Appendix A: Geotechnical Report

DYREGROV ROBINSON INC.
Consulting Geotechnical Engineers

MARKET LANDS MXU BUILDING – SOUTH PARCEL

GEOTECHNICAL INVESTIGATION

PRINCESS STREET

WINNIPEG, MANITOBA

Prepared for:

Market Lands Inc.
Winnipeg, Manitoba

September 2021 **File #214522**

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September 2, 2021

File No. 214522

Market Lands Inc.
c/o UWCRC 2.0 Inc.
515 Portage Avenue
Winnipeg, MB
R3B 2E9

Attn: Jeremy Read, President

RE: Market Lands MXU Building South Parcel – Geotechnical Investigation

Dyregrov Robinson Inc. is pleased to submit our final report for the geotechnical investigation that has been completed for the proposed mixed-use (MXU) building on Princess Street in Winnipeg, Manitoba.

If we can be of further assistance, please contact the undersigned directly.

Sincerely,

DYREGROV ROBINSON INC.

per Gil Robinson

Gil Robinson, M.Sc., P.Eng
President / Senior Geotechnical Engineer

Distribution List

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3	Yes	Market Lands Inc.

Dyregrov Robinson Inc.

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No. 86 **Date:** 2 Sept 2021

PROVINCE OF MANITOBA

GR

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ROBINSON

2 Sept 2021

REGISTERED PROFESSIONAL ENGINEER

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1.0 INTRODUCTION

As requested, Dyregrov Robinson Inc. (DRI), has undertaken a geotechnical investigation for the proposed mixed-use (MXU) building to be constructed on the south parcel of the Market Lands development located on the property at 151 to 171 Princess Street in Winnipeg, Manitoba. The purpose of the investigation was to evaluate the subsurface conditions to provide limit state design recommendations for foundations along with recommendations for other geotechnically related aspects of the development such as floor slabs and pavement design recommendations. Authorization to proceed with the investigation was provided by Jeremy Read, President of Market Lands Inc. on July 13, 2021.

2.0 PROPOSED DEVELOPMENT

It is understood that the proposed mixed use development will include a 10 storey building (no basement level) with a footprint of 13,000 ft^2 (approx.). The foundation loads are expected to be large enough to warrant consideration for driven end bearing piles. Some new surface pavements may also be provided.

3.0 SITE CONDITIONS AND FORMER BUILDINGS

The site is located on the vacant property between Princess Street and King Street directly north of William Avenue. The site was previously developed (i.e. Public Safety Building and the Civic Centre parkade) and the buildings were recently demolished. The excavations were backfilled and the site now has a gravel surface that is sloped towards the perimeter of the site.

We understand that the original buildings (i.e. Public Safety Building and the Civic Centre Parkade) had full basements and were supported on pile foundations, which we understand were cast-in-place concrete caissons end bearing on glacial till. A site survey drawing, prepared by Barnes and Duncan (provided to DRI by Market Lands Inc.), is attached in Appendix D for information purposes only. The drawing and indicates that a large number of the original caisson foundations are still in place. The drawing identifies the various foundation locations that were surveyed at the time of demolition but there are no details of the caisson sizes. The caissons generally appear to be on a grid spacing of approximately 3 to 4 m. *Although not available at the time of this report, the original foundation drawings should be searched for to determine the size of the caissons that were to be installed. It is expected that some of the caissons had belled bases that could be on the order of 1 to 2 m in diameter and could conflict with installation of new foundations.*

4.0 FIELD INVESTIGATION

Six test holes were drilled between August 3 and 5, 2021 at the locations illustrated on Figure 1 and described on the test hole logs. The test hole locations were targeted to be approximately halfway between the abandoned foundations. Test Holes 1 and 2 were cored into bedrock and Test Holes 3 to 6 were terminated below the fill layer in natural clay. The test holes were drilled by Paddock Drilling Ltd. using an Acker MP8 drill rig equipped with 125 mm solid stem augers and in Test Holes 1 and 2, coring was performed using an HQ coring system with casing advancer tools.

General site supervision and geotechnical logging of the test holes was performed by DRI. Representative disturbed (auger cuttings, split barrel sampler) and undisturbed (Shelby tube) soil samples were collected. Continuous HQ core samples (65 mm diameter) of the bedrock were also recovered and placed in wooden core boxes. Standard Penetration Tests (SPT's) were performed in the glacial till by driving a split barrel sampler 450 mm into the base of the test hole using an automatic slide hammer weighing 63.5 kg and dropped from a height of 760 mm. The number of blows for every 150 mm of penetration was recorded. The SPT N values on the test hole logs are the number of hammer blows required to drive the split barrel 300 mm after the initial 150 mm of penetration. The test holes were backfilled with bentonite chips and auger cuttings. Excess auger cuttings were bagged and removed from the site.

The soil samples and bedrock cores were returned to our Soils Testing Laboratory for testing including visual classification and determination of moisture contents on all of the soil samples. Bulk densities and undrained shear strengths were measured from the Shelby tube samples. The bedrock cores were logged, and photographed, two samples were tested to measure the unconfined compressive strength. Three soil samples were submitted for testing to evaluate the soil chemistry properties.

The test hole logs are provided in Appendix A and include a description of the soil and bedrock encountered, laboratory testing results and comments on the subsurface conditions observed at the time of drilling. Appendix B includes a summary of the bedrock core samples (Table B1), bedrock strength testing (Table B2) and photographs of the bedrock cores (Figures B1 and B2).

5.0 SUBSURFACE CONDITIONS

The general stratigraphy encountered in the test holes from grade includes fill materials, silty clay, glacial silt till and bedrock. Refer to the test hole logs for additional information. A general description of the main stratigraphic units is provided below.

5.1 Fill Materials

A 250 to 350 mm thick layer of crushed limestone gravel was encountered at grade in the test holes. The crushed limestone gravel has a 19 mm down gradation.

A layer of silty clay fill was encountered beneath the crushed limestone gravel in each test hole. The silty clay fill ranges in thickness from approximately 3 to 5 m and contains traces of sand and gravel. It is generally brown and grey in color, moist and has a variable consistency (firm to stiff). The moisture content of the clay fill ranges from about 20 to 43 percent with an average of 30 percent. It is not known what level of effort was used to compact the clay fill, but it is suspected that it was compacted as engineered fill. The bulk unit weight of the clay fill is approximately 17.5 kN/m^3. One sample of the clay fill (Test Hole 1 - sample T3 was submitted for testing to determine soil chemistry properties including; chloride content, conductivity / resistivity, pH and sulphate (SO4) content. The results are summarized in Table 5.1 and the laboratory test report is provided in Appendix A.

In Test Hole 6, a 300 mm thick layer of sand and gravel fill was encountered below the clay fill at a depth of 2.7 m below grade. The sand and gravel fill is brown in color and moist with a loose compactness condition. The moisture content of the sand and gravel fill is around 11 percent.

5.2 Silty Clay

Lake Agassiz lacustrine silty clay was encountered beneath the fill material in all test holes at depths ranging from approximately 3 to 5 m below grade. The clay deposit is about 11 to 12.5 m thick and is mottled brown / grey to a depth of about 6 m and turns grey below this depth. It is moist with a stiff consistency becoming firm below 11 m. The clay contains trace gravel and trace till inclusions below 10 m. The moisture content of the clay ranges from around 32 to 51 percent with an average around 43 percent.

The undrained shear strength of the clay was measured using Torvane, penetrometer and unconfined compressive strength tests. The clay has undrained shear strengths ranging from about 40 to 70 kPa to a depth of 10 m. Below this depth, the undrained shear strengths range from about 35 to 50 kPa. The bulk unit weight of the clay is around 16.5 kN/m³.

One sample of the clay (Test Hole 1 - sample T8 was submitted for testing to determine soil chemistry properties including; chloride content, conductivity / resistivity, pH and sulphate (SO4) content. The results are summarized in Table 5.1 and the laboratory test report is provided in Appendix A.

5.3 Glacial Silt Till

Glacial silt till was encountered below the silty clay in Test Holes 1 and 2 at a depth of about 15.7 m (elevation 218 m+/-). The thickness of the till layer was about 7 m and 8 m in Test Holes 1 and 2, respectively. The glacial till deposit in the Winnipeg area is typically a heterogeneous mixture of sand, gravel, cobble and boulder size materials within a predominantly silt matrix that has a low but variable clay content.

The silt till encountered in the test holes typically contains traces of sand, gravel and cobbles. No boulders were recovered but should be expected within the till deposit. The till is grey in color, moist to wet and loose becoming dense to very dense below a depth of approximately 17 m. The moisture content of the till ranges from approximately 5 to 18 percent with an average around 11 percent. Four standard penetration tests (SPT) were successfully completed with SPT-N values of 58, 81, 119 and 129.

One sample of the till (Test Hole 1 - sample S14 was submitted for testing to determine soil chemistry properties including; chloride content, conductivity / resistivity, pH and sulphate (SO4) content. The results are summarized in Table 5.1 and the laboratory test report is provided in Appendix A.

Table 5.1 – Soil Chemistry Test Results

Test Hole	Sample		Material	Soluble Chloride (CL)	Soluble Conductivity	Resistivity @25C	Soluble pH	Soluble Sulphate (SO4)	
	ID#	Depth (m)		mg/L	dS/m	Ohm-m		mg/L	%
TH-1	T3	3	Clay Fill	320	6.1	1.7	7.78	3500	0.34
TH-1	T8	9	Clay	340	6.6	1.5	7.82	3900	0.36
TH-1	S14	16.8	Till	180	1.5	6.7	8.08	410	0.011

5.4 Bedrock

Test Holes 1 and 2 were cored into bedrock. Bedrock was encountered beneath the glacial silt till at a depth of 23.9 m (elevation 210 m+/-) in Test Hole 1 and 22.9 m (elevation 211 m+/-) in Test Hole 2 . The bedrock geology maps in this area of Winnipeg (Manitoba Geological Survey's Geologic Scientific Report GR2002-1) classify the bedrock as a dolomitic mudstone (i.e. dolomite) belonging to the Lower Fort Garry Member of the Red River Formation.

The colour of the bedrock is generally whitish to reddish grey in color and has horizontal and vertical joints with some evidence of water flow. The length of bedrock core recovered from each core run was typically greater than 93 percent of the cored length and the Rock Quality Designation (RQD) for the bedrock ranged from about 70 to 100 percent (average of 85 percent) indicating fair to excellent quality. The bedrock recovered from the test holes is strong to very strong with close to moderately close discontinuity spacing, the spacing becomes wide below 26 m in Test Hole 1. The bedrock has gapped to open joint apertures. Two samples of the bedrock were tested to measure the unconfined compressive strength: the sample from Test Hole 1 (core sample C19) had a strength of 142.5 MPa and from Test Hole 2 (core sample C43) the strength was 124.6 MPa. Appendix B includes a summary of the bedrock core samples (Table B1), the unconfined compressive strength test results (Table B2) and photographs of the bedrock cores (Figures B1 and B2).

5.5 Test Hole Stability and Groundwater Conditions

In Winnipeg, groundwater usually occurs in shallow perched water tables within fill layers and silt deposits that are quite permeable and underlain by the relatively impermeable Agassiz clays. A groundwater table is not apparent during drilling within the clay soil due to its low permeability. The water table within the clay layer is typically not of significance for design and construction of foundations.

No significant sloughing or seepage conditions were observed in Test Holes 1 and 2 prior to switching drilling methods, at a depth of 16.8 m, from augering to HQ coring with casing advancer.

No sloughing or seepage conditions were observed in the shallow test holes (Test Holes 3 to 6) and upon completion of drilling the test holes were open to their completion depths and they were dry.

In general, the water level in the limestone bedrock aquifer below the glacial till has been rising since the early 1970's. In the general area of the Manitoba Law Courts building on Kennedy Street, the bedrock water levels in the early 1970's were around elevation 222 to 223 m. The water levels have risen by about 3 m since that time to around elevation 225 to 226 m with local spikes approaching 227 m. The local spikes are assumed to be associated with spring freshet and flooding events. The rise in the bedrock aquifer levels has been attributed, by others, to the reduced demand for groundwater by industrial users in the greater Winnipeg area. The bedrock hydrographs from the Provincial Groundwater Monitoring well (ID# G05OJ021) at the Law Courts Building and at the well (ID# G05MJ042) on Vaughan Street north of Portage Avenue (YMCA Building) are provided in Appendix C for reference.

Groundwater conditions should be expected to vary seasonally, from year to year and possibly as a result of construction activities.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 Foundations

The subsurface conditions at this site are suitable for driven piles including precast prestressed concrete hexagonal (PPCH) piles bearing on dense to very dense glacial till and steel HP piles end bearing on bedrock. Relatively light loads (e.g. structural floor slabs) could possibly be supported on CIP concrete friction piles. The abandoned foundations are not recommended for re-use as these piles may have been damaged during demolition of the former buildings and there are no foundation installation records available, that we are aware of, to confirm the installed conditions.

The main consideration for the foundation design will be avoiding the abandoned pile foundations, it would be extremely difficult and costly to remove existing foundations to allow installation of new foundations. New foundations should be installed as far away as possible from former foundations, a minimum clear distance of approximately 1 m should be provided between new piles and the nearest abandoned foundation (this distance needs to take into consideration belled pile bases). *Although not available at the time of this report, the original foundation drawings should be searched for to determine the size of the caissons that were to be installed. It is expected that some of the caissons had belled bases that could be on the order of 1 to 2 m in diameter and could conflict with installation of new foundations.*

Driven precast prestressed hexagonal piles appear to be the more feasible foundation for this site provided the foundations can be positioned to avoid the abandoned foundations. Steel piles bearing on bedrock have a relatively high capacity compared to the PPCH piles however, due to the condition and thickness of glacial till at this site the piles will be long (splicing required) and the driving times to refusal on bedrock could be significant.

6.1.1 Cast-In-Place Concrete Friction Piles

The geotechnical resistance of cast-in-place concrete friction piles under axial compressive loading can be designed in accordance with the current Manitoba Building Code (i.e. MBC 2011) using the service limit state (SLS) shaft adhesion values provided in Table 6.1. For the ultimate limit state (ULS) case, the piles can be designed with the factored shaft adhesion values and the factored end bearing pressure provided in Table 6.1. A resistance factor of 0.4 was used to calculate the factored ULS design values. Higher resistance factors (i.e. 0.5 or 0.6) cannot be considered unless dynamic testing or static load testing of friction piles is performed in advance of construction. Under the SLS loads, pile settlements are expected to be approximately 6 mm with differential settlements between piles around 3 to 6 mm.

Table 6.1: Geotechnical Design Parameters – Axial Compressive Loading

Depth Below Existing Site Grade (m)	SLS Shaft Adhesion (kPa)	Factored ULS (ϕ = 0.4)	
		Shaft Adhesion (kPa)	End Bearing (kPa)
0 to 4 (see Note 1)	0	0	0
4 to 5	10.0	12.0	0
5 to 10	18.0	22.0	145
10 to 14	13.0	16.0	145

Note: When determining effective pile lengths, the upper 4 m of the pile shaft below existing site grade should be ignored to account for the presence of fill and silt layers and the potential for soil shrinkage away from the pile.

The geotechnical resistance of cast-in-place concrete friction piles under axial tensile (uplift) loading can be designed in accordance with the current Manitoba Building Code (i.e. MBC 2011) using the service limit state (SLS) shaft adhesion values provided in Table 6.2. For the ultimate limit state (ULS) case, the piles can be designed with the factored shaft adhesion values provided in Table 6.2. A resistance factor of 0.3 was used to calculate the factored ULS design values. A higher resistance factor (i.e. 0.4) cannot be considered unless load testing of friction piles is performed in advance of construction. Under the SLS loads, pile movements are expected to be approximately 6 mm with differential movements between piles around 3 to 6 mm.

Table 6.2: Geotechnical Design Parameters – Axial Tensile (Uplift) Loading

Depth Below Existing Site Grade (m)	Shaft Adhesion (kPa)	
	SLS	Factored ULS (ϕ = 0.3)
0 to 4 (see Note 1 Table 6.1)	0	0
4 to 5	10.0	9.0
5 to 10	18.0	16.5
10 to 14	13.0	12.0

The pile depth should be limited to 14 m below existing site grade to avoid drilling into softer clays and the glacial till layer.

Piles should have a minimum diameter of 400 mm, a minimum length of 8 m and a minimum spacing of 3 pile diameters on center between piles. Where this spacing cannot be achieved DRI should be contacted for additional input. Small pile groups (maximum of 3 piles) can be considered for moderately high column loads.

Concrete should be placed as soon as possible after each pile hole is completed. Temporary steel sleeves should be on site and used where sloughing/caving of the pile borings occur and/or if groundwater seepage is encountered.

Piles that are subjected to freezing conditions (e.g., piles outside the perimeter of a heated building) must be protected from potential frost heave effects by using minimum pile lengths of 8 m and installing full length reinforcement. For pile lengths greater than about 9 m, the length of reinforcing may not need to be full length and should be reviewed. The frost uplift load on a pile can be calculated using an adfreeze bond stress value of 65 kPa and a frost depth of 2.5 m below site grade. When evaluating the frost uplift resistance of a pile, a geotechnical resistance factor of 0.7 should be used and consideration should be given to using a load factor of 1.25. The use of flat lying rigid insulation, such as Styrofoam HI, can also be used to minimize frost penetration into the soil around the piles if the minimum pile length cannot be achieved. A greased, polyethylene wrapped sonotube could also be placed around the upper 1.8 m of the pile shaft to act as a bond breaker and provide additional protection against frost heave.

6.1.2 Driven Precast Prestressed Concrete Hexagonal Piles

Driven end bearing precast prestressed concrete hexagonal (PPCH) piles under axial compressive loading and driven to practical refusal into dense to very dense glacial silt till, or limestone bedrock if encountered, may be designed in accordance with the current Manitoba Building Code (MBC 2011) using the SLS and factored ULS pile capacities provided in Table 6.3. Under the SLS loads, pile settlements are expected to be approximately 6 mm with differential settlements between piles around 3 to 6 mm.

Table 6.3: PPCH Pile Geotechnical Capacities – Axial Compressive Loads

PPCH Pile Size (mm)	Pile Capacities SLS (kN)	Unfactored ULS (kN)	Factored ULS Capacities $\phi = 0.4$ (kN)	$\phi = 0.5$ (kN)	$\phi = 0.6$ (kN)
300	**450**	1200	480	**600**	720
350	**625**	1700	680	**850**	1020
400	**800**	2200	880	**1100**	1320

Based on DRI's local experience with driven PPCH piles in Winnipeg, we recommend that a resistance factor of 0.5 be used for design provided that dynamic testing with CAPWAP analysis is performed during foundation installation. If dynamic testing is not performed a resistance factor of 0.4 must be used for design. A resistance factor of 0.6 could be used if static load testing is performed in advance of construction and PDA testing is used during production piling to confirm the pile capacity has been achieved. Dynamic testing will provide data on the driving energy delivered to the pile and the driving stresses (tensile and compressive) in the piles. The CAPWAP analysis will utilize the data collected to provide a mobilized static pile resistance that can be compared to the unfactored and factored ULS pile capacities. Details for the dynamic testing program can be finalized once the foundation layout has been established. Approximately 3 percent of the piles should be tested during pile installation under restrike conditions, however; the number of piles to be tested will depend on the size of the building area as well as the number and sizes of piles to be installed. The piles to be tested will need at least 1.2 m of pile shaft above local grade around the pile to facilitate the testing.

Driven precast prestressed concrete hexagonal (PPCH) piles subjected to tensile loading can be designed in accordance with the current Manitoba Building Code (MBC 2011) using the service limit state (SLS) shaft adhesion values provided in Table 6.4. Under the SLS loads, pile uplift displacement is estimated to be 2 to 6 mm. For the ultimate limit state (ULS) case, the piles can be designed with the factored shaft adhesion values provided in Table 6.4. Resistance factors of 0.3 and 0.4 were used to calculate the factored ULS design value. A resistance factor of 0.4 could be considered if dynamic testing is performed during pile installation.

Table 6.4: PPCH Piles – Geotechnical Design Parameters for Axial Tensile (Uplift) Loads

Depth Below Existing Site Grade (*see note 1*) (m)	SLS Shaft Adhesion (kPa)	Factored ULS ($\phi = 0.3$) Shaft Adhesion (kPa)	Factored ULS ($\phi = 0.4$) Shaft Adhesion (kPa)
0 to 4	0	0	0
4 to 5	10.0	9.0	12.0
5 to 10	18.0	16.5	22.0
10 to 16	13.0	12.0	16.0

Note 1: When determining effective pile lengths, the upper 4 m of the pile shaft below existing site grade or the depth of prebore, whichever is deeper, should be ignored to account for the presence of fill materials, the potential for soil shrinkage away from the pile and the effects of any prebore holes that may be drilled. For the purpose of evaluating the uplift resistance, the pile tip depth below existing site grade has been limited to 16 m. The maximum prebore depth should be specified for piles with uplift loading conditions.

The piles can be driven with diesel pile hammers having a rated energy of not less than 40 kilojoules. Hydraulic drop hammers can also be used provided they have a rated energy not less than 19.5 kilojoules. The rated energy for hydraulic drop hammers is less than for diesel hammers due to the high efficiency of this type of pile hammer. The driving stresses (compressive and tensile) in the piles should not exceed the limits specified by the pile manufacturer.

The pile driving criteria should be confirmed once the type of pile hammer proposed for use on this project is provided. Conventionally, practical refusal has been defined as final penetration resistance sets of 5, 8 and 12 blows per 25 mm for the 300, 350 and 400 mm diameter pile sizes, respectively. At least three consecutive sets should be obtained for each pile. If followers are used, the final penetration resistance should be increased by 50 percent; that is, 8, 12 and 18 blows per 25 mm for the 300, 350 and 400 mm diameter pile sizes, respectively.

Pile spacing for these piles should not be less than 2.5 pile diameters, center to center. No reduction in individual pile capacity is necessary for reasons related to group action provided that pile heave is monitored, measures are undertaken to minimize pile heave (i.e., pre-boring) and re-driving is completed when pile heave greater than 6 mm is measured. Re-driving of all piles in groups or clusters should be specified along with the requirement to monitor for pile heave.

Construction practice in Winnipeg normally includes pre-boring at driven PPCH pile locations. The pre-bore holes are usually drilled to diameters that are 50 mm larger than the pile size and to depths of approximately 3 to 5 m. Pre-boring is effective in reducing pile heave and contributes positively to pile verticality. The maximum prebore depth should be specified for piles with uplift loading conditions.

The depth to practical refusal will likely vary across the site and may be deeper than interpreted from the test hole logs. Some piles may be driven out of alignment and/or damaged during driving if boulders are present in the glacial till.

6.1.3 Driven Steel HP Piles

Driven end bearing steel HP piles driven to refusal on/into competent limestone bedrock can also be considered for this site. The depth to practical refusal will likely vary across the site and may be deeper than interpreted from the test hole logs. Some piles may be driven out of alignment (e.g., rotated, out of plumb) and/or damaged during driving if boulders are present in the glacial till. The thickness of glacial till at this site is significant which increases the potential for piles to encounter boulders, which can result in pile damage and refusal before bedrock is reached.

Based on the depth and condition of the glacial till overlying bedrock, it is recommended that steel HP pile foundations be designed using steel HP piles manufactured with Grade 350W steel (f_y = 350 MPa) and a minimum flange / web thickness of 15 mm.

The geotechnical service limit state (SLS) bearing capacity for end bearing steel HP piles driven to practical refusal on/into competent bedrock can be taken as $0.3f_yA_{steel}$ provided the corresponding end bearing pressure of the pile does not exceed 18 MPa (pressure based on the nominal pile size - e.g., 310 x 310). Where the end bearing pressure for a pile section exceeds the maximum recommended end bearing pressure of 18 MPa the SLS load will need to be reduced or the next larger pile section can be considered. Under the SLS loads, pile settlements can be expected to be less than about 15 mm, including elastic compression of the pile. The majority of the settlement is expected to be due to elastic compression of the steel.

Based on our experience with dynamic testing of steel HP piles in the Winnipeg area the unfactored ultimate limit state (ULS) bearing capacity can be taken as 2.3 times the SLS bearing capacity calculated using the above recommendations. The Manitoba Building Code requires a resistance factor be applied to the ULS values to calculate the factored ultimate resistance. Based on our experience with dynamic testing of steel HP piles in the Winnipeg area we recommend that a resistance factor of 0.5 be used for design provided that dynamic testing with CAPWAP analysis is performed during foundation installation. A resistance factor of 0.6 could be used if static load testing is performed in advance of construction and PDA testing is used during production piling to confirm the pile capacity has been achieved. If dynamic testing with CAPWAP analysis is not performed a resistance factor of 0.4 must be used. Dynamic load testing will provide data on the driving energy delivered to the pile and the driving stresses (tensile and compressive) in the piles. The CAPWAP analysis will utilize the data collected to provide a mobilized static pile capacity that can be compared to the unfactored and factored ULS pile capacities. The details of the dynamic load testing program can be finalized once the foundation layout has been established. Approximately 3 percent of the piles should be tested during pile installation under restrike conditions; however, the number of piles to be tested will depend on the size of the building area and the number and sizes of piles to be installed. The piles to be tested will need at least 1.2 m of pile shaft above local grade around the pile to facilitate the testing.

For driven steel HP piles subjected to tensile loading, the geotechnical resistance can be calculated in accordance with the current Manitoba Building Code (MBC 2011) using the service limit state (SLS) shaft adhesion values provided in Table 6.5 below. Under the SLS loads, pile uplift displacement is estimated to be 2 to 6 mm. For the ultimate limit state (ULS) case, the piles can be designed with the factored shaft adhesion values provided in Table 6.5. The adhesion values should be applied to the square perimeter of the pile section. Resistance factors of 0.3 and 0.4 were used to calculate the factored ULS design values. A resistance factor of 0.4 could be considered if dynamic testing is performed during pile installation.

Table 6.5: Steel HP Piles – Geotechnical Design Parameters for Axial Tensile (Uplift) Loads

Depth Below Existing Site Grade (*see note 1*) (m)	**SLS Shaft Adhesion (kPa)**	**Factored ULS (ϕ = 0.3) Shaft Adhesion (kPa)**	**Factored ULS (ϕ = 0.4) Shaft Adhesion (kPa)**
0 to 4	0	0	0
4 to 5	10.0	9.0	12.0
5 to 10	18.0	16.5	22.0
10 to 16	13.0	12.0	16.0
16 to 22	25.0	22.0	30.0

Note 1: Apply the shaft adhesion values to the square perimeter of the HP section. When determining effective pile lengths, the upper 4 m of the pile shaft below existing site grade should be ignored to account for the presence of fill materials, the potential for soil shrinkage away from the pile. For the purpose of evaluating the uplift resistance, the pile tip depth below existing site grade has been limited to 22 m.

Selection of pile sizes should take into consideration the pile sections and weights that are readily available. The number of different pile sections, particularly for each section size (e.g., HP310), should be kept to a minimum to reduce the chance of a wrong pile section / size being installed.

Pile spacing within pile groups should be a minimum of 2.5 pile diameters measured center to center. No reduction in individual pile capacity is necessary for reasons related to group action provided that pile heave is monitored and redriving is completed when pile heave greater than 6 mm is measured. Redriving of all piles in groups or clusters should be specified along with the requirement to monitor for pile heave.

The piles should be fitted with a driving shoe (e.g., Hard Bite) to help protect the pile tip from damage and improve penetration of the pile through the glacial till which contains cobbles and boulders and reduce the potential for misalignment. The driving shoe should be flush with the outside faces of the pile.

The minimum rated energy of the pile hammer to be used for pile installation should be confirmed by wave equation analysis once the pile size(s) for the project have been established.

Practical pile refusal can generally be considered to be three consecutive sets of 15 blows per 25 mm of pile penetration however, the pile driving set criteria (i.e., blows per 25 mm) and driving energy for each pile size should be estimated in advance of pile driving using wave equation analysis once the pile sizes and pile driving system details are known. If followers are used, the final penetration sets should be increased by 25 percent (i.e., 20 blows per 25 mm of pile penetration). The pile driving set criteria should then be confirmed at the onset of pile driving, for each pile size, based on dynamic testing results, including CAPWAP analysis. Dynamic testing during pile driving provides information on the pile hammer performance, energy delivered to the pile, driving stresses in the pile and estimates of the mobilized pile capacity.

Driving stresses (compressive and tensile) in the pile should be limited to about 80 percent of the yield strength of the steel to reduce the potential for pile damage during hard driving.

6.1.4 Lateral Capacity of Foundations

The lateral loading resistance of caissons, single piles or groups of piles can be analyzed once the foundation sizes, layout and lateral loads have been established. This analysis can be performed using the software program Lpile which models the pile-soil reaction using the p-y curve method. For preliminary analysis, the horizontal modulus of subgrade reaction values in Table 6.6 can be used:

Table 6.6: Horizontal Modulus of Subgrade Reaction

Soil Type	Horizontal Modulus of Subgrade Reaction (kN/m^3)
Clay	25,000
Glacial Till	40,000

6.1.5 Foundation Installation Monitoring and Dynamic Testing

Based on Sub-Sections 4.2.2.3 Field Review and 4.2.2.4 Altered Subsurface Condition (ref: NBC 2010 Section 4.2 Foundations) and as the Geotechnical Engineering Consultant of record for this project we recommend that all deep foundations be monitored on a full-time basis by geotechnical personnel from DRI due to our familiarity with the subsurface conditions of the project site, foundation design considerations and the installation of major foundations.

If driven end bearing piles are to be installed, the dynamic testing and CAPWAP analysis should be performed by DRI. Approximately 3 percent of the piles should be tested during pile installation. The number of piles to be tested will depend on the size of the building area and the number and sizes of piles to be installed. The need for testing during initial driving and restrike conditions will need to be confirmed based on the foundation design and conditions encountered during production piling.

6.2 Pile Caps and Grade Beams

A void separation of at least 150 mm should be provided under grade beams and pile caps.

6.3 Below Grade Walls

Below grade walls (e.g. elevator pits) should be designed to resist lateral earth pressures that are derived on the basis of the following conventional relationship which produces a triangular pressure distribution:

$$P = K \gamma D$$

where P = lateral earth pressure at depth D (kPa)
K = at-rest earth pressure coefficient (0.5)
γ = soil/backfill unit weight 17.5 (kN/m^3)
D = depth from surface to point of pressure calculation (m)

A filter-protected positive drainage system should be provided at the base of the wall to prevent the build-up of hydrostatic pressure against the wall. The water should be directed to a sump pit and pumped away from the building. The walls can be backfilled with crushed limestone gravel (50 mm down gradation) or a sand and gravel material that is clean, free draining and well graded with a maximum particle size of 50 mm and 8 to 15 percent (maximum) passing the 75 μm sieve size. Other material types and gradations can be considered and reviewed. The backfill material should be compacted to 92 percent of the standard Proctor maximum dry density. A 600 mm (minimum) thick clay cap can be provided around the walls to reduce the potential for water infiltration into the granular backfill. The clay cap material should be compacted to 92 percent of the standard Proctor maximum dry density.

6.4 Floor Slabs

In areas where there is minimal tolerance for floor slab movement it is strongly recommended that structural floor slabs over a void space be considered as a preferred option due to the presence of high plastic (i.e., expansive) clay soils and fill materials at the site. Structural floor slabs will minimize the potential for movement of the floor slab due to volumetric changes (i.e., shrinkage / swelling) of the underlying clay soils and/or settlement of fill materials. It is possible that the total amount of heave and swelling could be as much as 100 to 150 mm in the long term. The potential for long term settlement of the fill is unknown and will be related to the compaction effort used to place the fill material. A void separation between the structural floor slab and underlying soil should be at least 150 mm thick. A vapour barrier should be provided below the floor slab.

Floor slabs on grade can be considered provided some movement of the floor slabs can be tolerated and the Owner and designer understand that placing floor slabs on fill materials comes with higher risk of vertical movement. Where a slab on grade floor is to be considered it must recognized that the floor slab will undergo some movements overtime due to volumetric changes (i.e., shrinkage / swelling) of the underlying clay soils and/or settlement of fill materials. Vertical movements on the order of 25 to 50 mm should be expected and in the longer term could reasonably be on the order of 100 to 150 mm. The movements will be differential and are not expected to be uniform across the floor slab. Factors impacting the magnitude of floor slab movements, which should be expected, are the level of effort used to place the fill materials and climatic effects during construction which might impact changes in the sub-soil moisture conditions. For these reasons, it is not possible to assess the amount of soil movement, which will occur, with any degree of accuracy.

If used, slab-on-grade floors could be isolated from fixed building components (e.g., grade beams) in an effort to allow for some floor slab movements to occur without affecting the structure. A vapour barrier should be provided below the floor slab. The floor slab should not be placed against frozen soil and should be supported on a minimum of 300 mm of compacted granular base material placed on a prepared subgrade consisting of compacted clay soil. The granular base should consist of a 19 mm down crushed limestone material compacted to 98 percent of the standard Proctor maximum dry density (SPMDD). For below grade floor slabs, a subfloor drainage system consisting of a perimeter weeping tile drain and interior lateral drains should be provided. The drains should be directed to a sump pit(s) for disposal. Refer to the Pavements section of this report for subgrade preparation requirements.

6.5 Pavements

Some vertical movements of the pavements are unavoidable and should be expected. Regular maintenance of the pavement surfaces (e.g., crack sealing) will help to provide longer life and serviceability of the surfaces.

The following pavement sections can be placed on a prepared subgrade consisting of high plastic silty clay soils. Assuming the clay backfill on the site was well compacted, it should serve as a suitable subgrade for the pavement. A non-woven geotextile should be placed on the prepared subgrade to provide separation between the clay subgrade and granular materials.

- Standard duty asphalt pavements can be designed using 75 mm of asphalt placed on 300 mm of granular base material or 100 mm of granular base and 200 mm of granular sub-base.
- Heavy duty asphalt pavements can be designed using 100 mm of asphalt placed on 150 mm of granular base material and 300 mm of granular sub-base material.
- For heavy duty traffic areas, such as refuse pick up areas, the pavement section should consist of 150 to 200 mm of reinforced concrete over 150 mm of granular base course and 200 mm of granular sub-base material. The reinforcing should be designed based on the anticipated loads.

The material selection and construction requirements should meet the City of Winnipeg's Standard Construction Specifications. The granular base should be a 19 mm down crushed limestone material and the granular sub-base material should be a 50 mm down crushed limestone that are placed over a uniformly prepared subgrade. The granular base and sub-base materials should be compacted to at least 98 percent of the standard Proctor maximum dry density (SPMDD). Where significantly more than 300 mm of sub-base material is required to achieve the design grades, a 100 mm or 150 mm down crushed limestone can be considered to build up the sub-base to the underside of the sub-base material recommended above. The crushed limestone granular materials should meet the requirements for 'Granular A' in the City of Winnipeg's Standard Construction Specifications, CW 3110-R21 for Sub-Grade, Sub-Base and Base Course Construction. The non-woven geotextile should meet the requirements of the City of Winnipeg's Standard Construction Specifications, CW 3130 for the Supply and Installation of Geotextile Fabrics.

Fill and deleterious materials should be stripped from the sub-grade surface prior to preparation. The clay sub-grade should be graded smooth and scarified to a depth of approximately 150 mm and then uniformly re-compacted to 95 percent of the standard Proctor maximum dry density (SPMDD) before the granular sub-base material is placed. The subgrade should be proof rolled with a fully loaded tandem gravel truck to check for weak / soft areas.

Areas identified as being weak or soft during subgrade preparation and/or proof rolling should be stabilized by additional re-working and compaction or removal and replacement with suitable material. If encountered, silt can be over excavated and replaced with suitable material (i.e., compacted clay or granular sub-base) or bridged with additional granular base and non-woven geotextile and geogrid to provide separation and reinforcement. The amount of additional granular base is expected to be 150 to 300 mm thick but should be determined at the time of construction to suit the conditions encountered. The geotextile should meet the requirements of the City of Winnipeg's Standard Construction Specifications, CW 3130 for the Supply and Installation of Geotextile Fabrics. The geogrid should meet the requirements of the City of Winnipeg's Standard Construction Specifications, CW 3135 for the Supply and Installation of Geogrid.

6.6 Excavations

All excavation work should be completed by the Contractor in accordance with the current Manitoba Workplace Health and Safety Regulations to suit the planned and expected construction activities and schedule. The earth pressure distribution shown on Figure 2 can be used for temporary braced shoring design. Local excavations in fill materials may need to be flatter than allowed in the Manitoba Workplace Health and Safety Regulations.

6.7 Seismic Design

Seismic loading is not required by the Manitoba Building Code which has adopted the National Building Code of Canada 2010, with an amendment to Sentence 4.1.8.4 (7). Sentence 7 was replaced with the following: 7) For the purposes of Sentence 4.1.8.1(1), the value of S_a (0.2) in Manitoba is deemed to be zero. (reference: The Buildings and Mobile Homes Act (C.C.S.M. c. B93), Manitoba Building Code Regulation 31/2011, Registered March 28, 2011).

6.8 Other

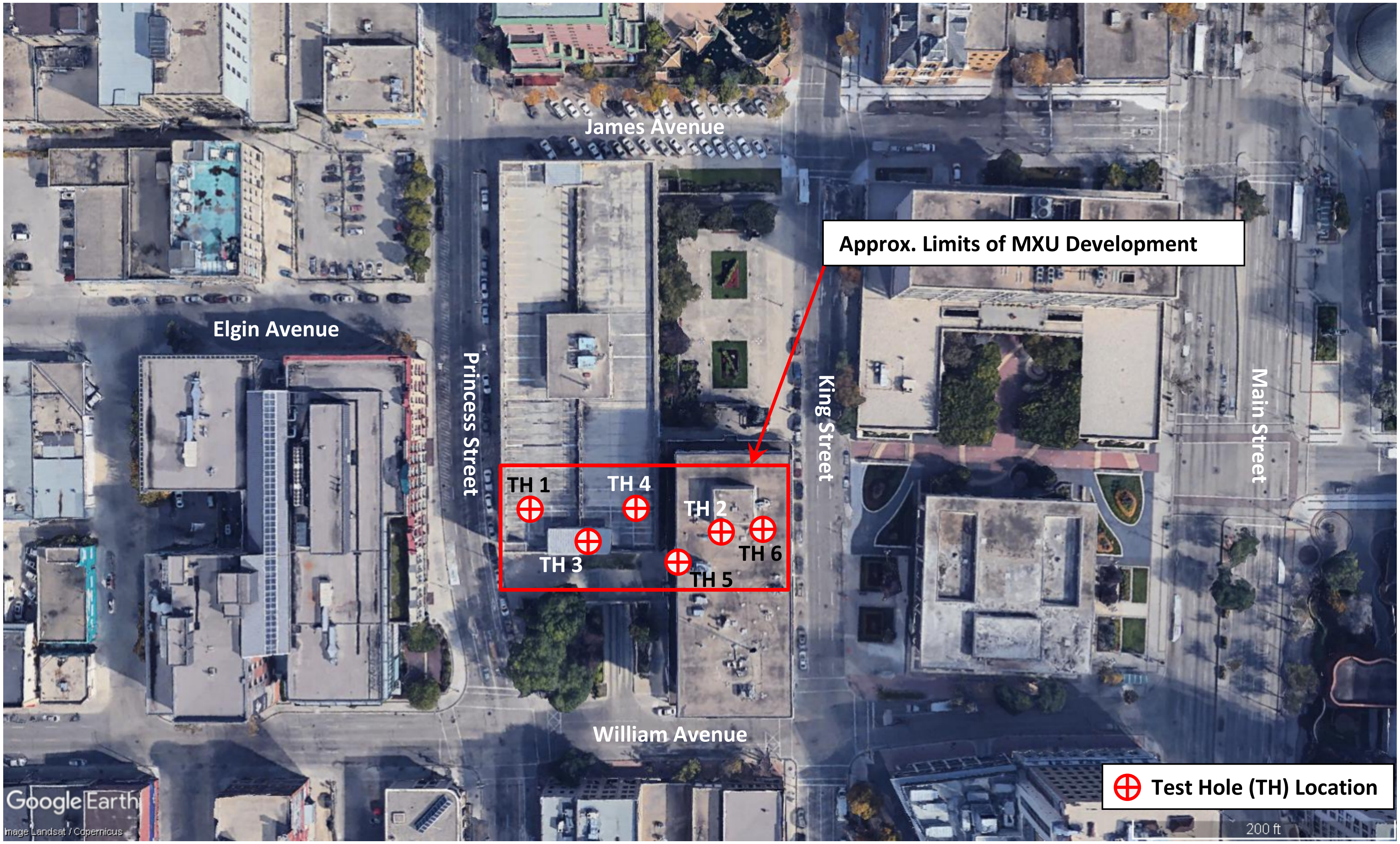
Positive drainage should be provided away from all structures at gradients of at least 2 percent.

The potential for sulphate attack in Winnipeg is considered to be severe (Exposure Class S-2). All concrete in contact with soil should be made with sulphate resistance cement (Type HS) in accordance with the Building Code and relevant CSA standards.

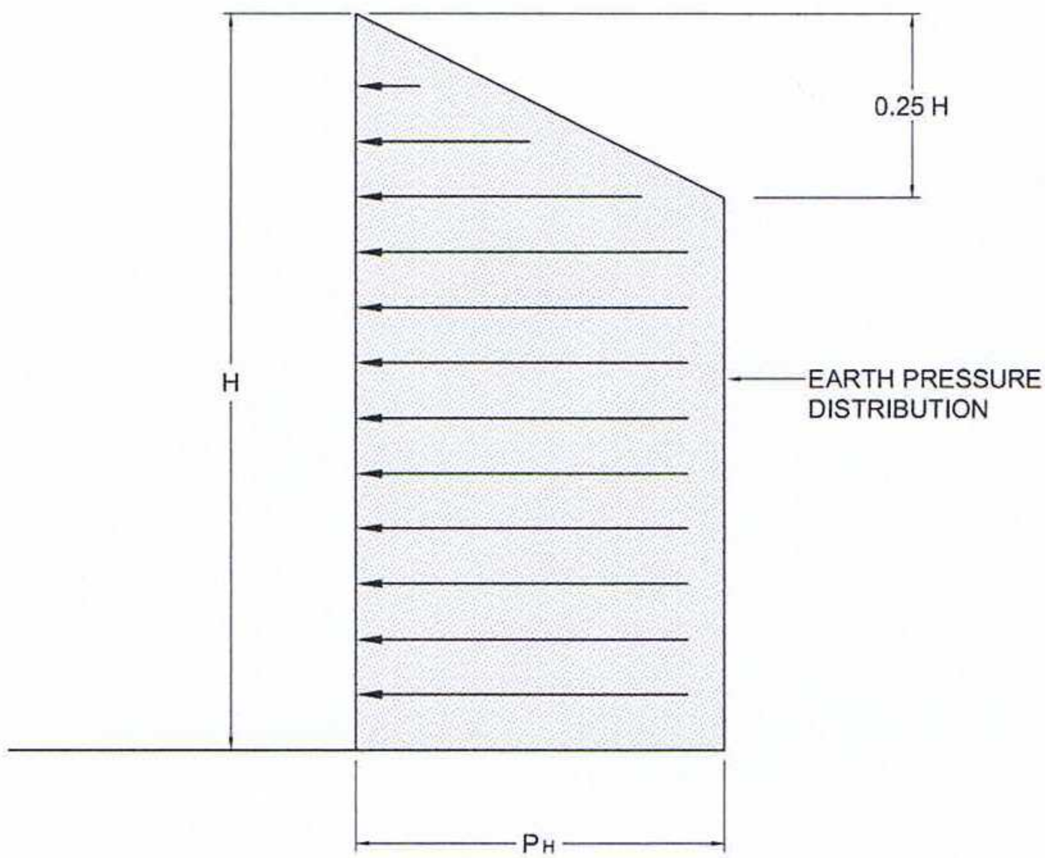
7.0 CLOSURE

This report and its findings were prepared based on the subsurface conditions encountered in the random representative sample of test holes drilled between August 3 and 5, 2021 for the sole purpose of this geotechnical investigation and our understanding of the proposed mixed-use development at the time of this report. Subsurface conditions are inherently variable and should be expected to vary across the site.

This report was prepared for the sole and exclusive use of Market Lands Inc. for the proposed mixed-use building to be constructed on the south parcel of the Market Lands development located on the property at 151 to 171 Princess Street in Winnipeg, Manitoba. The information and recommendations contained in this report are for the benefit of Market Lands Inc. only and no other party or entity shall have any claim against Dyregrov Robinson Inc., or the author, nor may this report be used for any other projects, including but not limited to changes in this proposed development without the consent of Dyregrov Robinson Inc. The findings and recommendations in this report have been prepared in accordance with generally accepted geotechnical engineering principles and practises. No other warranty, expressed or implied, is provided.



DYREGROV ROBINSON INC. CONSULTING GEOTECHNICAL ENGINEERS			Market Lands MXU South Parcel - Princess Street Winnipeg, MB Test Hole Location Plan (Pre-Demolition Image)		
SCALE: NTS	MADE BY: JW	CHKD BY: GR	PROJECT NO. 214522	DATE: August 2021	**Figure 1**



$Ph = 0.4 (\gamma H + q)$

Where:

Ph = Lateral Earth Pressure (kPa)

γ = Soil Unit Weight (17.3 kN/m^3)

H = Depth of Excavation (m)

q = surface surcharge load (kPa)

DYREGROV ROBINSON INC. CONSULTING GEOTECHNICAL ENGINEERS	**EARTH PRESSURE DISTRIBUTION TEMPORARY BRACED SHORING**

SCALE:	MADE BY:	CHKD BY:	PROJECT NO.	DATE:	
NTS	GR	AA	214522	August 2021	**FIGURE 2**

APPENDIX A

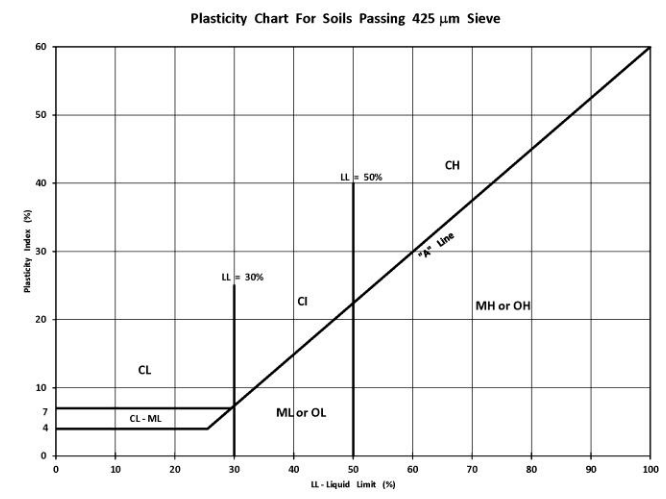
Test Hole Logs

Soil Chemistry Test Results

EXPLANATION OF TERMS & SYMBOLS

Description				TH Log Symbols	USCS Classification	Laboratory Classification Criteria			
						Fines (%)	Grading	Plasticity	Notes
COARSE GRAINED SOILS	GRAVELS (More than 50% of coarse fraction of gravel size)	CLEAN GRAVELS (Little or no fines)	Well graded gravels, sandy gravels, with little or no fines		GW	0-5	$C_U > 4$ $1 < C_C < 3$		
			Poorly graded gravels, sandy gravels, with little or no fines		GP	0-5	Not satisfying GW requirements		Dual symbols if 5-12% fines. Dual symbols if above “A” line and $4<W_P<7$
		DIRTY GRAVELS (With some fines)	Silty gravels, silty sandy gravels		GM	> 12		Atterberg limits below “A” line or $W_P<4$	
			Clayey gravels, clayey sandy gravels		GC	> 12		Atterberg limits above “A” line or $W_P<7$	
	SANDS (More than 50% of coarse fraction of sand size)	CLEAN SANDS (Little or no fines)	Well graded sands, gravelly sands, with little or no fines		SW	0-5	$C_U > 6$ $1 < C_C < 3$		$C_U = \frac{D_{60}}{D_{10}}$
			Poorly graded sands, gravelly sands, with little or no fines		SP	0-5	Not satisfying SW requirements		$C_C = \frac{(D_{30})^2}{D_{10} x D_{60}}$
		DIRTY SANDS (With some fines)	Silty sands, sand-silt mixtures		SM	> 12		Atterberg limits below “A” line or $W_P<4$	
			Clayey sands, sand-clay mixtures		SC	> 12		Atterberg limits above “A” line or $W_P<7$	
FINE GRAINED SOILS	SILTS (Below ‘A’ line negligible organic content)	$W_L<50$	Inorganic silts, silty or clayey fine sands, with slight plasticity		ML			Classification is Based upon Plasticity Chart	
		$W_L>50$	Inorganic silts of high plasticity		MH				
	CLAYS (Above ‘A’ line negligible organic content)	$W_L<30$	Inorganic clays, silty clays, sandy clays of low plasticity, lean clays		CL				
		$30<W_L<50$	Inorganic clays and silty clays of medium plasticity		CI				
		$W_L>50$	Inorganic clays of high plasticity, fat clays		CH				
	ORGANIC SILTS & CLAYS (Below ‘A’ line)	$W_L<50$	Organic silts and organic silty clays of low plasticity		OL				
		$W_L>50$	Organic clays of high plasticity		OH				
HIGHLY ORGANIC SOILS			Peat and other highly organic soils		Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

	Asphalt		Glacial Till		Bedrock (Igneous)	
	Concrete		Clay Shale		Bedrock (Limestone)	**DYREGROV ROBINSON INC.** CONSULTING GEOTECHNICAL ENGINEERS
	Fill				Bedrock (Undifferentiated)	



FRACTION		PARTICLE SIZE (mm) Min.	PARTICLE SIZE (mm) Max.
Boulders		>300	
Cobbles		75	300
Gravel	Coarse	19	75
	Fine	4.75	19
Sand	Coarse	2.0	4.75
	Medium	0.425	2.0
	Fine	0.075	0.425
Silt (non-plastic) or Clay (plastic)		< 0.075 mm	

RELATIVE PROPORTIONS (by weight) Percent	Descriptor
>35%	main fraction
35 - 50	"and"
20 – 35	Adjective e.g. silty, clayey
10 – 20	"some"
1 - 10	"trace"

Soil Classification Example

Clay 50% (main fraction), Silt 25%, Sand 17%, Gravel 8%

Clay – silty, some sand, trace gravel

TERMS and SYMBOLS

Laboratory and field tests are identified as follows:

Unconfined Comp.: undrained shear strength (kPa or psf) derived from unconfined compression testing.

Torvane: undrained shear strength (kPa or psf) measured using a Torvane

Pocket Pen.: undrained shear strength (kPa or psf) measured using a pocket penetrometer.

Unit Weight bulk unit weight of soil or rock (kN/m^3 or pcf).

SPT – N Standard Penetration Test: The number of blows (N) required to drive a 51 mm O.D. split barrel sampler 300 mm into the soil using a 63.5 kg hammer with a free fall drop height of 760 mm.

DCPT Dynamic Cone Penetration Test. The number of blows (N) required to drive a 50 mm diameter cone 300 mm into the soil using a 63.5 kg hammer with a free fall drop height of 760 mm.

M/C insitu soil moisture content in percent

PL Plastic limit, moisture content in percent

LL Liquid limit, moisture content in percent

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

Su (kPa)	Su (psf)	CONSISTENCY
<12	250	very soft
12 – 25	250 – 525	soft
25 – 50	525 – 1050	firm
50 – 100	1050 – 2100	stiff
100 – 200	2100 – 4200	very stiff
200	4200	hard

The SPT - N of non-cohesive soil is related to compactness condition as follows:

N – Blows / 300 mm	COMPACTNESS
0 - 4	very loose
4 - 10	loose
10 - 30	compact
30 - 50	dense
50 +	very dense

References:

ASTM D2487 – Classification of Soils For Engineering Purposes (Unified Soil Classification System)

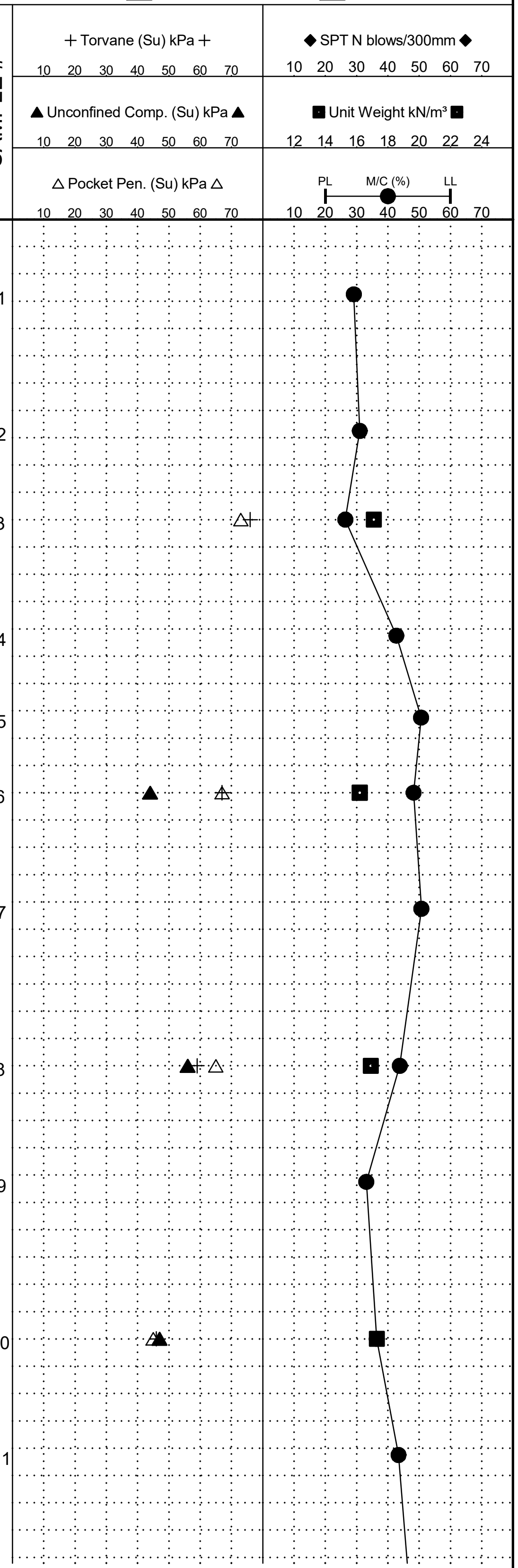
Canadian Foundation Engineering Manual, 4th Edition, Canadian Geotechnical Society, 2006

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **1**
LOCATION: UTM 14U: 633,513 m E, 5,529,169 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker MP8 Drill w/ 125 mm SS Augers & HQ Coring	ELEVATION (m): 233.769

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

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

ELEVATION (m)	DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #
			CRUSHED LIMESTONE (FILL) - 19 mm down gradation		
233	1		CLAY (FILL) - silty, trace sand, trace gravel - brown / grey - moist, stiff, high plasticity - trace silt inclusions	GRAB	G1
232	2				
231	3			GRAB	G2
230	4			SHELBY TUBE	T3
229	5			GRAB	G4
228	6		CLAY - silty - mottled brown and grey - moist, stiff, high plasticity, trace silt inclusions	GRAB	G5
227	7		- grey below 6 m	SHELBY TUBE	T6
226	8			GRAB	G7
225	9				
224	10		- trace gravel below 9 m	SHELBY TUBE	T8
223	11		- firm below 11 m	GRAB	G9
222	12				
221	13			SHELBY TUBE	T10
220	14			GRAB	G11
219					



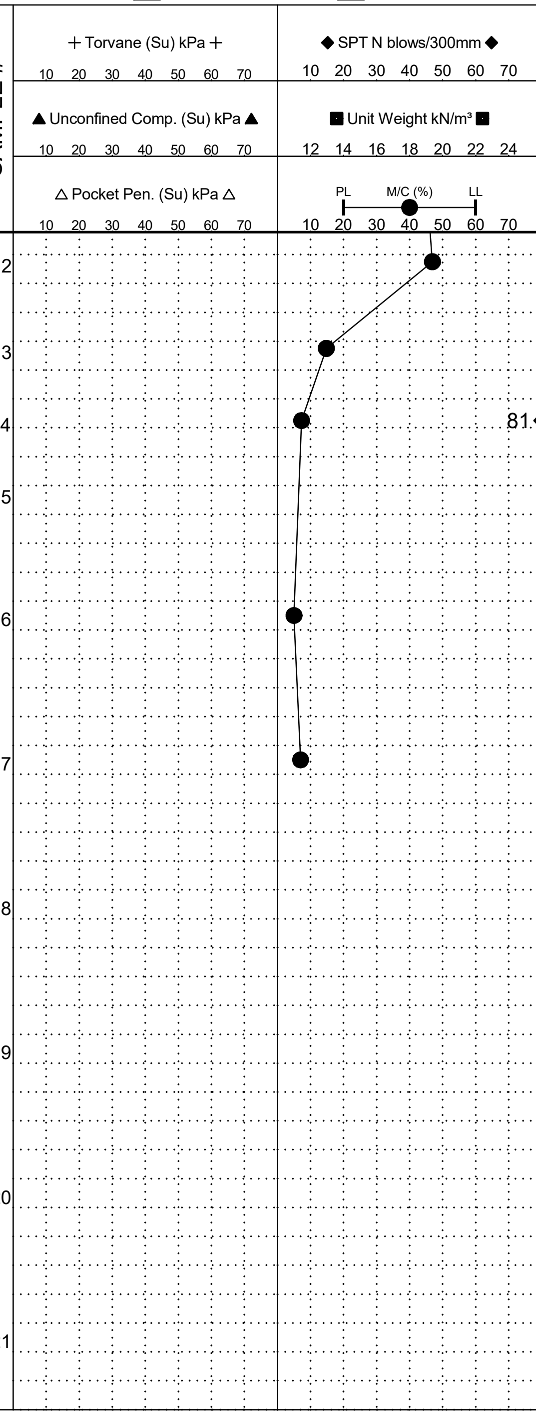
DYREGROV ROBINSON INC.
Consulting Geotechnical Engineers

LOGGED BY: CR/JW	COMPLETION DEPTH: 27.43 m
REVIEWED BY: GR	COMPLETION DATE: 3/8/21
PROJECT ENGINEER: Gil Robinson	

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **1**
LOCATION: UTM 14U: 633,513 m E, 5,529,169 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker MP8 Drill w/ 125 mm SS Augers & HQ Coring	ELEVATION (m): 233.769

SAMPLE TYPE	GRAB	SHELBY TUBE	SPLIT SPOON	BULK	NO RECOVERY	CORE
BACKFILL TYPE	BENTONITE	GRAVEL	SLOUGH	GROUT	CUTTINGS	SAND

ELEVATION (m)	DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #
218			CLAY - silty - mottled brown and grey - moist, stiff, high plasticity, trace silt inclusions *continued from previous page*	GRAB	G12
	16		SILT (TILL) - some sand, trace gravel, trace cobbles - grey - moist to wet, loose to 17 m - dry to moist, dense to very dense below 17 m	GRAB	G13
217	17			SPLIT SPOON	S14
216	18			CORE	C15
215	19			CORE	C16
214	20				
213	21			CORE	C17
212	22			CORE	C18
211	23				
210	24		BEDROCK - Red River Formation, Lower Fort Garry Member (dolomitic mudstone) - fair to good quality - excellent below 25.9 m - reddish to whitish grey color - strong to very strong - horizontal and vertical fractures - close to moderately close discontinuity spacing - close to wide discontinuity spacing below 25.9 m - gapped to open joint aperture - evidence of water flow (Class 3) - trace small vugs (< 0.5 mm)	CORE	C19
209	25			CORE	C20
208	26				
207	27			CORE	C21



END OF TEST HOLE AT 27.4 m IN BEDROCK

Notes:

1. No sloughing or seepage observed before switching to HQ coring
2. Switched to HQ coring w/ casing advancer at 16.8 m. Advanced casing to 17.3 m. Coring began at 17.3 m b/l grade.
3. Test hole backfilled with auger cuttings and bentonite chips.

DYREGROV ROBINSON INC.
Consulting Geotechnical Engineers

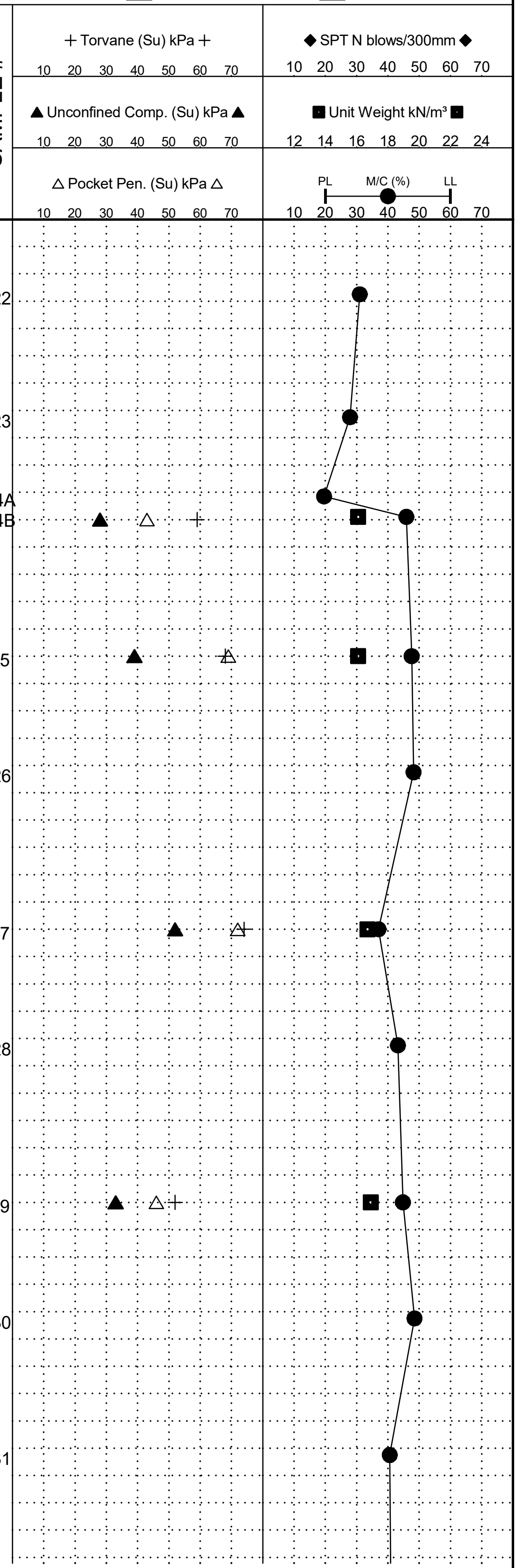
LOGGED BY: CR/JW	COMPLETION DEPTH: 27.43 m
REVIEWED BY: GR	COMPLETION DATE: 3/8/21
PROJECT ENGINEER: Gil Robinson	Page 2 of 2

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **2**
LOCATION: UTM 14U: 633,556 m E, 5,529,147 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker MP8 Drill w/ 125 mm SS Augers & HQ Coring	ELEVATION (m): 233.769

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

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

ELEVATION (m)	DEPTH (m)	SOIL DESCRIPTION	SAMPLE #
	0	CRUSHED LIMESTONE (FILL) - 19 mm down gradation	
233	1	CLAY (FILL) - silty, some sand, trace gravel - brown / grey - moist, firm, high plasticity	G22
232	2		G23
231	3	CLAY - silty - brown - moist, stiff, high plasticity - trace silt inclusions	T24A, T24B
229	4.5	- below 4.5 m: mottled brown and grey	T25
228	6	- grey below 6 m	G26
226	8		T27
225	9		G28
224	10	- trace gravel below 10 m	
223	11	- below 11 m; trace till inclusions, trace gravel, firm	T29
222	12		G30
220	14		G31



BH GEOTECH PLOTS-AUGUST 2013 214522_MARKET LANDS PHASE 1_GINT.GPJ DATA TEMPLATE - AUGUST 2, 2013.GDT 2/9/21

DYREGROV ROBINSON INC.
Consulting Geotechnical Engineers

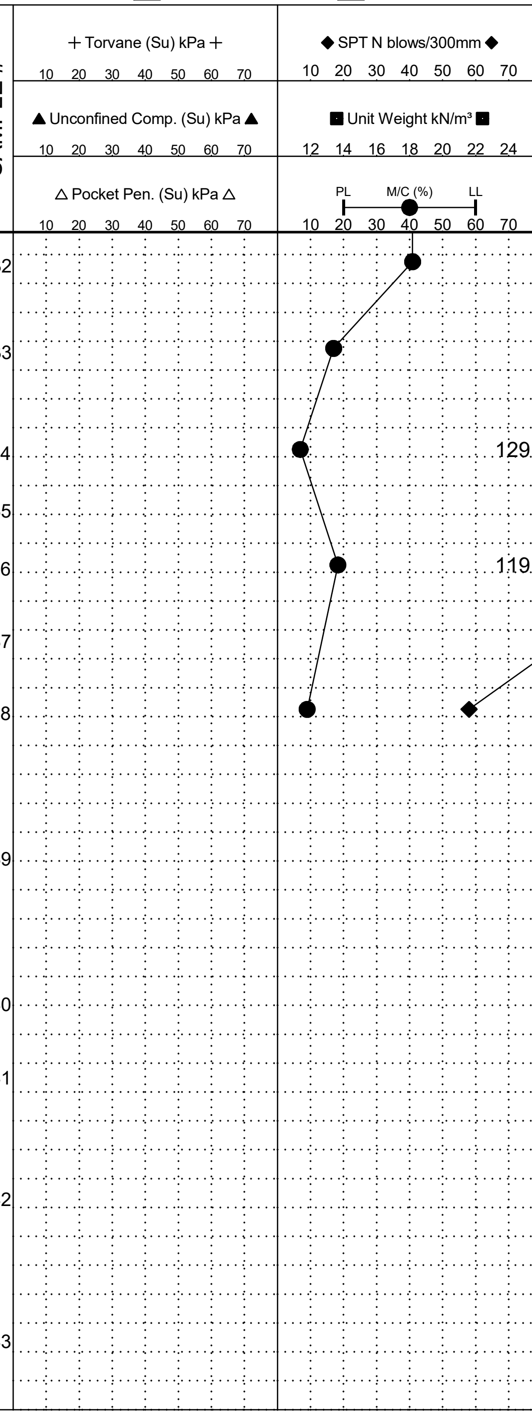
LOGGED BY: CR/JW	COMPLETION DEPTH: 27.43 m
REVIEWED BY: GR	COMPLETION DATE: 4/8/21
PROJECT ENGINEER: Gil Robinson	Page 1 of 2

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **2**
LOCATION: UTM 14U: 633,556 m E, 5,529,147 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: Acker MP8 Drill w/ 125 mm SS Augers & HQ Coring	ELEVATION (m): 233.769

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

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

ELEVATION (m)	DEPTH (m)	SOIL DESCRIPTION	SAMPLE #
218–211	16–23	SILT (TILL) - trace sand, trace gravel, trace cobbles - grey - moist to wet, loose to 17 m - dry to moist, dense to very dense below 17 m S36) moisture content may have been affected by coring water S38) sand and gravel recovered in split barrel sampler	G32, G33, S34, C35, S36, C37, S38, C39
211–207	23–27	BEDROCK - Red River Formation, Lower Fort Garry Member (dolomitic mudstone) - fair to excellent quality - reddish to whitish grey color - strong to very strong - horizontal and vertical fractures - close to moderately close discontinuity spacing - gapped to open joint aperture - evidence of water flow (Class 3) - trace small vugs (< 0.5 mm)	C40, C41, C42, C43



END OF TEST HOLE AT 27.4 m IN BEDROCK

Notes:

1. No sloughing or seepage observed before switching to HQ coring
2. Switched to HQ coring w/ casing advancer at 16.8 m. Advanced casing to 17.3 m. Coring began at 17.3 m b/l grade.
3. Test hole backfilled with auger cuttings and bentonite chips.

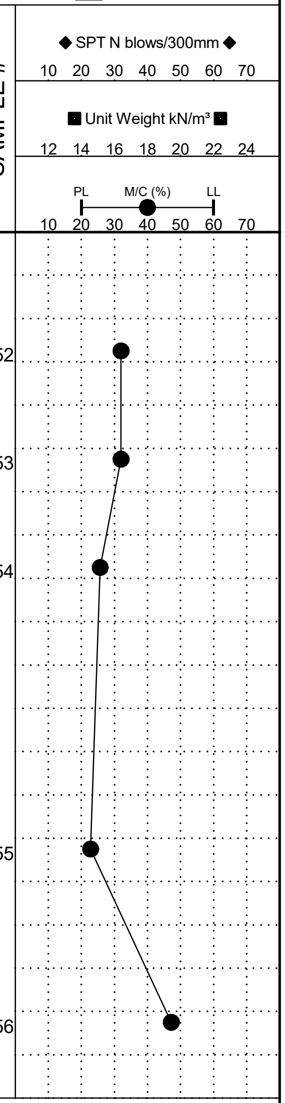
DYREGROV ROBINSON INC.
Consulting Geotechnical Engineers

LOGGED BY: CR/JW	COMPLETION DEPTH: 27.43 m
REVIEWED BY: GR	COMPLETION DATE: 4/8/21
PROJECT ENGINEER: Gil Robinson	Page 2 of 2

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **3**
LOCATION: UTM 14U: 633,516 m E, 5,529,160 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: ACKER MP8 Drill w/ 125 mm SS Augers	ELEVATION (m): 233.791

SAMPLE TYPE	GRAB	SHELBY TUBE	SPLIT SPOON	BULK	NO RECOVERY	CORE
BACKFILL TYPE	BENTONITE	GRAVEL	SLOUGH	GROUT	CUTTINGS	SAND

ELEVATION (m)	DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #
			CRUSHED LIMESTONE (FILL) - 19 mm down gradation		
233	1		CLAY (FILL) - trace sand, trace gravel - brown / grey - moist, stiff, high plasticity	GRAB	G52
232	2			GRAB	G53
231	3			GRAB	G54
230	4			GRAB	G55
229	5				
228	6		CLAY - silty - mottled brown and grey - moist, stiff, high plasticity - grey below 5.5 m	GRAB	G56



END OF TEST HOLE AT 6.1 m IN CLAY

Notes:

1. No sloughing and seepage observed during drilling.
2. Upon completion of drilling, test hole open to 6.1 m and dry.
3. Test hole backfilled with auger cuttings.

BH GEOTECH PLOTS-AUGUST 2013 214522_MARKET LANDS PHASE 1_GINT.GPJ DATA TEMPLATE - AUGUST 2, 2013.GDT 2/9/21

DYREGROV ROBINSON INC.
Consulting Geotechnical Engineers

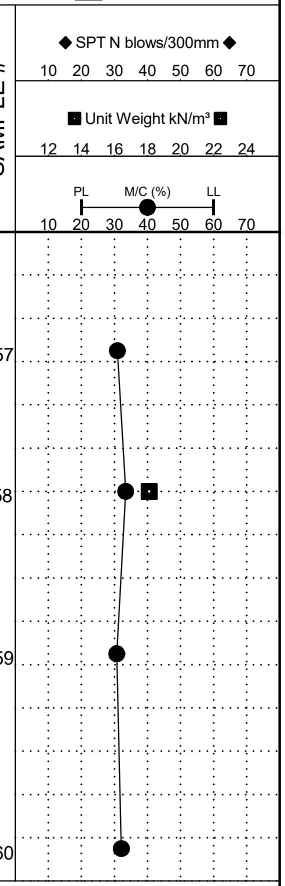
LOGGED BY: JW	COMPLETION DEPTH: 6.10 m
REVIEWED BY: GR	COMPLETION DATE: 5/8/21
PROJECT ENGINEER: Gil Robinson	

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **4**
LOCATION: UTM 14U: 633,534 m E, 5,529,163 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: ACKER MP8 Drill w/ 125 mm SS Augers	ELEVATION (m): 234.089

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

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

ELEVATION (m)	DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #
234	0	CRUSHED LIMESTONE (FILL) - 19 mm down gradation		
		CLAY (FILL) - trace sand, trace gravel - brown / grey - moist, stiff, high plasticity	GRAB	G57
233	1			
232	2		SHELBY TUBE	T58
231	3		GRAB	G59
230	4	CLAY - silty - mottled brown and grey, moist, stiff, high plasticity	GRAB	G60



END OF TEST HOLE AT 4.6 m IN CLAY

Notes:

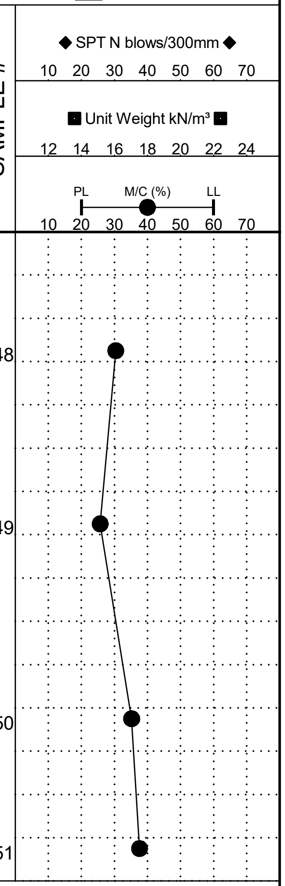
1. No sloughing and seepage observed during drilling.
2. Upon completion of drilling, test hole open to 4.6 m and dry.
3. Test hole backfilled with auger cuttings.

DYREGROV ROBINSON INC. Consulting Geotechnical Engineers	LOGGED BY: JW	COMPLETION DEPTH: 4.57 m
	REVIEWED BY: GR	COMPLETION DATE: 5/8/21
	PROJECT ENGINEER: Gil Robinson	Page 1 of 1

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **5**
LOCATION: UTM 14U: 633,539 m E, 5,529,143 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: ACKER MP8 Drill w/ 125 mm SS Augers	ELEVATION (m): 234.089

SAMPLE TYPE	GRAB	SHELBY TUBE	SPLIT SPOON	BULK	NO RECOVERY	CORE
BACKFILL TYPE	BENTONITE	GRAVEL	SLOUGH	GROUT	CUTTINGS	SAND

ELEVATION (m)	DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #
234			CRUSHED LIMESTONE (FILL) - 19 mm down gradation		
			CLAY (FILL) - trace sand, trace gravel - brown / grey - moist, stiff, high plasticity		
				GRAB	G48
233	1				
232	2			GRAB	G49
231	3				
				GRAB	G50
230	4		CLAY - silty - mottled brown and grey - moist, stiff, high plasticity, trace silt inclusions		
				GRAB	G51



END OF TEST HOLE AT 4.6 m IN CLAY

Notes:

1. No sloughing and seepage observed during drilling.
2. Upon completion of drilling, test hole open to 4.6 m and dry.
3. Test hole backfilled with auger cuttings.

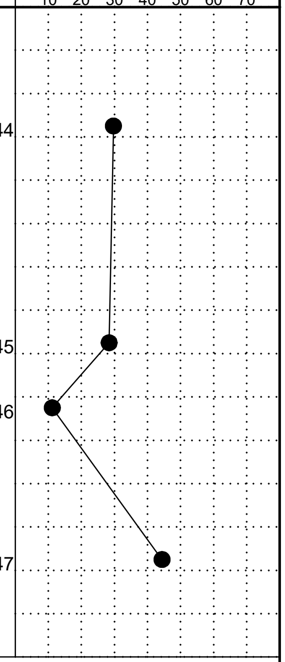
DYREGROV ROBINSON INC. Consulting Geotechnical Engineers	LOGGED BY: JW	COMPLETION DEPTH: 4.57 m
	REVIEWED BY: GR	COMPLETION DATE: 5/8/21
	PROJECT ENGINEER: Gil Robinson	Page 1 of 1

PROJECT: Market Lands MXU South Parcel	CLIENT: Market Lands Inc.	TEST HOLE NO: **6**
LOCATION: UTM 14U: 633,565 m E, 5,529,145 m N		PROJECT NO.: 214522
CONTRACTOR: Paddock Drilling Ltd.	METHOD: ACKER MP8 Drill w/ 125 mm SS Augers	ELEVATION (m): 233.559

SAMPLE TYPE	GRAB	SHELBY TUBE	SPLIT SPOON	BULK	NO RECOVERY	CORE
BACKFILL TYPE	BENTONITE	GRAVEL	SLOUGH	GROUT	CUTTINGS	SAND

ELEVATION (m)	DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #
			CRUSHED LIMESTONE (FILL) - 19 mm down gradation		
233			CLAY (FILL) - trace sand, trace gravel - brown / grey - moist, stiff, high plasticity, trace silt inclusions		
	1				G44
232	2				
231					G45
	3		SAND AND GRAVEL (FILL) - brown, moist, loose		G46
230			CLAY - silty - mottled brown and grey - moist, stiff, high plasticity, trace silt inclusions		
	4				G47
229					

◆ SPT N blows/300mm ◆
10 20 30 40 50 60 70
■ Unit Weight kN/m³ ■
12 14 16 18 20 22 24
PL M/C (%) LL
10 20 30 40 50 60 70



END OF TEST HOLE AT 4.6 m IN CLAY
Notes:
1. No sloughing and seepage observed during drilling.
2. Upon completion of drilling, test hole open to 4.6 m and dry.
3. Test hole backfilled with auger cuttings.

BH GEOTECH PLOTS-AUGUST 2013 214522_MARKET LANDS PHASE 1_GINT.GPJ DATA TEMPLATE - AUGUST 2, 2013.GDT 2/9/21

DYREGROV ROBINSON INC. Consulting Geotechnical Engineers	LOGGED BY: JW	COMPLETION DEPTH: 4.57 m
	REVIEWED BY: GR	COMPLETION DATE: 5/8/21
	PROJECT ENGINEER: Gil Robinson	Page 1 of 1

Your Project #: 214522
Site Location: WINNIPEG
Your C.O.C. #: 1OF1

Attention: GIL ROBINSON

DYREGROV ROBINSON INC
UNIT 1, 1692 DUBLIN AVENUE
WINNIPEG, MB
CANADA R3H 1A8

Report Date: 2021/08/25
Report #: R3062882
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C160098
Received: 2021/08/17, 15:11

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (Soluble)	3	2021/08/23	2021/08/24	AB SOP-00033 / AB SOP-00020	SM 23-4500-Cl-E m
Resistivity	3	N/A	2021/08/23		Auto Calc
Conductivity @25C (Soluble)	3	2021/08/23	2021/08/23	AB SOP-00033 / AB SOP-00004	SM 23 2510 B m
pH @25C (Soluble)	3	2021/08/23	2021/08/23	AB SOP-00033 / AB SOP-00006	SM 23 4500 H+B m
Soluble Ions	3	2021/08/23	2021/08/23	AB SOP-00033 / AB SOP-00042	EPA 6010d R5 m
Soluble Ions Calculation	3	2021/08/18	2021/08/23		Auto Calc
Soluble Paste	3	2021/08/23	2021/08/23	AB SOP-00033	Carter 2nd ed 15.2 m

Remarks:

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Bureau Veritas liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Bureau Veritas has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Bureau Veritas, unless otherwise agreed in writing. Bureau Veritas is not responsible for the accuracy or any data impacts, that result from the information provided by the customer or their agent.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Bureau Veritas Laboratories Calgary: 2021 - 41st Avenue N.E. T2E 6P2 Telephone (403) 291-3077 Fax (403) 291-9468

Your Project #: 214522
Site Location: WINNIPEG
Your C.O.C. #: 1OF1

Attention: GIL ROBINSON

DYREGROV ROBINSON INC
UNIT 1, 1692 DUBLIN AVENUE
WINNIPEG, MB
CANADA R3H 1A8

Report Date: 2021/08/25
Report #: R3062882
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BV LABS JOB #: C160098
Received: 2021/08/17, 15:11

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Customer Solutions, Western Canada Customer Experience Team
Email: customersolutionswest@bureauveritas.com
Phone# (403) 291-3077
==
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BV Labs Job #: C160098
Report Date: 2021/08/25

DYREGROV ROBINSON INC
Client Project #: 214522
Site Location: WINNIPEG

RESULTS OF CHEMICAL ANALYSES OF SOIL

BV Labs ID		ADZ064		ADZ065		ADZ066		
Sampling Date		2021/08/03		2021/08/03		2021/08/03		
COC Number		1OF1		1OF1		1OF1		
	UNITS	**TH1-T3-10'**	**RDL**	**TH1-T8-30'**	**RDL**	**TH1-S14-55'**	**RDL**	**QC Batch**
Calculated Parameters								
Resistivity @ 25 °C	ohm-m	1.7	0.050	1.5	0.050	6.7	0.050	A324564
Calculated Sulphate (SO4)	%	0.34	0.00013	0.36	0.00013	0.011	0.00013	A324563
Soluble Parameters								
Soluble Chloride (Cl)	mg/L	320	10	340	20	180	10	A329684
Soluble Conductivity	dS/m	6.1	0.020	6.6	0.020	1.5	0.020	A328725
Soluble pH	pH	7.78	N/A	7.82	N/A	8.08	N/A	A328212
Saturation %	%	97	N/A	92	N/A	27	N/A	A326599
Soluble Sulphate (SO4)	mg/L	3500	5.0	3900	5.0	410	5.0	A328812

RDL = Reportable Detection Limit
N/A = Not Applicable

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GENERAL COMMENTS

Results relate only to the items tested.

Bureau Veritas Laboratories Calgary: 2021 - 41st Avenue N.E. T2E 6P2 Telephone (403) 291-3077 Fax (403) 291-9468

BV Labs Job #: C160098
Report Date: 2021/08/25

DYREGROV ROBINSON INC
Client Project #: 214522
Site Location: WINNIPEG

QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
A326599	KKC	QC Standard	Saturation %	2021/08/23		100	%	75 - 125
A326599	KKC	RPD	Saturation %	2021/08/23	3.1		%	12
A328212	JHC	QC Standard	Soluble pH	2021/08/23		99	%	98 - 102
A328212	JHC	Spiked Blank	Soluble pH	2021/08/23		100	%	97 - 103
A328212	JHC	RPD	Soluble pH	2021/08/23	0		%	N/A
A328725	LZ3	QC Standard	Soluble Conductivity	2021/08/23		107	%	75 - 125
A328725	LZ3	Spiked Blank	Soluble Conductivity	2021/08/23		99	%	90 - 110
A328725	LZ3	Method Blank	Soluble Conductivity	2021/08/23	ND, RDL=0.020		dS/m	
A328725	LZ3	RPD	Soluble Conductivity	2021/08/23	9.1		%	20
A328812	MAP	QC Standard	Soluble Sulphate (SO4)	2021/08/23		114	%	75 - 125
A328812	MAP	Method Blank	Soluble Sulphate (SO4)	2021/08/23	ND, RDL=5.0		mg/L	
A328812	MAP	RPD	Soluble Sulphate (SO4)	2021/08/23	27		%	30
A329684	ZI	Matrix Spike	Soluble Chloride (Cl)	2021/08/24		110	%	75 - 125
A329684	ZI	QC Standard	Soluble Chloride (Cl)	2021/08/24		108	%	75 - 125
A329684	ZI	Spiked Blank	Soluble Chloride (Cl)	2021/08/24		106	%	80 - 120
A329684	ZI	Method Blank	Soluble Chloride (Cl)	2021/08/24	ND, RDL=10		mg/L	
A329684	ZI	RPD	Soluble Chloride (Cl)	2021/08/24	29		%	30

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

QC Standard: A sample of known concentration prepared by an external agency under stringent conditions. Used as an independent check of method accuracy.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

Bureau Veritas Laboratories Calgary: 2021 - 41st Avenue N.E. T2E 6P2 Telephone (403) 291-3077 Fax (403) 291-9468

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Ghayasuddin Khan, M.Sc., P.Chem., QP, Scientific Specialist, Inorganics

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APPENDIX B

Bedrock Core Summary (Table B1)

Compressive Strength Test Results (Table B2)

Bedrock Core Photographs (Figures B1 & B2)

DYREGROV ROBINSON INC.

Project: Market Lands MXU South Parcel
Project No. 214522

Table B1) Bedrock Core Sample Summary

Test Hole	Sample #	Coring Details					Core Sample		%Recovery	RQD	Notes
		Depth (m)		Depth (ft)		Length	Lrecovered	L > 4"			
		from	to	from	to	(inches)	(inches)	(inches)			
1	C15	17.22	18.29	56.5	60.0						Glacial Till
	C16	18.29	19.81	60.0	65.0						Glacial Till
	C17	19.81	21.34	65.0	70.0						Glacial Till
	C18	21.34	22.86	70.0	75.0						Glacial Till
	C19	22.86	24.38	75.0	80.0	22	22	15.5	100%	70%	Bedrock
	C20	24.38	25.91	80.0	85.0	60	56	49.5	93%	83%	Bedrock
	C21	25.91	27.43	85.0	90.0	60	62	62	103%	103%	Bedrock
2	C35	17.53	18.29	57.5	60.0						Glacial Till
	C37	18.75	19.81	61.5	65.0						Glacial Till
	C39	20.27	21.41	66.5	70.3						Glacial Till
	C40	21.41	23.34	70.3	76.6	18	18	17	100%	94%	Bedrock
	C41	23.34	24.44	76.6	80.2	43	43	32	100%	74%	Bedrock
	C42	24.44	25.91	80.2	85.0	58	60	52.5	103%	91%	Bedrock
	C43	25.91	27.43	85.0	90.0	60	56	52	93%	87%	Bedrock

DYREGROV ROBINSON INC.

Project: Market Lands MXU South Parcel
Project #: 214484

Table B2 - Bedrock Strength Testing Summary

Test Hole	Sample	Depth (ft / m)	Unconfined Compressive Strength (MPa)
1	C19	79' / 24.1 m	142.5
2	C43	85' / 25.9 m	124.6

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

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

"Engineering and Testing Solutions That Work for You"

Dyregrov Robinson Inc.
Unit 1 - 1692 Dublin Avenue
Winnipeg, Manitoba
R3H 1A8

File No.: 21-174-01
Ref. No.: 21-174-1-3

Attention: Gil Robinson, M.Sc., P. Eng.

Project: PROJECT NO. 214522; BEDROCK CORE COMPRESSIVE STRENGTH

Contractor: -
Date Cored: -
Submitted By: Client

Page: 1 of 1
Date Received: Aug 10/21
Tested By: ENG-TECH (Kevin Dowbeta)

Core No.	Location	Length		Average Diameter (mm)	Rate of Loading (kN/s)	Compressive Strength (MPa)	Date Tested (m/d/y)
		Cored (mm)	Tested (mm)				
1	Test Hole 1: Sample C19 - 79 feet	270	141	63.0	1.0	142.5	Aug 14/21
2	Test Hole 2: Sample C43 - 85 feet	227	156	63.0	1.1	124.6	Aug 14/21

Comments: The unconfined strength was determined in accordance with ASTM D2938-95 procedure with the cores in the as received moisture condition and at room temperature. Strain measurements were not recorded.

Email: gilrobinson@drgeotechnical.com

Enclosure: Photograph 1 - Core Failures

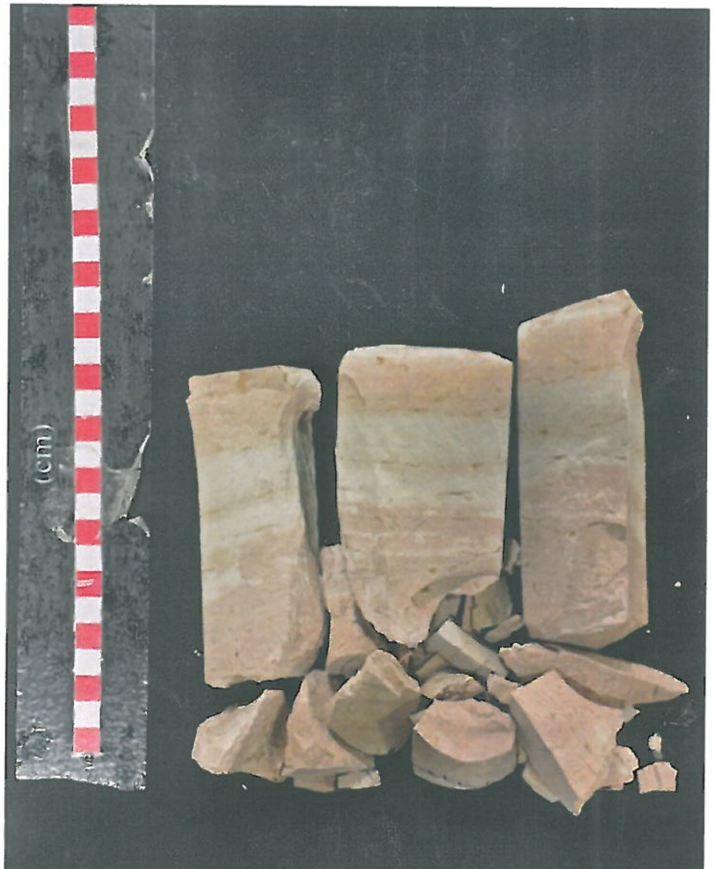
ENG-TECH Consulting Limited

Per

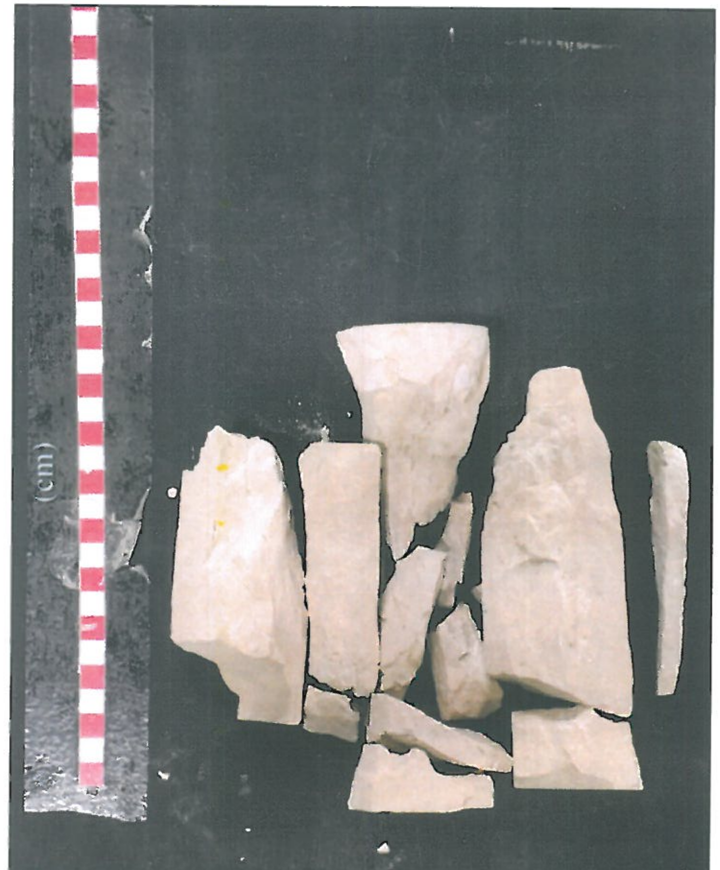
Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

1999 20 2019 YEARS OF EXCELLENCE

Photograph 1: Compressive Strength Failures



Core No. 1
Test Hole 1, Sample C19



Core No. 2
Test Hole 2, Sample C43

ENG-TECH
CONSULTING LIMITED

C15) Core Depth:	56.5 – 60.0 ft (17.2 – 18.3 m)	Glacial Till		
C16) Core Depth:	60.0 – 65.0 ft (18.3 – 19.8 m)	Glacial Till		
C17) Core Depth:	65.0 – 70.0 ft (19.8 – 21.3 m)	Glacial Till		
C18) Core Depth:	70.0 – 75.0 ft (21.3 – 22.9 m)	Glacial Till		
C19) Core Depth:	75.0 – 80.0 ft (22.9 – 24.4 m)	% Recovered = 100,	RQD = 70%	
C20) Core Depth:	80.0 – 85.0 ft (24.4 – 25.9 m)	% Recovered = 93,	RQD = 83%	
C21) Core Depth:	85.0 – 90.0 ft (25.9 – 27.4 m)	% Recovered = 103,	RQD = 100%	

DYREGROV ROBINSON INC.
CONSULTING GEOTECHNICAL ENGINEERS

Market Lands MXU – South Parcel
Bedrock Core Photograph – Test Hole 1

SCALE:	MADE BY:	CHKD BY:	PROJECT NO.	DATE:	
NTS	AA	GR	214522	August 2021	**FIGURE B1**

C35) Core Depth:	57.5 – 60.0 ft (17.5 – 18.3 m)	Glacial Till
C37) Core Depth:	61.5 – 65.0 ft (18.8 – 19.8 m)	Glacial Till
C39) Core Depth:	66.5 – 70.3 ft (20.3 – 21.4 m)	Glacial Till
C40) Core Depth:	70.3 – 76.6 ft (21.4 – 23.3 m)	% Recovered = 100, RQD = 94%
C41) Core Depth:	76.6 – 80.2 ft (23.3 – 24.4 m)	% Recovered = 100, RQD = 74%
C42) Core Depth:	80.2 – 85.0 ft (24.4 – 25.9 m)	% Recovered = 103, RQD = 91%
C43) Core Depth:	85.0 – 90.0 ft (25.9 – 27.4 m)	% Recovered = 93, RQD = 87%

DYREGROV ROBINSON INC.
CONSULTING GEOTECHNICAL ENGINEERS

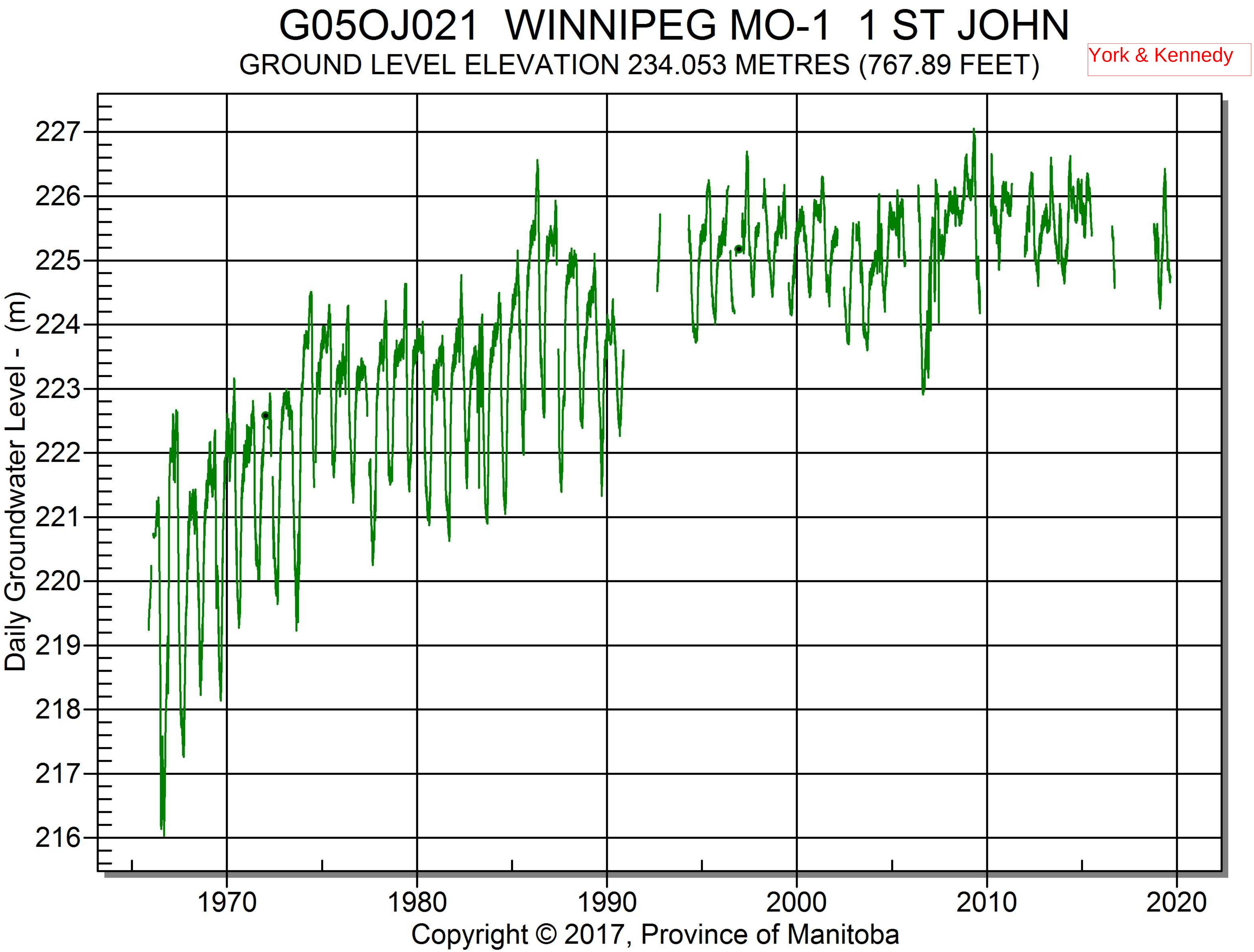
Market Lands MXU – South Parcel
Bedrock Core Photograph – Test Hole 2

SCALE:	MADE BY:	CHKD BY:	PROJECT NO.	DATE:	
NTS	AA	GR	214522	August 2021	**FIGURE B2**

APPENDIX C

Provincial Bedrock Well Hydrograph

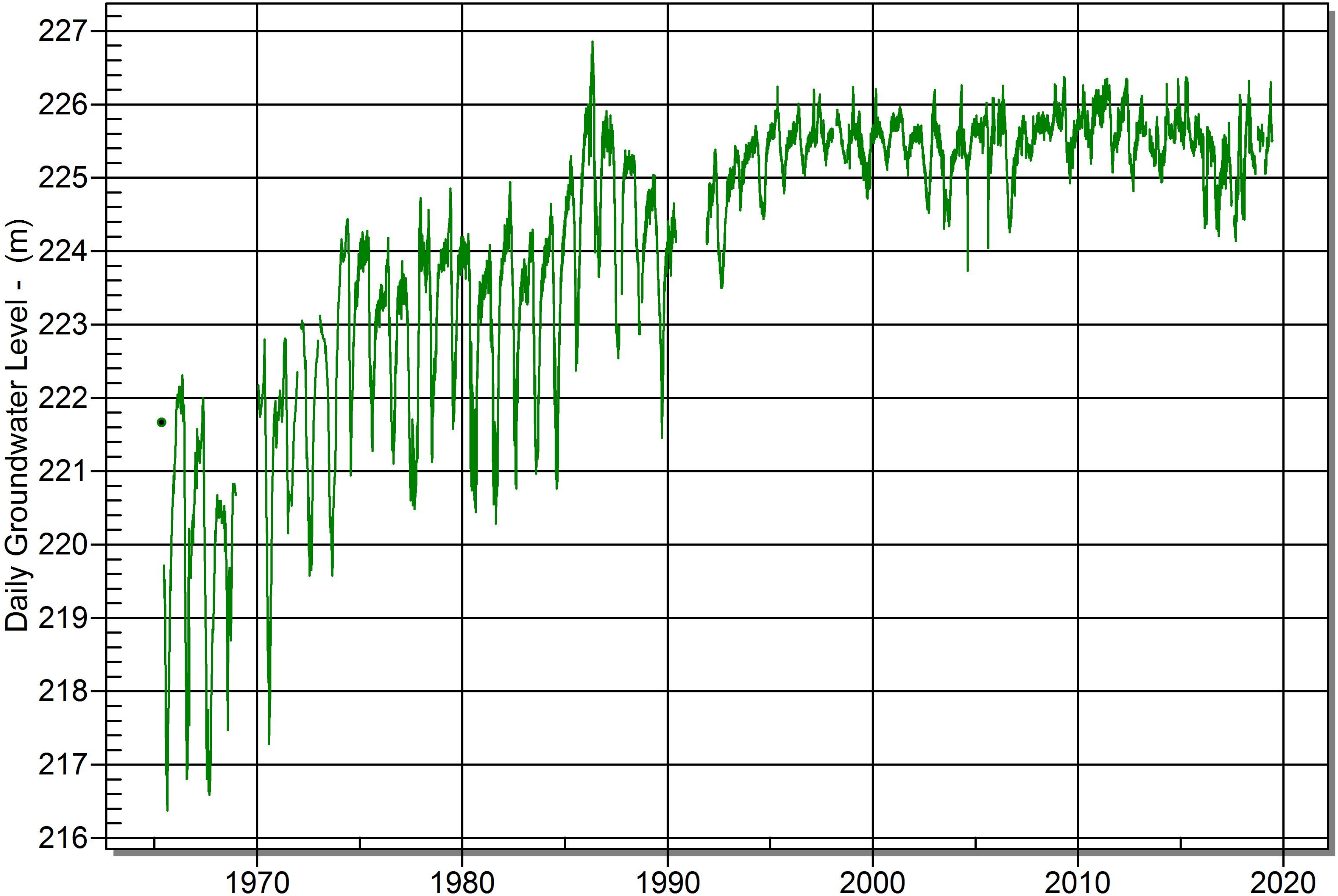
Note that the attached hydrograph data may be subject to mechanical or human error, is largely unchecked, and is provided as-is; it comes with no implied or expressed guarantee as to the accuracy of the data and the values are subject to correction. Reference: Hydata (2020), Province of Manitoba, Groundwater Management



G05OJ021 WINNIPEG MO-1 1 ST JOHN
GROUND LEVEL ELEVATION 234.053 METRES (767.89 FEET)
York & Kennedy
Daily Groundwater Level - (m)
227
226
225
224
223
222
221
220
219
218
217
216
1970
1980
1990
2000
2010
2020
Copyright © 2017, Province of Manitoba

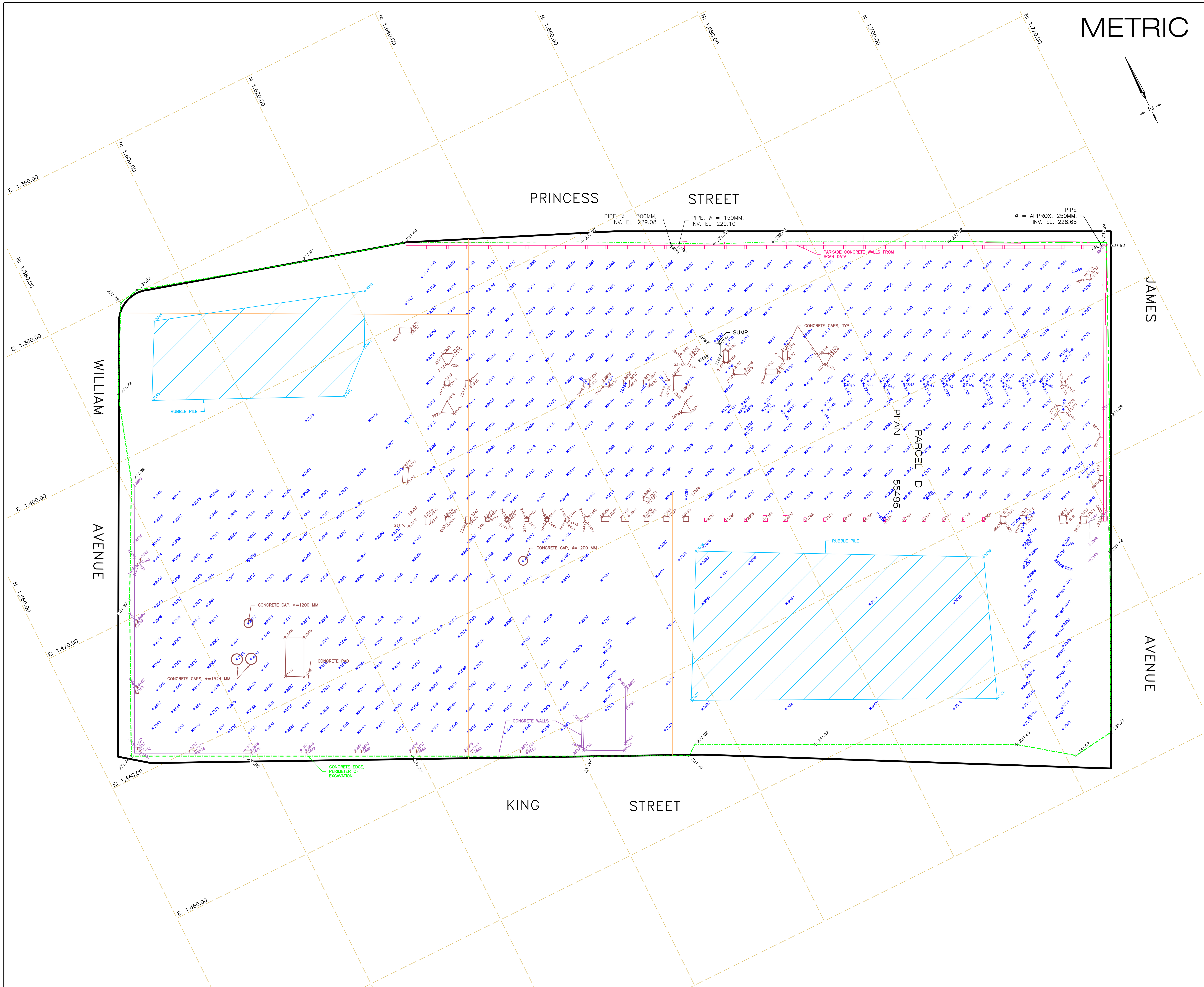
G05MJ042 YMCA M-36 ST JOHN

GROUND LEVEL ELEVATION 232.145 METRES (761.63 FEET)



APPENDIX D

Site Survey Drawing (for information only)



FILE: 20-0192
DWG: 20-0192 MET BLDG FOOTPRINT_R0
Drafter: AS

BARNES 1906 &DUNCAN
SURVEYING, ENGINEERING & GEOMATICS
6 DONALD STREET
WINNIPEG, MANITOBA
R3L 0K6

CLIENT:
RAKOWSKI CARTAGE & WRECKING LTD.

WINNIPEG, MANITOBA

PLAN SHOWING CERTAIN FEATURES OF PART OF IN PARCEL D, PLAN 55495 WLTO

BEING

151 TO 171 PRINCESS STREET

CITY OF WINNIPEG
MANITOBA

SCALE - 1 : 250

LEGEND

- Property Lines
- Concrete Walls from Scan Data
- Outer Conc. Edge for Excavation
- Conc. Walls from Survey Data
- Rubble Piles Areas
- Concrete Pile Caps
- Piles
- Existing Conc. Elevations

NOTES

All dimensions are in metres and may be converted to feet by multiplying by 3.28084 .

Elevations are referred to Geodetic Datum and referred to City of Winnipeg Bench Mark No. 35–004.

See included Table 1 for coordinates and elevations for survey data. Coordinates are in local coordinate system.

The surveys were made between the 1st day of April and the 14th day of October, 2020.

SHOULD INFORMATION ON THE DIGITAL FILE DIFFER FROM THE INFORMATION SHOWN ON THE ORIGINAL HARD COPY, AS PROVIDED BY OUR FIRM, THE LATTER WILL GOVERN.

PROVISIONAL

Michael E. Sippola, M.L.S.
DATED THIS 22ND DAY OF OCTOBER, 2020