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City of Winnipeg

St. John's Library Addition Geotechnical Investigation Report

Prepared for:

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Project Number:
0015 015 00

Date:
January 18, 2016

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January 18, 2016 Our File No. 0015 015 00

Mr. Evan C. Wiebe, C.E.T.
City of Winnipeg
4th Floor, 185 King Street
Winnipeg, Manitoba
R3B 1J1

RE: St. John's Library Addition
Geotechnical Investigation Report

TREK Geotechnical Inc. is pleased to submit our Final Report for the Geotechnical Investigation for the above noted project.

Please contact the undersigned if you have any questions. Thank you for the opportunity to serve you on this assignment.

Sincerely,

TREK Geotechnical Inc.
Per:

Ryan Belbas M.Sc., P.Eng
Geotechnical Engineer
Tel: 204.975.9433 ext. 113

Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	SMH	January 18, 2016	Final Report

Authorization Signatures

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APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Jan. 18, 2016

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1.0 Introduction

This report summarizes the results of the geotechnical investigation completed by TREK Geotechnical Inc. (TREK) for the proposed St. John's Library addition located at 500 Salter Street in Winnipeg, Manitoba. The terms of reference for the investigation are included in our proposal to Mr. Evan Wiebe of the City of Winnipeg, dated August 27th, 2015. The scope of work includes a subsurface investigation, laboratory testing, and the provision of recommendations for the design and construction of foundations, concrete slabs, and pavements. Other considerations relative to site development such as water management, foundation and site drainage, cement specifications, materials testing and inspection requirements are also included.

2.0 Background and Existing Information

TREK understands that the proposed addition will be a two storey steel-framed structure with a basement. The main floor will be approximately 54 square metres in size and the ground floor and basement will be 52 square metres in size. Foundation loads are anticipated to range between 150 and 250 kN. The existing building has a basement; however, the foundation type supporting the existing library is unknown at the time of this report.

3.0 Field Program

3.1 Subsurface Investigation

A subsurface investigation was undertaken on December 4th, 2015 under the supervision of TREK personnel to determine the soil stratigraphy and groundwater conditions at the site. One test hole (TH15-01) was drilled within the footprint of the proposed addition as indicated on Figure 01. The test hole was drilled to power auger refusal at a depth of 18.7 m below ground surface using a CME-850 track-mounted drill rig equipped with 125 mm diameter solid stem augers. A standard Penetration Tests (SPT) was conducted in the till during drilling. The test hole was backfilled with bentonite and auger cuttings.

Subsurface soils observed during drilling were visually classified based on the Unified Soil Classification System (USCS). Samples retrieved during drilling included disturbed auger cutting and split spoon samples and relatively undisturbed Shelby tube samples. All samples retrieved during drilling were transported to TREK's testing laboratory in Winnipeg, Manitoba. Laboratory testing consisted of water content determination on all samples as well as bulk unit weight measurements and unconfined compressive strength testing on selected Shelby tube samples.

The test hole location was measured relative to the existing library. The test hole elevation was surveyed using a rod and level relative to a manhole (the outside rim) located on Machray Avenue approximately 80 m west of Salter Street (denoted as TBM 1 on Figure 01), which was assigned an arbitrary elevation of 100.0 m. The attached test hole log includes a description of the soil units encountered and other pertinent information such as groundwater and sloughing conditions, and a summary of the laboratory testing results.

3.2 Subsurface Conditions

A brief description of the soil stratigraphy and groundwater conditions encountered during drilling is provided in the following sections. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the attached test hole logs.

3.2.1 Soil Stratigraphy

The soil stratigraphy within the the upper 2.8 m consists of alternating layers of clay and silt at thicknesses ranging between 0.2 and 0.9 m. Silty clay was encountered below 2.8 m depth and extended to an underlying silt till layer at 17.1 m depth. The silty clay is generally highly plastic and stiff becoming soft with depth and the till is loose to compact becoming very dense with depth.

3.2.2 Groundwater Conditions

Groundwater seepage into the test hole was observed at a depth of 17.0 m below ground surface. Approximately 30 minutes after completion of drilling, the test hole remained open to a depth of 14.6 m where squeezing of the test hole was observed in the silty clay and a groundwater depth of 5.8 m was measured.

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

4.0 Foundation Recommendations

Based on the subsurface conditions encountered during drilling and the laboratory test results, cast-in-place concrete friction and driven precast concrete end bearing piles are considered the most suitable foundation types for this site. Limit states design parameters for these pile types are provided in accordance with the National Building Code of Canada (NBCC, 2010).

4.1 Limit States Design

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

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

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

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

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

It should be noted that to use resistance factors of $\phi = 0.6$ and $\phi = 0.4$ for resistance to axial-compression and axial-uplift loads, respectively, a static load test must be performed. However, it is unlikely that a static load test would be cost-effective for this project.

4.2 Cast-In-Place Concrete Friction Piles

Cast-in-place concrete (CIPC) friction piles will derive a majority of their resistance in shaft friction (adhesion) with a relatively small contribution from end bearing. Table 2 provides recommended factored ULS and SLS axial resistances for shaft adhesion and end bearing. Pile settlements are expected to be less than 10 mm at the pile tip (bottom of pile). The elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement.

Table 2. Recommended ULS and SLS Resistances for CIPC Friction Piles

Pile Depth Below Existing Grade (m)	Approximate Elevation (m)	ULS Axial Unit Resistance (kPa)			SLS Axial-Compressive Unit Resistance Shaft Adhesion (kPa)
		Compression $\phi = 0.4$		Uplift $\phi = 0.3$	
		Shaft Adhesion	End Bearing	Shaft Adhesion	
0 to 1.5 (interior piles) 0 to 2.5 m (exterior piles)	100.5 to 99.0 100.5 to 98.0	0	0	0	0
1.5 (or 2.5) to 14.5	99.0 to 86.0	14	70	11	14

<u>CIPC Friction Pile Design Recommendations:</u>

1. The weight of the embedded portion of the pile may be neglected.
2. The piles should have a minimum shaft diameter of 406 mm.
3. For piles supporting heated structures (excluding perimeter piles), shaft adhesion in compression and uplift within the upper 1.5 m below final grade should be neglected. For piles subjected to freezing conditions (including perimeter piles), shaft adhesion in compression and uplift within the upper 2.5 m below final grade should be neglected.
4. Pile lengths should be limited to a depth of 14.5 m below existing ground surface to avoid penetrating into the soft clay and to protect against squeezing of the pile shaft.
5. Piles should have a minimum spacing of 3 pile diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
6. Steel reinforcement should be designed by a qualified structural engineer to resist the anticipated axial (compression and tension), lateral and bending loads induced from the structure. Piles subjected to freezing conditions should be designed with adequate reinforcement length to resist ad-freezing and uplift forces related to frost action.

<u>CIPC Friction Pile Installation Recommendations:</u>

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

4.3 Driven Precast Prestressed Concrete Hexagonal Piles

Precast prestressed concrete hexagonal (PPCH) piles driven to practical refusal will derive a majority of their resistance in end bearing with a relatively small contribution from shaft adhesion. The recommended SLS and factored ULS capacities for PPCH piles driven to practical refusal are provided in Table 3. Pile settlements are expected to be less than 10 mm at the pile tip (bottom of pile). The elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement. Potential impacts to the existing building from pile installation will need to be evaluated if this pile option is preferred. In this regard, TREK should be contacted prior to the start of pile installation to review installation methodology.

Power auger refusal is often a good indicator of practical refusal depth for this type of driven pile. However, due to the inherently variable conditions of the till and bedrock underlying Winnipeg, the depth to practical refusal may vary across the site.

Table 3. Recommended ULS and SLS Resistances for Driven PPCH Piles

Pile Size (mm)	Refusal Criteria (Blows/ 25mm)	ULS Axial Resistance Compression Capacity (kN)			SLS Axial-Compressive Capacity (kN)
		$\phi = 0.4$	$\phi = 0.5$	$\phi = 0.6$	
305	5	550	690	825	445
356	8	770	965	1,155	625
406	12	990	1,240	1,485	800

The piles should be driven to at least three consecutive sets of the refusal criteria outlined in Table 3, using a diesel hammer having a minimum rated energy of 40 kJ or a hydraulic drop hammer having a minimum rated energy of 20 kJ.

Power auger refusal is often a rough indicator of practical refusal depth for this type of driven pile. However, the depth to practical refusal of the pile may vary across the site and may be deeper than encountered during drilling and as indicated on the test hole logs.

Driven PPCH Pile Design Recommendations:

1. The weight of the embedded portion of the pile may be neglected.
2. Pile spacing should not be less than 2.5 pile diameters. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
3. Pre-boring should be completed to reduce ground vibrations and protect against heave of, and consequently damage to, the existing building. Pre-boring also contributes to maintaining verticality and alignment of the piles. Pre-bore diameter should be no more than 50 mm larger than the pile diameter. A typical pre-bore depth is 3 m; however, pre-bore depth should be increased to at least 6 m below the base of existing structures (grade beams) for piles to be driven directly adjacent to existing structures. Once the pile design is complete, TREK should be contacted to assist in developing an appropriate pre-boring plan for the piles prior to construction.

4. A factored ULS shaft adhesion of 11 kPa (a geotechnical resistance factor of 0.3 has been applied to this value) can be used to design for uplift resistance for piles in clay and till. The entire pre-bore length should be neglected from uplift resistance. It should be noted that uplift loads will also be resisted by structural dead loads.
5. Piles should be designed by a qualified structural engineer to withstand design loads, handling stresses, driving stresses, and tensile forces induced from seasonal movements (*e.g.* frost-related movements) of the bearing soils.

Driven PPCH Pile Installation Recommendations

1. The pile-driving hammer should have the capability of adjusting the delivered energy to operate at higher settings during driving if the delivered energy is not sufficient to mobilize the ultimate pile capacity. The driving system should also have the capability of adjusting the delivered energy to operate at lower settings during easy driving and to prevent pile damage upon sudden pile refusal.
2. The pile-driving hammer should be equipped with a pile cushion to protect the pile head from damage during driving from direct impact with the steel driving helmet. The pile cushion should consist of a minimum of 100 mm of compressible material such as plywood or hardwood (*e.g.* oak). The pile cushion should fit tightly inside the pile helmet.
3. The piles should be cured for at least 7 days prior to driving.
4. Piles should be driven continuously once driving is initiated to the required refusal criteria.
5. Where a steel follower is required to install piles below the ground surface, the refusal criteria should be increased by 50% in order to account for additional energy losses through the use of the follower.
6. Re-driving of all piles in groups should be specified along with the requirement to monitor for pile heave. All piles exhibiting heave of 6 mm or more should be re-driven to a minimum of one set of the practical refusal criteria.
7. Pile verticality (plumbness) should be measured on all piles with adequate stick-up after practical refusal has been achieved to check if verticality is within the limits of the structural design. It is common local practice to specify a maximum acceptable percentage that the pile can be out of vertical plumbness (*e.g.* 2% out of plumb).
8. Any piles damaged, out of plumb an excessive amount, or reaching premature refusal may need to be replaced. The structural designer will have to assess non-conforming piles to determine if they are acceptable. PDA testing with CAPWAP analysis is recommended for any piles that are suspected to not meet the design capacity or to be damaged if a structural solution is not possible.

4.4 Lateral Pile Analysis

For preliminary design of pile foundations, the soil response (subgrade reaction) to lateral loads can be modeled in a simplified manner that assumes the soil around a pile can be simulated by a series of horizontal springs. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral subgrade reaction modulus (K_s) as provided in Table 4. The majority of lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the relative stiffness of the pile and soil units. Void spaces surrounding piles due to pre-boring activities should be in-filled with leanmix concrete to ensure compliance with the surrounding soil. If in-filling is not completed, the depth of the pre-bore should be neglected from lateral pile resistance calculations.

Table 4. Recommended Values for Lateral Subgrade Reaction Modulus (K_s)

Depth Below Existing Grade (m)	Approximate Elevation (m)	K_s (kN/m^3)
0 to 1.5 (or to depth of pre-bore)	100.5 to 99.0 (or elev. of pre-bore)	0
1.5 (or to depth of pre-bore) to 8.0	99.0 (or to elev. of pre-bore) to 92.5	3,300/d*
8.0 to 17.0	92.5 to 83.5	1,600/d*
> 17.0	< 83.5	22,000/d*

**d = pile diameter.*

It should be understood that using the lateral subgrade reaction modulus assumes a linear response to lateral loading and therefore is only appropriate under the following conditions:

- maximum pile deflections are small (less than 1% of the pile diameter),
- loading is static (no dynamic cycling), and
- pile material behaviour is confirmed to be within linear elastic limits by the structural engineer.

If one or more of these conditions are not met, a more rigorous analysis that includes non-linear behavior of the piles and surrounding soil is required. In this regard, as part of detailed design, a lateral pile analysis should be carried out by TREK to confirm the lateral load capacity of the piles. This analysis, which is not part of our current scope of work, should incorporate the material and section properties of the piles, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading once the final design grades are determined.

4.5 Ad-freezing Effects

Piles subjected to freezing conditions should be designed to resist ad-freezing and uplift forces related to frost action acting along the vertical faces of the pile and pile cap within the depth of frost penetration (2.5 m below ground surface). In this regard, piles may be subject to an ad-freeze bond stress of 65 kPa within the depth of frost penetration. These forces will be resisted by structural dead loads and uplift resistance provided by the length of the pile below the depth of frost penetration (and pre-bore). Alternatively, measures such as flat lying rigid polystyrene insulation could be incorporated into the design to reduce frost penetration depths and thereby ad-freezing effects and uplift forces.

4.6 Grade Beams and Pile Caps

A minimum void space of 150 mm underneath all grade beams and pile caps should be provided to avoid uplift pressures from developing on the underside of the pile cap as a result of swelling or frost action. Excavations for grade beams and pile caps could be backfilled with sand compacted to a minimum of 95% of the Standard Proctor Maximum Dry Density (SPMDD). Positive site drainage around the perimeter of the foundation units should be provided at a gradient of at least 2% to promote runoff away from the structure.

4.7 Foundation Concrete

All foundation concrete should be designed by a qualified structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads from the structure. Based on local experience gathered through previous work in Winnipeg, the degree of exposure for concrete subjected to sulphate attack is classified as severe according to Table 3, CSA A23.1-09 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56-day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-09 for concrete with severe sulphate exposure (S2). Concrete that may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-09.

4.8 Foundation Inspection Requirements

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

1. a continuous basis during:
    i. the construction of all deep foundation units,
    ii. the installation and removal of retaining structures and related backfilling operations, and
    iii. during the placement of engineered fills.
2. on an as-required basis for the construction of shallow foundation units and in excavating, dewatering and other related works.

In consideration of the above and relative to this particular project, we recommend that TREK, as the geotechnical engineer of record, be retained to inspect the installation of any foundation elements. TREK is familiar with the geotechnical conditions and the basis for the foundation recommendations and can provide any design modifications deemed to be necessary should altered subsurface conditions be encountered

5.0 Floor Slabs

5.1 Structural Floor Slabs

If floor slabs cannot tolerate movements that are typically associated with grade supported floor slabs, a structural floor slab will be required. A minimum void space of 150 mm is recommended beneath the structural floor slab to accommodate volumetric changes in the underlying sub-grade soils (*i.e.* swelling, shrinkage, and thermal expansion and contraction in unheated areas). The void space can consist of a compressible layer (*e.g.* low density polystyrene) to permit sub-grade soil movements without engaging the floor slab, or alternatively a crawl space. A vapour barrier below the slab is also recommended to minimize long-term moisture changes within the sub-grade soils.

5.2 Grade Supported Slabs

If some movement of floor and exterior slabs can be tolerated, grade supported slabs can be used. Vertical deformation of grade supported slabs should be expected due to moisture and volume changes of the underlying clay and silt soils. Although the magnitude of this movement is difficult to predict, vertical displacements of 50 mm or more are possible. Additionally, slabs in unheated areas (exterior slabs) will be subject to additional movements from freeze/thaw of the subgrade soils.

1. For best long-term performance, organics, silt, and any other deleterious material should be stripped such that the sub-grade consists of undisturbed native silty clay.
2. Excavation for slabs should be completed with a backhoe equipped with a smooth bladed bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed sub-grade.
3. After excavation, the sub-grade should be inspected by TREK personnel. Where possible the sub-grade should be proof-rolled with a fully loaded tandem axle truck to detect soft areas or silt. Soft and/or silt areas should be repaired as per recommendations provided by TREK. This will likely consist of excavating an additional 300 to 600 mm and placing a non-woven geotextile on the sub-grade and backfilling with a 50 mm down crushed limestone sub-base. The crushed limestone should be placed in lifts no greater than 150 mm and compacted to a minimum of 95% of the SPMDD.
4. The sub-grade should be protected from freezing, drying, or inundation with water. If any of these conditions occur, the sub-grade should be scarified, moisture conditioned as appropriate, and re-compacted to a minimum of 95% of the SPMDD.
5. In heated areas, the floor slab should be placed on a 150 mm thick granular base constructed of 50 mm down crushed limestone underlying a 150 mm thick base consisting of 20 mm down crushed limestone. In unheated areas (*e.g.* exterior slabs) the thickness of 50 mm down crushed limestone sub-base should be increased to 250 mm. The crushed limestone should be placed in lifts no greater than 150 mm and compacted to 98% of the SPMDD.
6. All sub-base and base materials should be well-graded and free-draining.
7. A vapour barrier should be placed above the granular base and beneath the floor slab.
8. Floor slabs should be designed to resist all structural loads and to minimize slab cracking associated with movements as a result of swelling, shrinkage, and thermal expansion and contraction of the sub-grade soils.

6.0 Pavement Design

The performance of asphalt parking areas and sidewalks will depend on the sub-grade soils. Silt is present at the site within the upper 3 m below ground surface. For best long term performance of parking areas and sidewalks, silt should be stripped such that the sub-grade consists of undisturbed native silty clay; however, this may not be economical. In this regard, if movements of parking areas and sidewalks can be tolerated, the sub-grade can consist of undisturbed silt. Assuming this option is preferred, recommendations for asphalt and sidewalk pavement sections are provided below to reduce or accommodate potential movements.

6.1 Car Parking Areas

Asphalt pavement recommendations for car parking areas are provided in Table 5.

Table 5. Recommended Sections for Asphalts Pavements

Material	Layer Thickness	Compaction Requirements
Asphalt	75 mm	98% Marshall Density
20 mm down crushed limestone	150 mm	98% of SPMDD
50 mm down crushed limestone	400 mm	98% of SPMDD
Non-Woven Geotextile (Geotex 801 or equivalent)	Required	Install as per manufacturer's recommendations

1. Organics and any other deleterious material should be stripped from the sub-grade. Stripping of the sub-grade should be completed with a backhoe equipped with a smooth bladed bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed sub-grade.
2. A non-woven geotextile (Geotex 801 or equivalent) should be placed on the sub-grade prior to placement of base materials.
3. The sub-grade should be protected from freezing, drying, or inundation with water at all times. If any of these conditions occur the sub-grade should be scarified, moisture conditioned as appropriate, and re-compacted to a minimum of 95% of the SPMDD.

6.2 Sidewalks

Recommendations for concrete sidewalks for pedestrian traffic can be designed in accordance to the City of Winnipeg Standard Construction Specifications No. CW3325 *Portland Cement Concrete Sidewalk* and No. CW3110 *Sub-Grade, Sub-Base and Base Course Construction*. Minimum recommendations for sub-grade and base preparation are provided below.

1. Organics and any other deleterious material should be stripped from the sub-grade. Stripping should be completed with a backhoe equipped with a smooth bladed bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed sub-grade.
2. A non-woven geotextile (Geotex 801 or equivalent) should be placed on the sub-grade prior to placement of base materials.
3. The sub-grade should be protected from freezing, drying, or inundation with water at all times. If any of these conditions occur the sub-grade should be scarified, moisture conditioned as appropriate, and re-compacted to a minimum of 95% of the SPMDD.
4. As a minimum, the pavement structure should consist of a 150 mm thick layer of 20 mm down crushed limestone overlying 150 mm of 50 mm down crushed limestone. Both layers should be compacted to 98% of the SPMDD. The thickness of base layers can be increased to reduce seasonal movements and provide better performance of sidewalks.

7.0 Site Drainage

Drainage adjacent to the addition, exterior slabs and parking areas should promote runoff away from the structures. A minimum gradient of about 2% should be used for both landscaped and paved areas and maintained throughout the life of the structures. Water discharge from roof leaders should be directed away from the structures.

8.0 Excavations

Excavations must be carried out in compliance with the appropriate regulations under the Manitoba Workplace Safety and Health Act. It is anticipated that short term stability can be maintained for open-cut excavations less than 3 m deep with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). If existing (adjacent) structures prevent an open excavation, TREK can provide recommendations and design parameters for shoring systems upon request. Any open-cut excavation greater than 3 m deep must be designed and sealed by a professional engineer and should be reviewed by the geotechnical engineer of record (TREK).

Excavations must be completed in a manner that prevents undermining of existing structures. In this regard, excavations should not be permitted within 1 m of existing structures, otherwise shoring may be required. Once the foundation and basement design is completed, TREK should be contacted to review the design. Furthermore, maintaining the stability of the excavation slopes for the duration of construction should be the responsibility of the Contractor. To prevent wetting or drying of the exposed excavation side slopes, they should be protected with plastic covering or similar measures. Stockpiles of excavated material and heavy equipment should be kept away from the edge of the excavation by a distance equal to or greater than the depth of excavation.

Dewatering measures may be required at this location and should be completed as necessary to maintain a dry excavation and permit proper completion of the work. If seepage is encountered, it should be directed to a sump pit and pumped out of the excavation. If saturated silts are encountered, shoring or slope flattening may be required. To prevent wet silt soils from entering the excavation, gravel buttressing could be used in conjunction with sump pits for dewatering. Surface water should be diverted away from the excavation and the excavation should be backfilled as soon as possible following construction.

9.0 Closure

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

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

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

Figures

0015 015 00
City of Winnipeg
St John's Library Addition

KEY PLAN

SCALE: N.T.S.

NOTES :

1. AERIAL IMAGE FROM GOOGLE EARTH AUGUST 24, 2015
2. TBM-01 (MANHOLE RIM) LOCATED AT 14U 5532012, 633977 APPROXIMATELY 80m WEST OF SALTER ST. ON MACHRAY AVE.

0 2.5 5 7.5 10m

SCALE : 1:250 (279mm x 432mm)

LEGEND :

TEST HOLE (TREK, DECEMBER 4, 2015)

Figure 01

Test Hole Location Plan

8 1/2" x 11"

PLOT: 1/18/2016 8:52:06 AM

FILE NAME: FIG 001 2016-01-18 Site Plan 0_C_HA 0015 015 00.dwg

Test Hole Logs

EXPLANATION OF FIELD AND LABORATORY TESTING

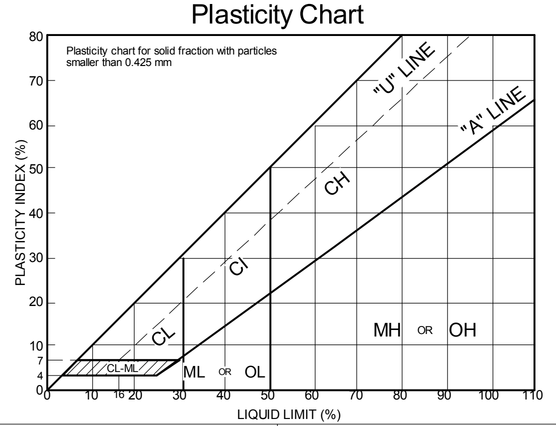
TREK GEOTECHNICAL

GENERAL NOTES

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

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

Plasticity Chart



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

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

Other Symbol Types

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

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

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

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

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

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

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

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

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

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

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

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

Sub-Surface Log

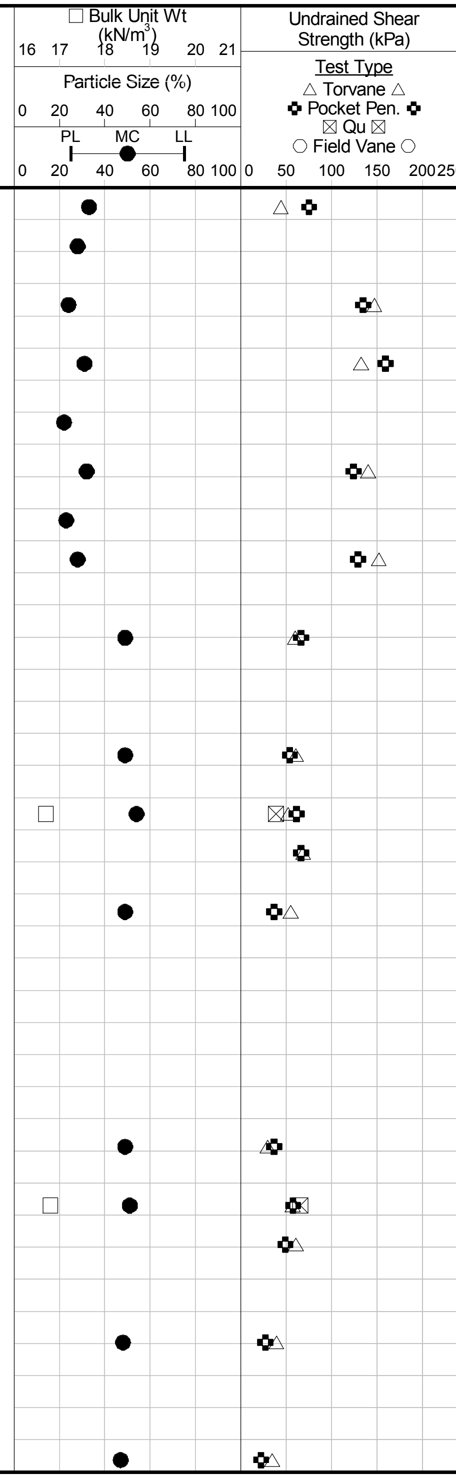
TREK GEOTECHNICAL

Client:	City of Winnipeg	**Project Number:**	0015 015 00
Project Name:	St. John's Library Addition	**Location:**	2.0 m E and 4.7 m S of SW corner of existing building
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	100.47 m
Method:	125 mm Solid Stem Auger, CME-850 Track Mount	**Date Drilled:**	4 December 2015

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

Particle Size Legend: Fines Clay Silt Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
100.2		Clay	CLAY - silty, trace organics, grey, moist, firm to stiff, high plasticity	Grab	G-01	
99.7	0.5	Silt	SILT - trace to some clay, brown, moist, firm, low plasticity - clayey below 0.5 m	Grab	G-02	
98.8	1.0	Clay	CLAY - silty, dark grey, moist, very stiff, high plasticity - trace organics, brown below 1.1 m	Grab	G-03, G-04	
98.3	1.5	Silt	SILT - trace to some clay - light brown, moist, soft to firm, low plasticity	Grab	G-05	
98.2	2.0	Clay	CLAY - silty, mottled grey and brown, moist, very stiff, high plasticity	Grab	G-06	
97.7		Silt	SILT - some clay - brown, moist, loose to compact	Grab	G-07	
	2.5 – 9.5	Clay	CLAY - silty, trace gravel (< 15 mm) - mottled grey and brown - moist, very stiff - high plasticity - trace silt inclusions (< 2 mm diam.), stiff below 3.7 m - grey below 5.2 m - no gravel below 6.1 m - firm to stiff below 7.3 m	Grab / Shelby Tube	G-08, G-09, G-10, T-11, G-12, G-13, T-14, G-15, G-16	

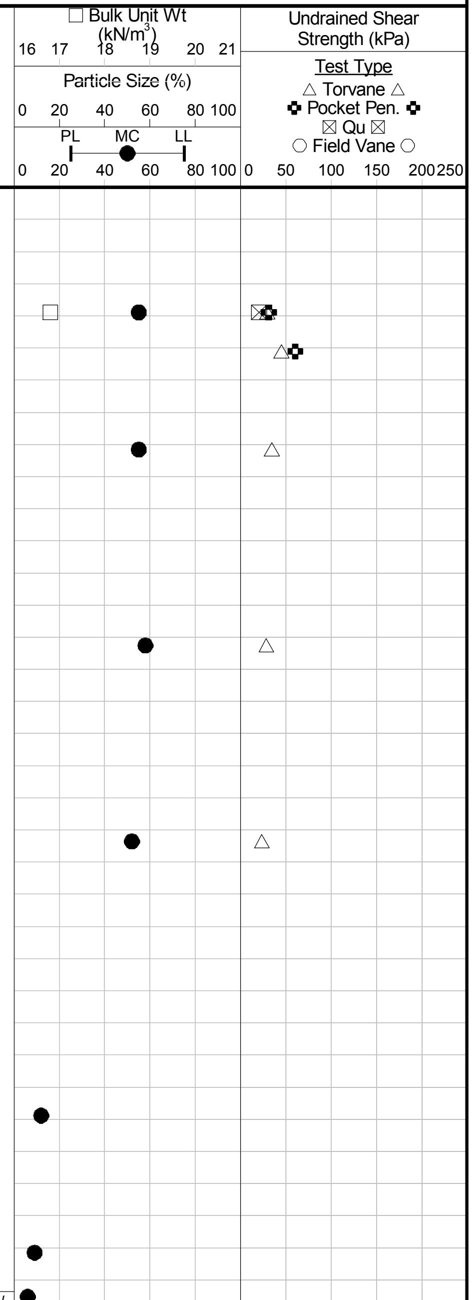


Logged By: Steven Harms **Reviewed By:** Ryan Belbas **Project Engineer:** Ryan Belbas

TREK GEOTECHNICAL

Sub-Surface Log

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
	10.5					
	11.0				T-17	
	11.5					
	12.0		- firm below 11.9 m		G-18	
	12.5					
	13.0					
	13.5		- no silt inclusions below 13.4 m		G-19	
	14.0					
	14.5					
	15.0		- soft below 14.9 m		G-20	
	15.5					
	16.0		- till inclusions below 16.2 m			
	16.5					
83.4	17.0		SILT (TILL) - clayey, some sand, trace gravel (< 25 mm) - light brown - moist, loose to compact		G-27	
	17.5		- trace clay, dense below 17.7 m			
	18.0				G-28	
81.8	18.5		- very dense below 18.6 m		SS-29	100 / 76mm



END OF TEST HOLE AT 18.7 m IN SILT (TILL)
Notes:
1. Power auger refusal at 18.6 m below ground surface.
2. Seepage at 17.0 m below ground surface.
3. Squeezing at 14.6 m below ground surface.
4. Test hole open to 14.6 m below ground surface immediately after drilling.
5. Water level measured at 5.8 m below ground surface immediately after drilling.
6. Test hole backfilled with bentonite and auger cuttings.
7. Test hole elevation in reference to temporary bench mark (manhole rim) located at 14U N-5532012, E-633977, approximately 80 m west of Salter St on Machray Ave.

Logged By: Steven Harms **Reviewed By:** Ryan Belbas **Project Engineer:** Ryan Belbas

Appendix A

Laboratory Testing Results

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Shelby Tube Visual

Project No.	0015-015-00
Client	City of Winnipeg
Project	St.John's Library Addition
Test Hole	TH15-01
Sample #	T11
Depth (m)	4.6 - 5.2
Sample Date	04-Dec-15
Test Date	22-Dec-15
Technician	Matt Medeiros

Tube Extraction

Recovery (mm) 660

Bottom - 5.2 m			Top - 4.6 m
Visual Moisture Content	Kept	Qu γ_{Bulk}	PP Tv
170 mm	160 mm	160 mm	170 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (~<30mm ø)	
trace sand	
Color	brown
Moisture	moist
Consistency	firm to stiff
Plasticity	high plasticity
Structure	homogeneous / blocky
Gradation	

Torvane

Reading	0.53
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	51.5

Pocket Penetrometer

Reading	**1**	1.25
	2	1.25
	3	1.25
	Average	1.25
Undrained Shear Strength (kPa)		61.3

Moisture Content

Tare ID	AB87
Mass tare (g)	6.6
Mass wet + tare (g)	397.8
Mass dry + tare (g)	260
Moisture %	54.4%

Unit Weight

Bulk Weight (g)		1043.1
Length (mm)	**1**	146.99
	2	147.24
	3	146.57
	4	146.42
Average Length (m)		0.147
Diam. (mm)	**1**	72.99
	2	73.02
	3	72.49
	4	73.12
Average Diameter (m)		0.073
Volume (m^3)		6.13E-04
Bulk Unit Weight (kN/m^3)		16.7
Bulk Unit Weight (pcf)		106.3
Dry Unit Weight (kN/m^3)		10.8
Dry Unit Weight (pcf)		68.8

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Unconfined Compressive Strength

ASTM D2166

Project No.	0015-015-00
Client	City of Winnipeg
Project	St.John's Library Addition

Test Hole	TH15-01
Sample #	T11
Depth (m)	4.6 - 5.2
Sample Date	4-Dec-15
Test Date	22-Dec-15
Technician	Matt Medeiros

Unconfined Strength

	kPa	ksf
Max q_u	77.4	1.6
Max S_u	38.7	0.8

Specimen Data

Description CLAY - silty, trace silt inclusions (~<30mm ø), trace sand, brown, moist, firm to stiff, high plasticity, homogeneous / blocky.

Length	146.8	(mm)	**Moisture %**	54%	
Diameter	72.9	(mm)	**Bulk Unit Wt.**	16.7	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	10.8	(kN/m^3)
Initial Area	0.00417	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

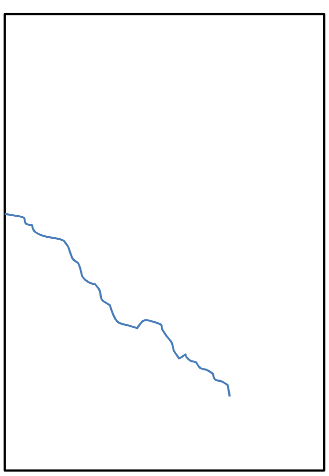
Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.53	51.5	1.08
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	kPa	ksf
	1.25	61.3	1.28
	1.25	61.3	1.28
	1.25	61.3	1.28
Average	**1.25**	**61.3**	**1.28**

Failure Geometry

Sketch:



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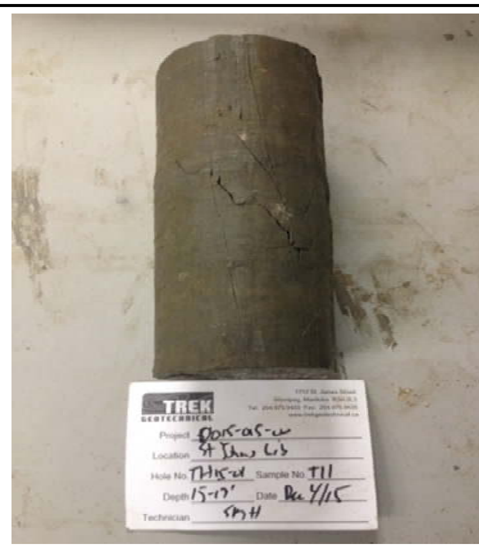

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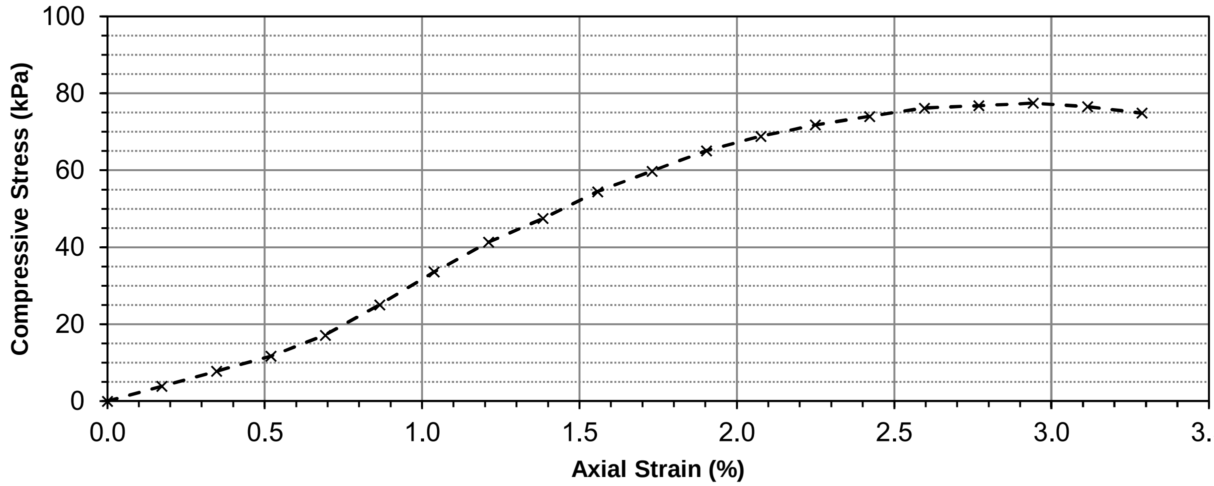
Unconfined Compressive Strength
ASTM D2166

Project No. 0015-015-00
Client City of Winnipeg
Project St.John's Library Addition

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004175	0.0	0.00	0.00
10	5	0.2540	0.17	0.004182	16.3	3.91	1.95
20	10	0.5080	0.35	0.004189	32.7	7.81	3.90
30	15	0.7620	0.52	0.004196	49.1	11.70	5.85
40	22	1.0160	0.69	0.004204	72.1	17.14	8.57
50	32	1.2700	0.87	0.004211	105.5	25.06	12.53
60	43	1.5240	1.04	0.004218	141.8	33.61	16.80
70	53	1.7780	1.21	0.004226	174.7	41.35	20.67
80	61	2.0320	1.38	0.004233	201.1	47.51	23.75
90	70	2.2860	1.56	0.004241	230.8	54.42	27.21
100	77	2.5400	1.73	0.004248	253.9	59.76	29.88
110	84	2.7940	1.90	0.004255	276.9	65.08	32.54
120	89	3.0480	2.08	0.004263	293.4	68.84	34.42
130	93	3.3020	2.25	0.004271	306.6	71.80	35.90
140	96	3.5560	2.42	0.004278	316.5	73.99	36.99
150	99	3.8100	2.60	0.004286	326.4	76.16	38.08
160	100	4.0640	2.77	0.004293	329.7	76.79	38.40
170	101	4.3180	2.94	0.004301	333.1	77.44	38.72
180	100	4.5720	3.11	0.004309	329.7	76.52	38.26
190	98	4.8260	3.29	0.004316	323.1	74.86	37.43

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Shelby Tube Visual

Project No.	0015-015-00
Client	City of Winnipeg
Project	St.John's Library Addition

Test Hole	TH15-01
Sample #	T14
Depth (m)	7.6 - 8.3
Sample Date	04-Dec-15
Test Date	22-Dec-15
Technician	Matt Medeiros

Tube Extraction

Recovery (mm) 700

Bottom - 8.3 m **Top - 7.6 m**

PP Tv	Qu γ_{Bulk}	Kept	Visual Moisture Content
250 mm	160 mm	160 mm	130 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (~<20mm ⌀)	
trace sand	
Color	brown
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	homogeneous
Gradation	

Torvane

Reading	0.58
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	56.9

Pocket Penetrometer

Reading	**1**	1.00
	2	1.25
	3	1.25
	Average	1.17
Undrained Shear Strength (kPa)		57.2

Moisture Content

Tare ID	AB64
Mass tare (g)	6.6
Mass wet + tare (g)	297.8
Mass dry + tare (g)	199
Moisture %	51.4%

Unit Weight

Bulk Weight (g)		1048.8
Length (mm)	**1**	148.22
	2	148.14
	3	148.10
	4	148.16
Average Length (m)		0.148
Diam. (mm)	**1**	72.51
	2	72.24
	3	72.72
	4	72.77
Average Diameter (m)		0.073
Volume (m^3)		6.13E-04
Bulk Unit Weight (kN/m^3)		16.8
Bulk Unit Weight (pcf)		106.9
Dry Unit Weight (kN/m^3)		11.1
Dry Unit Weight (pcf)		70.6

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Unconfined Compressive Strength
ASTM D2166

Project No.	0015-015-00
Client	City of Winnipeg
Project	St.John's Library Addition

Test Hole	TH15-01
Sample #	T14
Depth (m)	7.6 - 8.3
Sample Date	4-Dec-15
Test Date	22-Dec-15
Technician	Matt Medeiros

Unconfined Strength

	kPa	ksf
Max q_u	130.9	2.7
Max S_u	65.5	1.4

Specimen Data

Description CLAY - silty, trace silt inclusions (~<20mm ⌀), trace sand, brown, moist, stiff, high plasticity, homogeneous.

Length	148.2	(mm)	**Moisture %**	51%	
Diameter	72.6	(mm)	**Bulk Unit Wt.**	16.8	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.1	(kN/m^3)
Initial Area	0.00414	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.58	56.9	1.19
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	1.00	49.1	1.02
	1.25	61.3	1.28
	1.25	61.3	1.28
Average	**1.17**	**57.2**	**1.20**

Failure Geometry

Sketch:

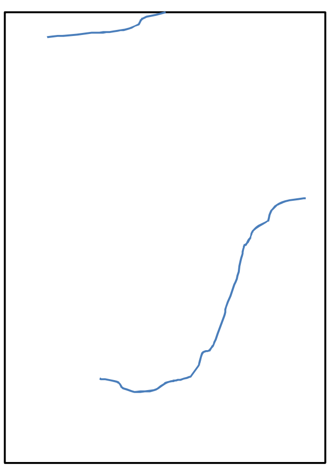

Photo:

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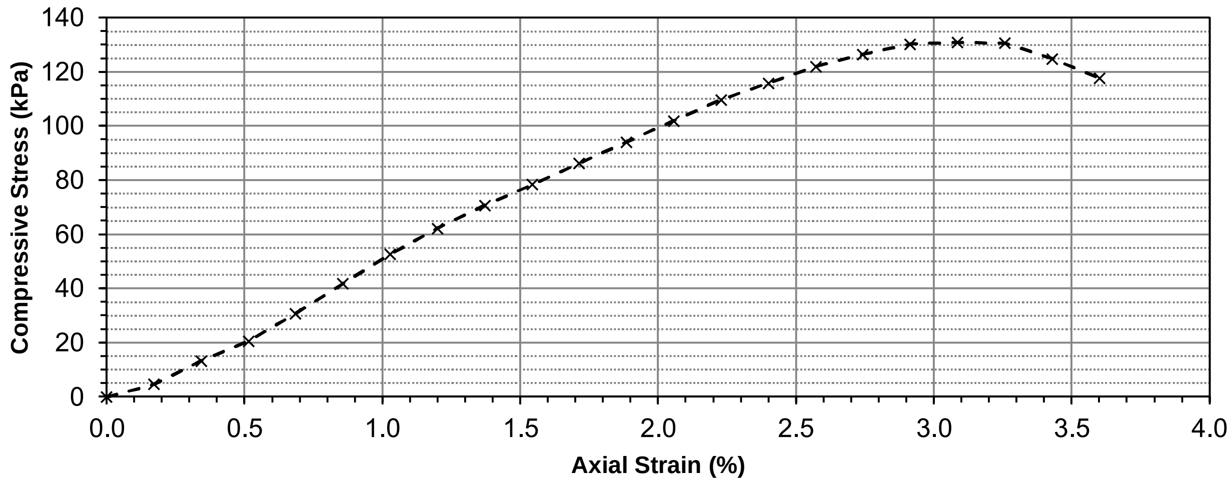
Unconfined Compressive Strength

ASTM D2166

Project No. 0015-015-00
Client City of Winnipeg
Project St.John's Library Addition

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004135	0.0	0.00	0.00
10	6	0.2540	0.17	0.004142	19.6	4.74	2.37
20	17	0.5080	0.34	0.004149	55.7	13.41	6.71
30	26	0.7620	0.51	0.004156	85.7	20.62	10.31
40	39	1.0160	0.69	0.004164	128.6	30.89	15.44
50	53	1.2700	0.86	0.004171	174.7	41.89	20.95
60	67	1.5240	1.03	0.004178	220.9	52.87	26.44
70	79	1.7780	1.20	0.004185	260.4	62.23	31.11
80	90	2.0320	1.37	0.004193	296.7	70.78	35.39
90	100	2.2860	1.54	0.004200	329.7	78.50	39.25
100	110	2.5400	1.71	0.004207	363.4	86.37	43.18
110	120	2.7940	1.89	0.004215	397.0	94.21	47.10
120	130	3.0480	2.06	0.004222	430.7	102.02	51.01
130	140	3.3020	2.23	0.004229	464.4	109.80	54.90
140	148	3.5560	2.40	0.004237	491.4	115.97	57.99
150	156	3.8100	2.57	0.004244	518.3	122.12	61.06
160	162	4.0640	2.74	0.004252	538.5	126.66	63.33
170	167	4.3180	2.91	0.004259	555.4	130.39	65.20
180	168	4.5720	3.09	0.004267	558.7	130.94	65.47
190	168	4.8260	3.26	0.004274	558.7	130.71	65.36
200	161	5.0800	3.43	0.004282	535.1	124.97	62.49
210	152	5.3340	3.60	0.004290	504.8	117.69	58.84

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Shelby Tube Visual

Project No.	0015-015-00
Client	City of Winnipeg
Project	St. John's Library Addition
Test Hole	TH15-01
Sample #	T17
Depth (m)	10.7 - 11.4
Sample Date	04-Dec-15
Test Date	18-Dec-15
Technician	Daniel Wiebe

Tube Extraction

Recovery (mm) 695

Bottom - 11.4 m	11.23 m	11.06 m	10.90 m Top - 10.7 m
Visual Moisture Content PP Tv	Kept	Qu γ_{Bulk}	Visual
170 mm	165 mm	160 mm	200 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (~<20mm ⦰)	
trace sand	
trace gravel (~<15mm ⦰)	
Color	grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	homogeneous / blocky
Gradation	

Torvane

Reading	0.30
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	29.4

Pocket Penetrometer

Reading	**1**	0.70
	2	0.60
	3	0.60
	Average	0.63
Undrained Shear Strength (kPa)		31.1

Moisture Content

Tare ID	N115
Mass tare (g)	8.6
Mass wet + tare (g)	364.6
Mass dry + tare (g)	238.6
Moisture %	54.8%

Unit Weight

Bulk Weight (g)		1087.2
Length (mm)	**1**	153.81
	2	154.14
	3	154.32
	4	154.11
Average Length (m)		0.154
Diam. (mm)	**1**	72.74
	2	72.07
	3	71.92
	4	72.68
Average Diameter (m)		0.072
Volume (m^3)		6.34E-04
Bulk Unit Weight (kN/m^3)		16.8
Bulk Unit Weight (pcf)		107.1
Dry Unit Weight (kN/m^3)		10.9
Dry Unit Weight (pcf)		69.2

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Unconfined Compressive Strength
ASTM D2166

Project No.	0015-015-00
Client	City of Winnipeg
Project	St. John's Library Addition

Test Hole	TH15-01
Sample #	T17
Depth (m)	10.7 - 11.4
Sample Date	4-Dec-15
Test Date	18-Dec-15
Technician	Daniel Wiebe

Unconfined Strength

	kPa	ksf
Max q_u	40.4	0.8
Max S_u	20.2	0.4

Specimen Data

Description CLAY - silty, trace silt inclusions (~<20mm ⌀), trace sand, trace gravel (~<15mm ⌀), grey, moist, firm, high plasticity, homogeneous / blocky.

Length	154.1	(mm)	**Moisture %**	55%	
Diameter	72.4	(mm)	**Bulk Unit Wt.**	16.8	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	10.9	(kN/m^3)
Initial Area	0.00411	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	**kPa**	**ksf**
0.30	29.4	0.61
Vane Size		
m		

Pocket Penetrometer

	Reading	Undrained Shear Strength	
	tsf	**kPa**	**ksf**
	0.70	34.3	0.72
	0.60	29.4	0.61
	0.60	29.4	0.61
Average	**0.63**	**31.1**	**0.65**

Failure Geometry

Sketch:

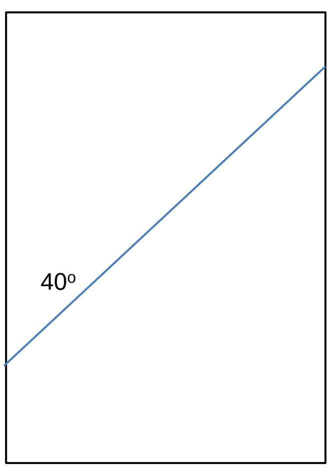
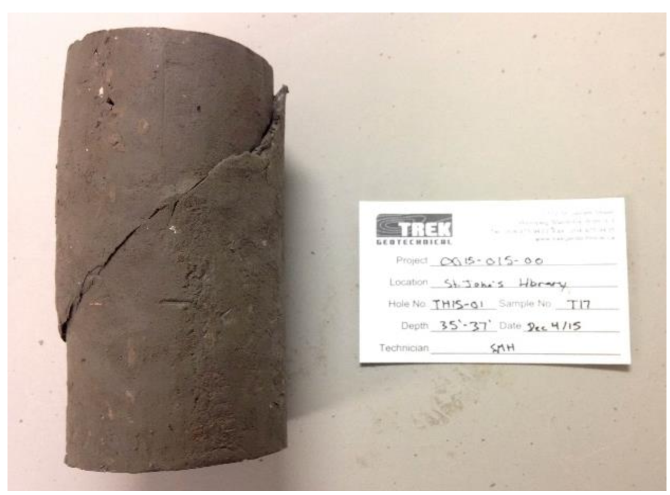


Photo:



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Unconfined Compressive Strength

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Project No. 0015-015-00
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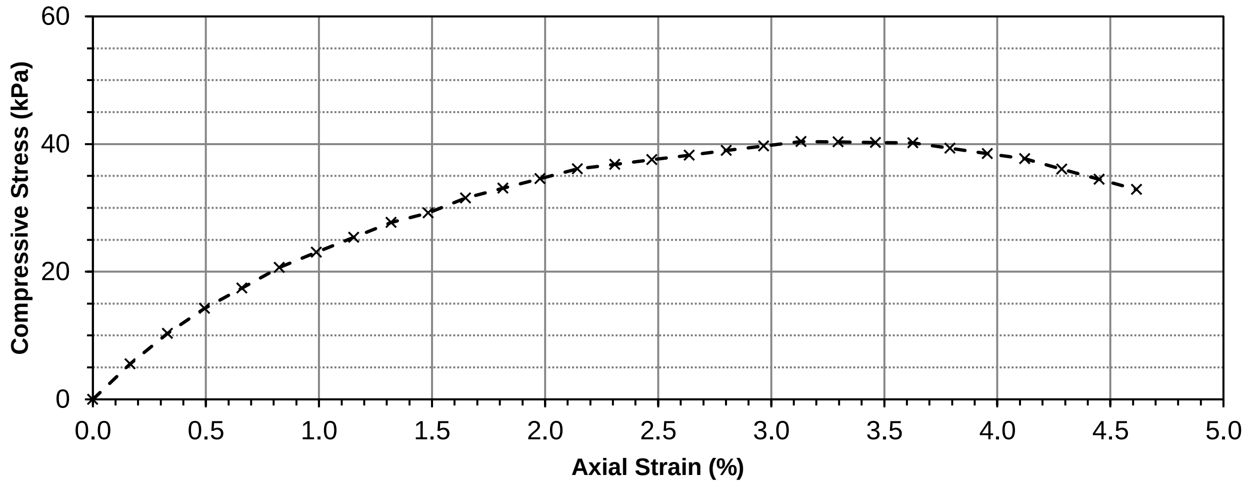
Project St. John's Library Addition

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004111	0.0	0.00	0.00
10	7	0.2540	0.16	0.004118	22.9	5.56	2.78
20	13	0.5080	0.33	0.004125	42.5	10.31	5.16
30	18	0.7620	0.49	0.004132	58.9	14.26	7.13
40	22	1.0160	0.66	0.004139	72.1	17.41	8.71
50	26	1.2700	0.82	0.004146	85.7	20.68	10.34
60	29	1.5240	0.99	0.004153	95.6	23.02	11.51
70	32	1.7780	1.15	0.004159	105.5	25.37	12.68
80	35	2.0320	1.32	0.004166	115.4	27.69	13.85
90	37	2.2860	1.48	0.004173	122.0	29.23	14.61
100	40	2.5400	1.65	0.004180	131.9	31.55	15.77
110	42	2.7940	1.81	0.004187	138.5	33.07	16.53
120	44	3.0480	1.98	0.004194	145.1	34.58	17.29
130	46	3.3020	2.14	0.004201	151.7	36.10	18.05
140	47	3.5560	2.31	0.004209	155.0	36.82	18.41
150	48	3.8100	2.47	0.004216	158.3	37.54	18.77
160	49	4.0640	2.64	0.004223	161.6	38.26	19.13
170	50	4.3180	2.80	0.004230	164.9	38.97	19.49
180	51	4.5720	2.97	0.004237	168.1	39.68	19.84
190	52	4.8260	3.13	0.004244	171.4	40.39	20.20
200	52	5.0800	3.30	0.004252	171.4	40.32	20.16
210	52	5.3340	3.46	0.004259	171.4	40.25	20.13
220	52	5.5880	3.63	0.004266	171.4	40.18	20.09
230	51	5.8420	3.79	0.004273	168.1	39.35	19.67

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Unconfined Compressive Strength
ASTM D2166

Project No. 0015-015-00
Client City of Winnipeg
Project St. John's Library Addition

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	50	6.0960	3.9560	0.004281	164.9	38.51	19.25
250	49	6.3500	4.12	0.004288	161.6	37.68	18.84
260	47	6.6040	4.29	0.004296	155.0	36.08	18.04
270	45	6.8580	4.45	0.004303	148.3	34.48	17.24
280	43	7.1120	4.62	0.004310	141.8	32.89	16.44