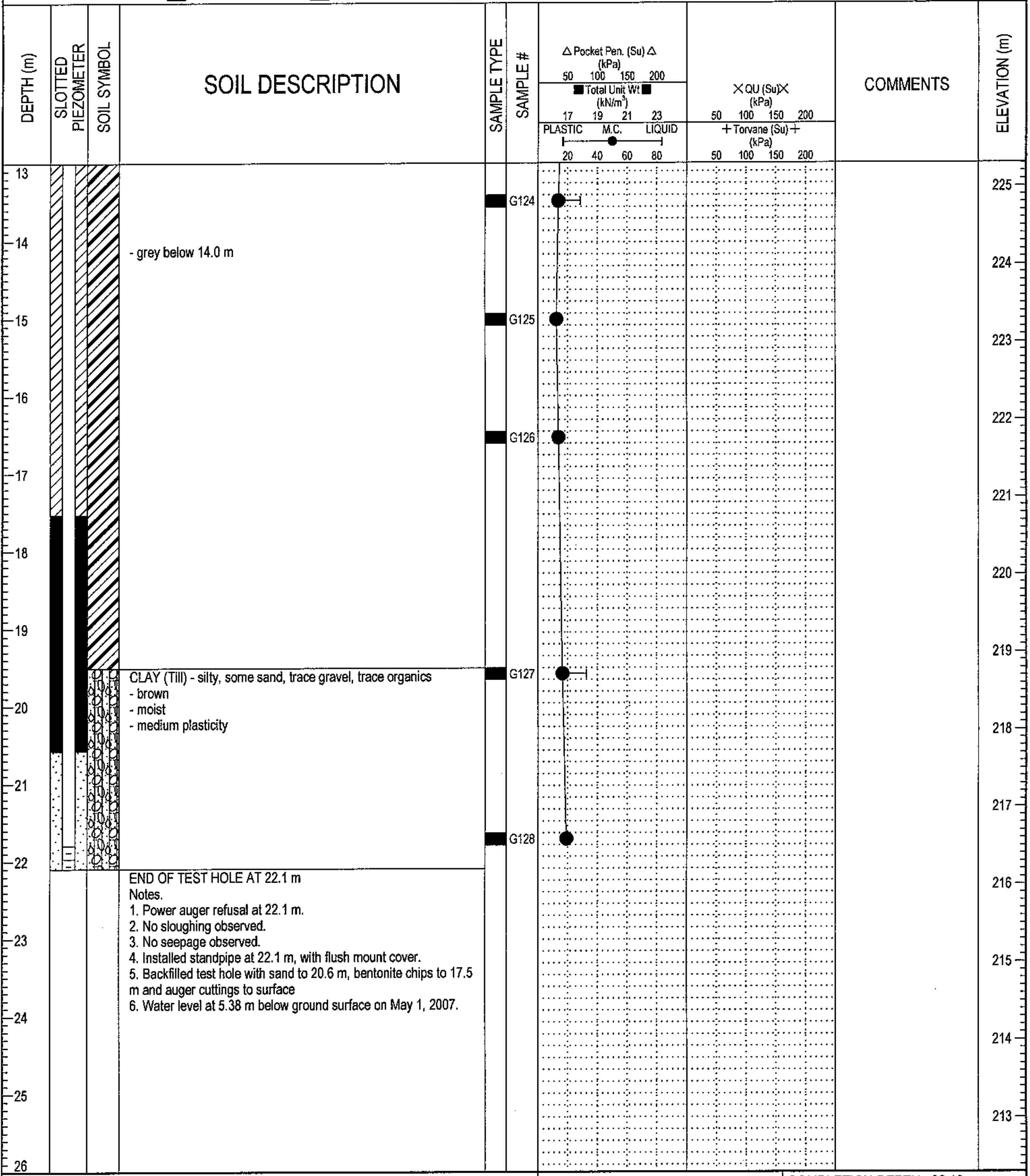
UMA | AECOM	LOGGED BY: S. Peters	COMPLETION DEPTH: 22.10 m
	REVIEWED BY:	COMPLETION DATE: 16/4/07
	PROJECT ENGINEER: Faris Khalil	Page 1 of 2

LOG OF TESTHOLE (OLD) 2007 04 02 WILKES AVE AND PERIMETER TH LOGS.GPJ UMA.GDT 12/9/07

PROJECT: PRPS Reliability - PRPS Upgrades	CLIENT: City of Winnipeg	TESTHOLE NO: **TH07-2**
LOCATION: North East corner of Wilkes Pumping Station N 5,522,087.2 E 620,804.7		PROJECT NO.: D265 202 02
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker MP5-T, 125 mm dia. SSA	ELEVATION (m): 238.305

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



- grey below 14.0 m

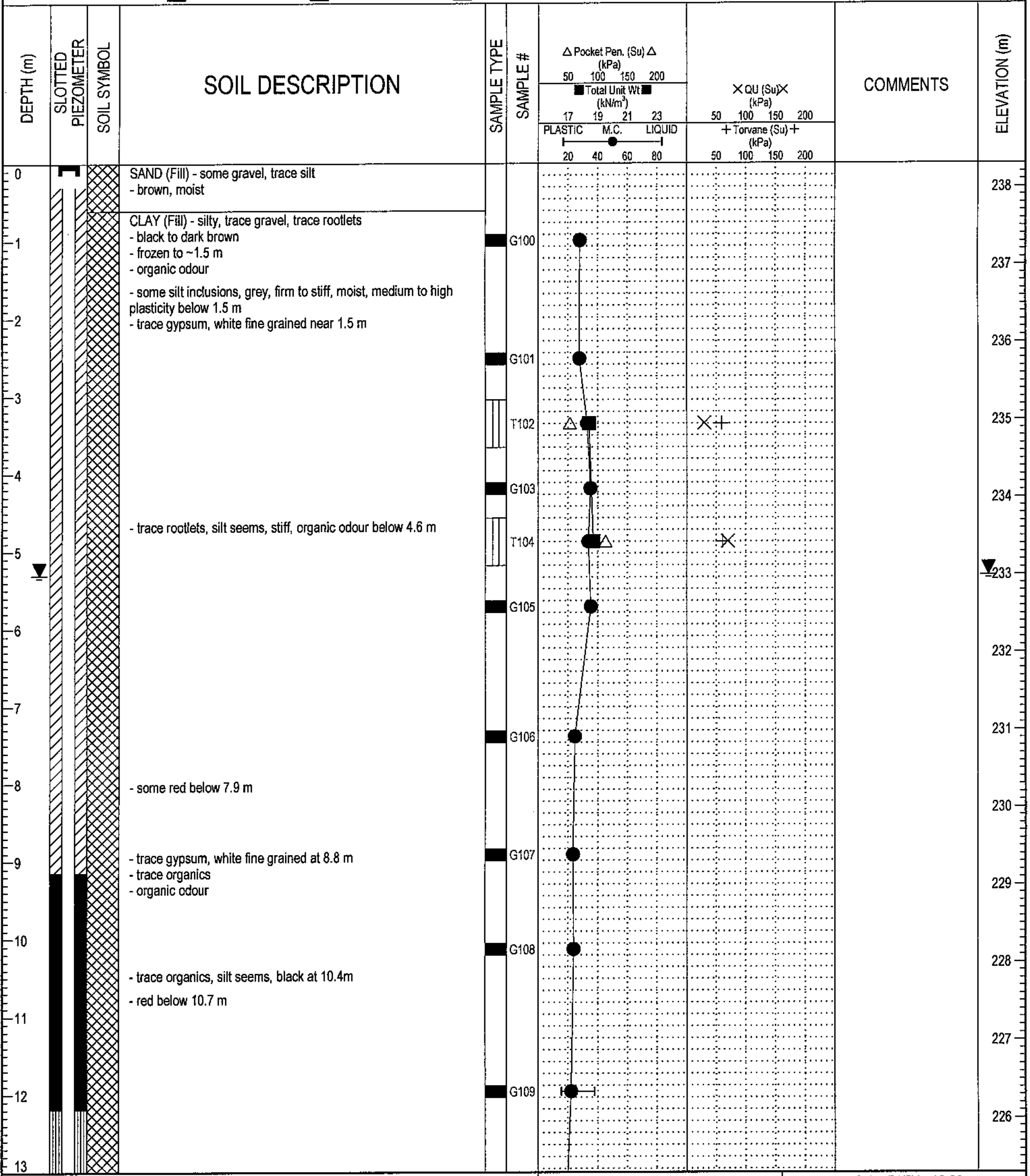
CLAY (Till) - silty, some sand, trace gravel, trace organics
- brown
- moist
- medium plasticity

END OF TEST HOLE AT 22.1 m
Notes.
1. Power auger refusal at 22.1 m.
2. No sloughing observed.
3. No seepage observed.
4. Installed standpipe at 22.1 m, with flush mount cover.
5. Backfilled test hole with sand to 20.6 m, bentonite chips to 17.5 m and auger cuttings to surface
6. Water level at 5.38 m below ground surface on May 1, 2007.

UMA | AECOM	LOGGED BY: S. Peters	COMPLETION DEPTH: 22.10 m
	REVIEWED BY:	COMPLETION DATE: 16/4/07
	PROJECT ENGINEER: Faris Khalil	Page 2 of 2

PROJECT: PRPS Reliability - PRPS Upgrades
CLIENT: City of Winnipeg
TESTHOLE NO: **TH07-3**
LOCATION: North West corner of Wilkes Pumping Station N 5,522,085.4 E 620,783.2
PROJECT NO.: D265 202 02
CONTRACTOR: Paddock Drilling Ltd.
METHOD: Acker MP5-T, 125 mm dia. SSA
ELEVATION (m): 238.304

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	◪ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND



SAND (Fill) - some gravel, trace silt
- brown, moist

CLAY (Fill) - silty, trace gravel, trace rootlets
- black to dark brown
- frozen to ~1.5 m
- organic odour
- some silt inclusions, grey, firm to stiff, moist, medium to high plasticity below 1.5 m
- trace gypsum, white fine grained near 1.5 m
- trace rootlets, silt seems, stiff, organic odour below 4.6 m
- some red below 7.9 m
- trace gypsum, white fine grained at 8.8 m
- trace organics
- organic odour
- trace organics, silt seems, black at 10.4m
- red below 10.7 m

UMA | AECOM

LOGGED BY: S. Peters
COMPLETION DEPTH: 18.39 m
REVIEWED BY:
COMPLETION DATE: 16/4/07
PROJECT ENGINEER: Faris Khalil

PROJECT: PRPS Reliability - PRPS Upgrades	CLIENT: City of Winnipeg	TESTHOLE NO: **TH07-3**
LOCATION: North West corner of Wilkes Pumping Station N 5,522,085.4 E 620,783.2		PROJECT NO.: D265 202 02
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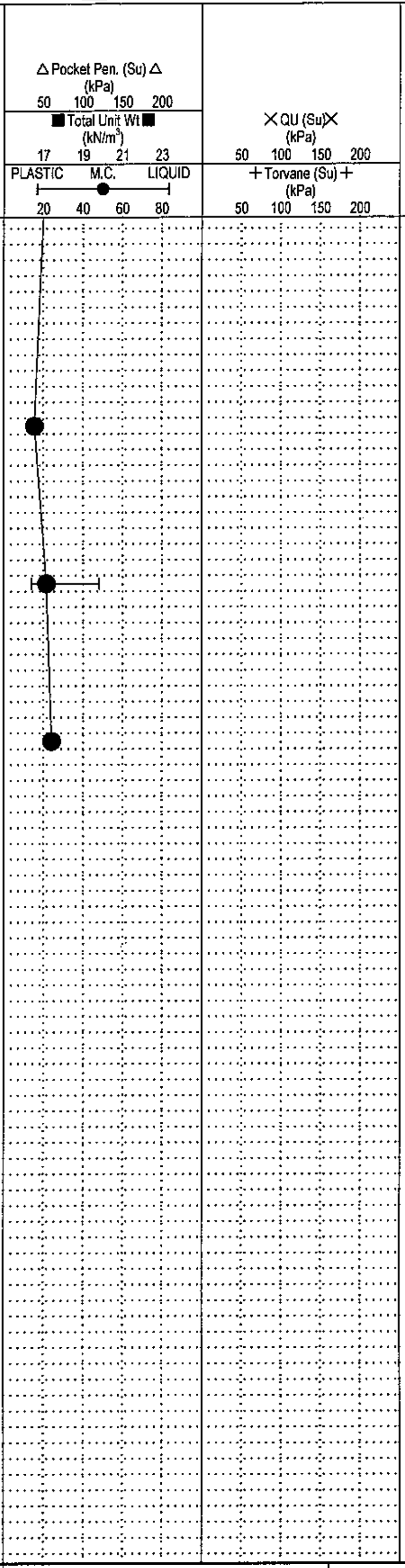
DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	COMMENTS	ELEVATION (m)
13					225
14	CLAY (Till) - silty, sandy, trace gravel - light grey - dense, dry to moist - trace gypsum, white fine grained				224
15		■	G110		223
16	- red, moist, compact to dense below 15.5 m	■	G111		222
17					221
18		■	G112		220
19	END OF TEST HOLE AT 18.4 m. Notes. 1. Power auger refusal at 18.4 m. 2. Squeezing near 12.2 m 3. Installed standpipe at 18.4 m, with flush mount cover. 4. Backfilled test hole with bentonite chips to 9.1 m and auger cuttings to surface. 5. Water level at 5.31 m below ground surface on May 1, 2007.				219
20					218
21					217
22					216
23					215
24					214
25					213
26					



LOGGED BY: S. Peters	COMPLETION DEPTH: 18.39 m
REVIEWED BY:	COMPLETION DATE: 16/4/07
PROJECT ENGINEER: Faris Khalil	Page 2 of 2

UMA | AECOM

ISS/REV: A
UMA FILE NAME: D265-202-02_01-B-F002_RX.dwg Saved By: stevensc PLOT: 07/09/13 9:15:26 AM B SIZE 11" x 17" (279.4mm x 431.8mm)

PLAN

0 5 10 m SCALE 1:500

City of Winnipeg
Perimeter Road Sewage Pumping Station Reliability
PRPS Upgrades

Test Holes Location Plan

UMA | AECOM

Figure - 01