Winnipeg

THE CITY OF WINNIPEG

APPENDIX A

Geotechnical Report

TENDER NO. 581-2023

MAIN AND HENRY COMMUNITY CORNER PLAZA – 715 MAIN STREET

Quality Engineering | Valued Relationships

City of Winnipeg
Planning, Property and Development Department

Community Public Washroom
715 Main Street, Winnipeg, MB
Geotechnical Report

Prepared for:
Greg Kucel
Project Officer, Planning, Property and Development Department
City of Winnipeg
4th Floor - 185 King Street, Winnipeg, MB
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Project Number:
0015-040-00

Date:
April 16, 2021

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April 16, 2021

Our File No. 0015-040-00

Greg Kucel
Project Officer, Planning, Property and Development Department
City of Winnipeg
4th Floor - 185 King Street, Winnipeg, MB
R3B 1J1

RE: Community Public Washroom, 715 Main Street, Winnipeg, MB
Geotechnical Report

TREK Geotechnical Inc. is pleased to submit our Geotechnical Report for the above noted project in Winnipeg, MB.

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Nelson Ferreira, M.Sc., Ph.D., P.Eng.
Senior Geotechnical Engineer
Tel: 204.975.9433

Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	RB	January 27, 2021	Final Report
1	RB	April 16, 2021	Final Report with Revised Seal

Authorization Signatures

PROVINCE OF MANITOBA
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Senior Geotechnical Engineer

ENGINEERS
GEOSCIENTISTS
MANITOBA
Certificate of Authorization
TREK GEOTECHNICAL INC.
No. 4877 Date: Apr 16 2021

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1.0 Introduction

This report provides geotechnical design recommendations prepared by TREK Geotechnical Inc. (TREK) for the City of Winnipeg (CoW). TREK's scope of work includes a review of existing geotechnical information and provision of foundation recommendations for a proposed public washroom at 715 Main Street in Winnipeg, MB.

2.0 Background

2.1 Project Description

The proposed development will be located at the northeast corner of Henry Avenue and Main Street approximately 25 m south of the Circle of Life Thunderbird House. The 573 m^2 lot is currently vacant and grass covered. Based on our interpretation of drawings provided by Bridgman Collaborative Architecture, the new development will consist of a three-storey structure comprised of stacked High Cube Corten steel shipping containers. The building will have footprint of 59 m^2 and contain 4 washrooms and an office. The property has been developed over the years with previous structures which have since been demolished. Details of the type, number and locations of previous structures and how they were demolished are unknown. Foundation loads for the proposed structure are also unknown but are anticipated to be relatively light.

3.0 Key Geotechnical Considerations

Key considerations presented within this report include, but are not limited to, the following:

- A sub-surface investigation was not performed for the proposed development. The geotechnical recommendations provided in this report are based TREK's experience in the area and the soils information provided in a geotechnical report prepared by Wardrop (dated February 3, 1999) for the Circle of Life Thunderbird House (Thunderbird House) located 25 m north of the proposed development and Neeginan Park between Higgins Avenue and the CP rail line approximately 110 m north of the development.
- Buried structures and construction debris from previous developments are likely present on site and will impact foundation constructability.

This section should not be relied upon for a complete understanding of design considerations, for which a review of the full report is required.

4.0 Field Program

4.1 Sub-Surface Investigation

Six boreholes (BH-1 to 6) were drilled during the 1999 investigation for the Thunderbird House and Neeginan Park. The boreholes were drilled using a piling rig equipped with a 460 mm diameter auger.

BH-1 to 3 were advanced to power auger refusal between 15.5 and 19 m below grade and BH-4 to 6 were drilled to 6 m depth.

4.2 Soil Stratigraphy

The 1999 geotechnical report (Appendix A) includes borehole logs for BH-1 to 3; logs for BH-4 to 6 were not included in the report. A brief description of the soil units encountered at the site during the 1999 investigation is provided below based on the information provided on the borehole logs. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided in the 1999 geotechnical report.

The soil stratigraphy in descending order generally consists of fill materials, clay and clay till. The fill is 2.5 m thick at the Circle of Life Thunderbird House site and comprised of gravelly clay. The gravelly clay is poorly compacted (soft) and contains concrete rubble and organics. At the Neeginan Park site, the fill is about 3 m thick and consists of silty clay, gravel, and crushed limestone in descending order. The underlying native clay is approximately 10 m thick and is of high plasticity and firm to stiff becoming soft to very soft with depth. The clay till is at a depth of 12 to 13 m below ground surface and extends to at least 19 m below ground surface based on the maximum depth of exploration during the 1999 investigation. The upper 1 to 3 m of the till is moist and very soft. Below, the till becomes gravelly, wet, and compact.

4.3 Power Auger Refusal

Power auger refusal occurred at depths ranging between 15.5 and 19 m below grade in BH-1 to 3 at the which is consistent with our experience in the area. The 1999 geotechnical report indicates that refusal occurred within the till or on bedrock.

4.4 Groundwater Conditions

Groundwater seepage occurred from within the till layer in BH-2 during the 1999 investigation and the static water level was measured at 15.5 m below ground surface at the time of drilling.

These observations 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.

5.0 Foundation Recommendations

Suitable foundations to support the proposed washroom facility based on the sub-surface conditions recorded during the 1999 investigation and the anticipated light loading conditions include:

- CIPC footings or belled piles bearing on clay
- CIPC friction piles in clay
- Driven PPCH piles end bearing in till or on bedrock

Recommendations for these foundation alternatives are provided below according to the NBCC (2010) are provided in the following sections.

5.1 Limit States Design

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

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

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

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

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
Resistance to Vertical Loads for Shallow Foundations (Analysis Methods)	**Resistance Factor**
Vertical resistance by semi-empirical analysis using laboratory and *in situ* test data	0.5

5.2 Shallow Foundations

Shallow foundations in the Winnipeg area are subject to vertical movements associated with moisture and volume changes within the bearing soils. Although difficult to predict, these movements could be in the order of 25 mm or more for spread footings and 50 mm or more for rafts. If these movements are considered unacceptable, a deep foundation system will be required to support the proposed structure.

5.2.1 Footings and Belled Piles

Footings or belled piles bearing on native, undisturbed, firm clay can be designed using a SLS bearing resistance of 75 kPa and a factored ULS bearing resistance of 110 kPa. The native clay was encountered below the fill at depths between 2.5 and 3 m in BH-1 to 3 but the fill thickness may be variable across the site. The SLS bearing resistances are based on limiting settlement to 25 mm or less and the factored ULS bearing resistances were calculated using a resistance factor of 0.5. It should be understood that seasonal movements are different than the settlement required to mobilize the SLS bearing resistances.

The top of the footings and top of the bells should be situated below the fill (estimated to be 2.5 to 3 m below existing grade) to provide resistance to frost heave and to minimize the risk of collapse of the bell due to gravel, rubble, debris, organics and any other deleterious material present in the fill. Footings may be installed by conventional open-cut excavation methods while belled piles can be constructed using a belling tool to form a short, expanded base. Belled piles may be preferable to reduce the costs associated with excavation which could be as deep as 2.5 m or more to remove the fill and install footings on native, undisturbed, firm clay. Footings or belled piles must not be founded on fill materials. It would be beneficial to perform a test belled pile during tendering or at the on-set of construction to assess constructability of this option.

Additional Design Recommendations:

1. Footings and expanded bases should have a minimum width of 0.75 m. Minimum widths must be verified with the applicable building code (e.g. Manitoba Building Code, NBCC).
2. For belled piles the ratio of bell diameter to shaft diameter should not exceed 3 to 1.
3. To minimize changes in moisture of the bearing soils, the water discharge from roof leaders and run-off from exposed slabs and landscaped areas should be directed away from the structure.
4. Footings and belled piles should be designed by a qualified structural engineer to resist vertical (axial), horizontal (lateral), and eccentric (bending) loads from the structure. Belled piles should be designed with full length reinforcement.

Additional Construction Recommendations:

1. All fill, rubble, debris and any other deleterious material should be completely removed such that the bearing surfaces consist of native, undisturbed, firm clay.
2. Excavations for footings should be completed by an excavator equipped with a smooth-bladed bucket operating from the edge of the excavation. The contractor should work carefully to prevent disturbance to the bearing surface at all times.

3. Temporary steel casings (*i.e.* sleeves) installed within the fill will be required to maintain stability of the drilled shaft and to control groundwater seepage when constructing belled piles. 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.
4. Final bearing surfaces should be inspected and documented by TREK prior to concrete placement to verify the adequacy of the bearing surface and proper installation of the foundation unit.
5. The bearing surfaces should be protected from freezing, drying, or inundation with water at all times. If any of these conditions occur, the disturbed zone must be scarified such that the bearing surface consists of native, undisturbed, firm clay.
6. Concrete must be placed under dry conditions. For belled piles, concrete should be placed in one continuous operation immediately after the completion of drilling the pile hole and forming the bell to avoid potential construction problems such as sloughing, caving, or groundwater seepage. Concrete can be placed by free-fall methods if the pile hole is dry. Concrete placed by free-fall methods should be directed through the middle of the pile shaft and steel reinforcing cage to prevent striking of the drilled shaft walls to protect against soil contamination of the concrete. If groundwater is encountered, it should be controlled or removed. If water cannot be controlled or removed, the concrete should be placed using tremie methods.
7. Footings should be backfilled with non-frost susceptible soils (clean, granular fill) above the insulation and compacted to 98% of the Standard Proctor Maximum Dry Density (SPMDD).

5.3 Deep Foundations

5.3.1 Cast-In-Place Concrete Friction Piles

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

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

<table>
<tr><th rowspan="3">Pile Depth Below Existing Site Grade
(m)</th><th rowspan="3">SLS Unit Resistance
(kPa)</th><th colspan="3">Factored ULS Unit Resistance (kPa)</th></tr>
<tr><th colspan="2">Compression
$\phi = 0.4$</th><th>Uplift
$\phi = 0.3$</th></tr>
<tr><th>Shaft Adhesion</th><th>End Bearing (Note 1 & 2)</th><th>Shaft Adhesion</th></tr>
<tr><td>0 to 3</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>3 to 7</td><td>8</td><td>10</td><td>-</td><td>8</td></tr>
<tr><td>7 to 11</td><td>7</td><td>8</td><td>40</td><td>7</td></tr>
</table>

1. *For piles with a diameter of less than 1.0 m. If larger pile diameters are required TREK should be contacted to provide revised end bearing values.*
2. *Piles must be installed at least 8 m below final grade.*

Additional Design Recommendations:

1. The weight of the embedded portion of the pile may be neglected.
2. Piles should be designed with a maximum depth of 11 m below existing ground surface to avoid penetration into the underlying till and to protect against heaving at the base of the pile shaft. In the event the till is encountered at shallower depths, the pile design may have to be re-evaluated by the structural engineer.
3. Piles should have a minimum spacing of 3 pile diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
4. Piles should be designed by a qualified structural engineer for the anticipated axial (compression and tension), lateral and bending loads induced from the structure as well as forces induced from seasonal movements (i.e. shrinkage/swelling and frost-related movements) of the bearing soils.

Additional Construction Recommendations:

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

5.3.2 Driven Precast Prestressed Concrete Hexagonal Piles

Precast prestressed concrete hexagonal piles driven to practical refusal in dense till or on bedrock will derive their resistance primarily from end bearing with a relatively small contribution for shaft friction. Table 3 provides SLS and factored ULS capacities for PPCH piles driven to practical refusal on dense till or bedrock. Piles designed based on the SLS resistances are expected to exhibit less than 10 mm of settlement at the pile toe. Elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement.

Table 3. Recommended Factored ULS and SLS Capacities 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

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 should be expected to vary across the site and may be deeper than encountered during drilling and as indicated on the test hole logs. It is possible that PPCH piles reach refusal on bedrock. There is also a potential of damaging PPCH piles due to the presence of cobbles and boulders within the till and this should be accounted for when considering this pile type.

Dynamic pile load testing (i.e. PDA testing with CAPWAP analysis) is recommended on driven piles during installation to verify pile capacity and set criteria, measure driving stresses and delivered energy, and evaluate pile integrity. If PDA testing with CAPWAP analysis is performed, a resistance factor of 0.5 can be used for design of the factored ULS capacities; if PDA testing with CAPWAP analysis is not performed, a resistance factor of 0.4 must be used. A dynamic monitoring program should consist of testing about 5% of the piles; however, the scale of the testing program will depend on the number and sizes of piles to be installed. The dynamic monitoring program can be established by TREK prior to construction after the pile design and layout has been determined.

The piles should be driven to at least three consecutive sets of the refusal criteria outlined in Table 2, 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.

<u>Additional Design Recommendations:</u>

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. Factored ULS unit axial-uplift resistances provided in Table 2 above can be used for design.
4. Piles should be designed by a qualified structural engineer to withstand the anticipated axial (compression and tension), lateral and bending loads induced from the structure, handling stresses, driving stresses, and tensile forces induced from seasonal movements (i.e. shrinkage/swelling and frost-related movements) of the bearing soils.

<u>Additional Construction Recommendations:</u>

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 and be of sufficient strength to meet manufacturer strength requirements and resist stresses that may be encountered during installation.
4. Pre-boring should be completed to reduce ground vibrations and protect against heave of, and consequently damage to, adjacent buildings. 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 may need to be deeper for piles driven near existing buildings (e.g. Circle of Life Thunderbird House and Salvation Army).
5. Piles should be driven continuously once driving is initiated to the required refusal criteria.
6. 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.
7. Re-driving of all piles in groups and at the discretion of the geotechnical engineer of record (TREK) 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.
8. 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).
9. 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. Dynamic load testing 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.
10. Adjacent buildings should be monitored for heave, vibrations, and damage during pile driving. A detailed inspection of the adjacent foundations by a qualified structural engineer should be completed prior to pile driving.

5.3.3 Ad-freezing Effects

Concrete piles, pile caps, and grade beams subjected to freezing conditions should be designed to resist ad-freeze and uplift forces related to frost action acting along the vertical face of the member within the depth of frost penetration (2.5 m). In this regard, buried concrete may be subject to an ad-freeze bond stress of 65 kPa within the depth of frost penetration. Ad-freeze forces will be resisted by structural dead loads and uplift resistance provided by the portion of the footing and length of the pile below the depth of frost penetration. The following design recommendations apply to piles subject to ad-freeze forces:

1. An ad-freeze bond stress of 65 kPa within the depth of frost penetration (2.5 m).

2. A load factor (α) of 1.2 may be used in the calculation of ad-freezing forces.
3. A reduction factor of 0.8 may be used in calculation of the geotechnical resistance for the factored ULS condition with an ultimate (nominal) uplift resistance of 25 kPa between 3 and 7 m depth and 20 kPa below. Resistance to ad-freezing within the depth of frost penetration should be neglected.
4. Structural dead loads should be added to the resistance.
5. The calculated geotechnical resistance plus the structural dead loads must be greater than the factored ad-freezing forces.
6. Piles subject to ad-freezing forces should be a minimum of 8.0 m or as calculated by the method above, whichever is greater.
7. Measures such as flat lying rigid polystyrene insulation could be considered to reduce frost penetration depths and thereby ad-freezing and uplift forces.

5.3.4 Pile Caps and Grade Beams

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

5.3.5 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 A23.1-14 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56-day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-14 for concrete with severe sulphate exposure (S2). Concrete that may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-14.

5.3.6 Foundation Inspection Requirements

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

a) continuous basis during:

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

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

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 sub-surface conditions be encountered. TREK, as the geotechnical engineer of record, should be retained to observe the installation of all foundation elements.

6.0 Concrete Slabs

6.1.1 Grade-Supported Slabs

If some movement can be tolerated, grade-supported concrete floor slabs can be used. Vertical deformation of grade-supported slabs should be expected due to volumetric changes in the underlying sub-grade soils (i.e. swelling and shrinkage). Although difficult to predict these movements could be in the order of 25 mm or more. Slabs in unheated areas or near the perimeter of the structure will be subject to additional movements from freeze/thaw of the sub-grade soils. If these movements cannot be tolerated, a structural floor slab will be required.

Additional recommendations:

1. Fill, rubble, debris, and any other deleterious material (e.g. concrete rubble) should be stripped such that the sub-grade consists of native, undisturbed, firm clay. Based on our soil's information in the 1999 geotechnical report, it is anticipated that this may require removal of 2.5 m of soil or more. Assuming that this will not be practical from a cost or constructability perspective and provided the potential for increased risk of seasonal movements is recognized, the sub-grade can consist of clay fill provided the upper 300 mm of the fill is scarified, moisture conditioned, and recompacted to 95% of the SPMDD.
2. After excavation, scarification and compaction, the sub-grade should be field reviewed and proof-roll inspected by TREK prior to placement of sub-base and base materials.
3. The prepared sub-grade surface should be protected from freezing, inundation, drying, or disturbance at all times. If any of these conditions occur, the sub-grade should be scarified, moisture conditioned, and re-compacted to a minimum of 95% of the SPMDD.
4. In heated areas, the floor slab should be placed on a 150 mm thick sub-base layer of 50 mm down crushed granular base course underlying a 150 mm thick base layer consisting of 20 mm down crushed granular base course. In unheated areas (e.g. exterior slabs) the thickness of 50 mm down crushed granular sub-base should be increased to 250 mm. The crushed granular materials should be placed in lifts no greater than 150 mm and compacted to 98% of the SPMDD. The granular base course materials should consist of a well-graded, durable crushed limestone, in accordance with the City of Winnipeg Specification No. CW 3110.
5. 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. To accommodate slab movements, it may be desirable to provide control joints to reduce random cracking and isolation joints to separate the slab from other structure elements.

Allowances should be made to accommodate vertical movements of light weight structures (e.g. partitions) bearing on the slab.

6.2 Structural Slabs

In areas where movement of floor slabs is not tolerable, a structural floor slab should be used. A minimum void of 150 mm should be provided underneath the 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 can consist of a compressible layer (e.g. void form) that is capable of deforming a minimum of 150 mm without transferring stress to the floor slab or, alternatively, a crawl space. A vapour barrier should be placed between the floor slab and the void form (if present).

7.0 Site Drainage

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

All paved areas should be provided with minimum slopes of 2% to improve long-term drainage. The water discharge from roof leaders and run-off from exposed slabs should be directed away from the structures.

8.0 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) by the Client. Soil conditions are natural deposits that can be highly variable across a site. If sub-surface 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 a mutually executed 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 relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Appendix A

Existing Geotechnical Report

Mr. John D'Ignazio, M.Arch., MAA
City of Winnipeg
Property and Development Services Department
Fourth Floor – 180 King Street
Winnipeg, MB R3B 3G3

January 8, 1998
99155601-00

Dear Mr. D'Ignazio:

**Re: Revised Proposal for Geotechnical Study And Other Engineering Services
Proposed Neeginan Round House and Neeginan Park
North Main Street Redevelopment – Winnipeg, Manitoba**

Wardrop Engineering is pleased to submit this revised proposal to provide engineering services related to the above-referenced site. We can complete this work for a fixed fee of $7,500, excluding the GST.

Our qualifications, proposed staff, methodology, work schedule, and costs are detailed below.

QUALIFICATIONS

Wardrop Engineering is a Canadian-owned, multidisciplined consulting firm with a staff of 300 and offices in Winnipeg (Head Office), Saskatoon, Toronto, and Thunder Bay, as well as Africa and Asia. Wardrop has been involved in engineering projects for over 40 years, and is recognized as one of the foremost firms in the country. Over the past ten years, Wardrop has undertaken Environmental Site Assessments (ESAs) of over 1,000 locations across Canada. Wardrop has conducted numerous geotechnical investigations during this time, as well.

We have conducted similar work for the City of Winnipeg including Fire Training Tower (geotechnical), Public Transit System ESA; Sludge Beds ESA; Public Markets Site and Ross Avenue Yards ESAs; and Lyndale Drive Road Failure (geotechnical). We have also completed a ground water supply feasibility study of the Sandilands Aquifer area for the City. The project involved the drilling of 30 boreholes and installation of 20 monitoring wells to determine the subsurface soil stratigraphy and hydrogeological characteristics of the area, east of Winnipeg.

Further, we have completed ESAs for real estate and legal firms (Sun-X, Colliers Pratt McGarry, Fillmore Riley, Buchwald Asper, Manulife, Garric Management, Pitblado & Hoskins); many financial institutions (Royal Bank, National Bank, Bank of Nova Scotia, and numerous credit unions), most of the major oil companies (Shell Canada, Imperial Oil, Petro-Canada, Husky, Co-op/Tempo, and Domo), and railways (Canadian Pacific, Canadian National and VIA Railways), among many others, including government departments, agricultural facilities, and insurance firms.

PROPOSED STAFF

This work will be undertaken by qualified staff of Wardrop's Winnipeg office. The Project Manager will be Mr. Mike Wiebe, P.Eng., a Senior Environmental Engineer, experienced in conducting both ESAs and Geotechnical Investigations. Mr. Joe Hyrich, B.Sc., Project Engineer, will conduct the data collection and field work, and assist Mr. Wiebe with reporting. Mr. Ed Wolowich, M.Sc., P.Eng., a Principal of Wardrop and Manager of the Firm's Environmental Earth Sciences Department, will have overall responsibility for this project. He will review the report, prior to submission, to ensure that it has been produced in accordance with Wardrop's strict standards for both technical accuracy and good engineering practice; however, his involvement will be minimal.

INVESTIGATIVE METHODOLOGY

We understand that the scope of services includes the following:

- Geotechnical Survey and Analysis
- Environmental Site Assessment
- Topographical Survey

Parcel A

The subject properties are at the southeast corner of Main Street/Higgins Avenue (not including Bunzy's Autobody and the Royal Hotel). This block is currently being acquired by the City of Winnipeg on behalf of the North Main Street Task Force for the purpose of constructing a Round House and Medicine Wheel Plaza. At present, a number of buildings on this site are undergoing demolition with an expected completion early in December 1998.

Parcel B

The second property is on the north side of Higgings, east of Main Street and west of the Aboriginal Centre and will become the site for Neeginan Park. The Aboriginal Centre currently owns this property.

Geotechnical Study

We propose the drilling of a minimum of four boreholes at the Parcel A site and two boreholes at the Parcel B site to characterize the geotechnical properties of the soil. If subsurface conditions are found to be inconsistent, warranting the drilling of additional boreholes, the City will be contacted while on-site. Since additional boreholes would be at an additional cost, this work will not be undertaken until approval is obtained from the City.

...3

Prior to beginning the drilling program, we will arrange to have all on- and off-site services (underground utility corridors) located and marked.

Two of the boreholes advanced at the Parcel A site and one of the boreholes at the Parcel B site will be drilled to auger refusal, at an expected depth of 24 m below grade.

The remaining boreholes will be drilled to an estimated depth of 6 m below grade, or until any silt or other deleterious material (i.e., organics) is no longer encountered.

We propose to engage Subterranean (Manitoba) Limited of Winnipeg, Manitoba, for the borehole drilling program. The boreholes will be augered using a piling rig, equipped with 460 mm diameter solid stem, continuous-flight augers.

During the drilling program, disturbed samples will be removed directly from the auger bit (dependent on soil conditions), every 1.5 m for examination and will be placed in plastic sampling bags. At least two undisturbed soil samples will be collected from the boreholes advanced to refusal using a thin-walled Shelby Tube sample. The sample will be subjected to pocket penetrometer and field torvane testing to estimate the undrained shear strength of the soils. These values will approximate the bearing capacity of the soils and will be useful for the design of the proposed developments at each site. All of soil samples collected will be placed in freezer storage, in the event that the additional testing is required in the future.

Three undisturbed soil samples will be submitted to a local laboratory for unconfined compression testing. This test will provide confirmatory undrained shear strength valves for the soils, necessary for foundation design. The majority of disturbed soil samples will be submitted for moisture content determination.

Each borehole will be carefully logged during drilling. The detailed borehole logs will include soil stratigraphy descriptions based on the Unified Soil Classification System, as well as visible composition, odour descriptions, and sample depth. If any abnormolous odours are experienced, the soil sample in question will be subjected to hexane vapour concentration screening, using a combustible gas meter, to check for petroleum hydrocarbon contamination.

Following the completion of each borehole, the borehole will be backfilled with drill cuttings and grouted near grade to limit the downward migration of any surface water or possible future contaminants.

All borehole locations will be referenced to on-site property boundaries, and plotted on a site plan.

Environmental Site Assessment (ESA)

We propose to undertake an ESA (Phase I) of the Parcel A property in accordance with CMHC's Environmental Site Investigations Procedures, Canadian Standards Association Z751 and Z768 protocols, as well as ASTM 1404 Phase I Auditing Standards, and applicable Provincial Regulations

...4

and Guidelines. These standards will address all of the issues normally raised by legal firms, financial institutions, and regulators.

An ESA addresses issues of potential environmental concern at a particular site and would include:

- a review of available construction drawings and historic information;
- regulatory file searches;
- an interview with the property owners/managers or long-term maintenance staff;
- a site inspection; and
- reporting.

We propose to conduct Manitoba Environment and Manitoba Workplace Safety and Health file searches to determine whether there have been environmental or health and safety violations.

An interview with the present property manager will be undertaken to assess whether past or present practices at the sites or on the adjacent properties could be of environmental concern. Such practices might include storage of hazardous materials and disposal of wastes.

We will perform an inspection of the site to determine if there are any potential environmental concerns regarding the various legislation relating to environmental matters, including, but not limited to the following:

- The Environment Act;
- The Dangerous Goods Handling and Transportation Act;
- The Public Health Act;
- The Waste Reduction and Prevention Act; and
- The Ozone Depleting Substances Act.

In this regard, we will note any chemical and material storage, waste disposal, occupancy conditions, site drainage, wastewater disposal, and land use of adjacent properties that may impact this site.

Topographic Survey

Using electronic Total Station survey equipment, a topographic survey will be conducted for each of Parcels A and B. Ground elevations will be determined using a 6 m grid spacing. The elevations of property lines, curbs, adjacent lanes and roadways, as well as the other items listed in the Terms of Reference will be determined. The survey information will be provided in hard copy and in AutoCAD™ format on computer disk. The survey and drawings will be in S.I. units.

REPORTING

A letter report will be prepared for each site, detailing the above investigations and findings, which will be sealed by a Professional Engineer. This document will be suitable for submission to Manitoba Environment, any financial institution, or any other third party. We will keep you informed of our progress throughout this project, and contact you immediately if additional boreholes or unusual odours or staining are encountered during drilling. We will report any important results as they become available (by facsimile).

SCHEDULE

We propose to conduct the data collection portion of the geotechnical study and the ESA by January 15, 1998. This work includes the site inspection, personnel interviews (if available), and borehole drilling. A draft report will be produced for your review by January 29, 1999. It may take up to six weeks from initiation of the project to receive a written response from Manitoba Environment and Manitoba Workplace Health and Safety, detailing the information they may have on file. Upon receipt of this information, a final report, will be issued, incorporating the results of these file searches and any comments you may have had concerning the initial draft report.

INSURANCE COVERAGE

Wardrop maintains the following minimum insurance coverage (letters from our insurers will be provided upon request):

- Professional Liability, including Environmental Liability (Errors and Omissions)
  - Limit: $7,500,000 each claim and aggregate annually (including Environmental Impairment)
- Commercial General Liability (Third Party Liability, including nonowned automobile)
  - Limit: $10,000,000.

Wardrop is in "Good Standing" with the Workers Compensation Board and a letter from them to that effect will be provided to the City of Winnipeg, upon request.

COST

We propose to complete the requested geotechnical study, ESA, and topographic surveys for a fixed fee of $7,500, not including GST. These costs are detailed in Table 1, attached.

Should any additional work be required beyond that proposed, it will be undertaken on an hourly basis, with disbursements charged at cost plus 10% for handling. Unit rates for fees and disbursements are listed in Table 2, attached.

TERMS AND CONDITIONS

We propose to undertake this work under our normal conditions for pollution-related services, attached.

Thank you for this opportunity to submit this proposal. Should you have any questions regarding this matter, please contact me or Mr. Ed Wolowich.

Sincerely,

WARDROP ENGINEERING INC.

M. Wiebe

M.P. Wiebe, P.Eng.
Project Manager
Environmental Earth Sciences

MPW/pp

Copy Mr. Ed Wolowich, Wardrop
Mr. Joe Hyrich, Wardrop

TABLE 1 Cost for Provision of Engineering Services City of Winnipeg	
Task	**Fees**
Engineering Fees:	
Geotechnical/Historical Data Review	$ 600
Site Inspection/Soil Sampling	750
Personnel Interviews	100
Topographic Survey	1,500
Data Analyses and Reporting	1,800
Subtotal Estimated Fee	4,750
Disbursements:	
Project Supplies	100
Regulatory File Search	450
Equipment Rental (Total Station)	150
Printing, Photocopy, Phone, Fax, Computers, etc.	300
Borehole Drilling (3 deep holes, 3 shallow holes)	1,200
Soil Testing	300
Subtotal	2,500
10% Handling	250
Subtotal Disbursements	2,750
TOTAL FIXED FEE COST	**$7,500**
Note: GST is not included in the above costs	

TABLE 2 Unite Rates City of Winnipeg	
Item	**Rate**
Fees:	
Mr. Mike Wiebe	$ 75/hour
Mr. Joe Hyrich	60/hour
Mr. Ed Wolowich	100/hour
Drafting/Survey Technician	45/hour
Administration	40/hour
Disbursements:	
Boreholes	120/hour
Unconfined Compression Testing	75/sample
Moisture Content	10/sample
Total Station	150/day

GENERAL CONDITIONS FOR POLLUTION-RELATED SERVICES

Acceptance of our engineering services by the client shall limit the liability of Wardrop Engineering to:

- Claims related to, or attributed to, proven negligence by Wardrop Engineering in the performance of its services.
- The amount of our Professional Liability Insurance in effect.

The client shall indemnify and hold harmless Wardrop Engineering, its subsidiaries and related companies, officers, employees, agents, and invitees from and against all:

- Claims arising out of the actual, alleged, or threatened discharge, dispersal, release, explosion, fire, or escape of pollutants, caused by the client's negligence.
- Loss, cost or expense, arising out of any governmental direction, or request that Wardrop Engineering test for, monitor, cleanup, remove, contain, treat, detoxify, or neutralize any pollutants.
- Fines, penalties, punitive or exemplary damages, arising directly or indirectly out of the discharge, dispersal, release, explosion, fire, or escape of any pollutants, caused by the client's negligence.
- Liability, whether in contract or tort, for loss or damage occasioned by delays beyond our control or for loss of earnings, loss of production, loss of use, or other consequential damage, howsoever caused, limited to the dollar amount of this contract.

For the purposes of the above clauses, the following definition shall apply.

- "Pollutants" - any solid, liquid, gaseous, or thermal irritant or contaminant, including, but not limited to smoke, vapour, soot, fumes, acids, alkalis, chemicals, and waste. Waste includes materials to be recycled, reconditioned, or reclaimed.

WARDROP Engineering Inc.

TRANSIT DOCUMENT

To: The City of Winnipeg
Property and Development Services
4th floor-180 King St.
Attention: Mr. John D'Ignazio

From: Gary Dingle
Ph: 956-0980 Fax: 957-5389

Date: February 3, 1999

Reason: As requested by you

Proj. No.: 991556-01-00

Copy to: File

Proj. Name: Neeginan Park Topographical Survey

Quantity	Drawing/Reference No.	Description
1	991556-01-00	Site Plan
1		Cad file (3 ½" floppy)

Remarks:

Sincerely,

Gary Dingle
WARDROP ENGINEERING INC.

WARDROP Engineering Inc.

Mr. John D'Ignazio, M.Arch., MAA
City of Winnipeg
Property and Development Services Department
Fourth Floor – 180 King Street
Winnipeg, MB R3B 3G3

February 3, 1999
99155601-00

DRAFT

Dear Mr. D'Ignazio:

Re: Proposed Neeginan Round House and Park Sites – Geotechnical Investigation

Wardrop Engineering performed geotechnical investigations of the above-referenced sites, in Winnipeg, Manitoba. The geotechnical investigations were performed to determine the soil and ground water conditions, and to provide recommendations on the geotechnical aspects for the proposed building and park structures. The investigative methodology, site geology, and foundation design recommendations are detailed in this report (in draft).

SITE CONDITIONS

The proposed location of the Neeginan Round House and Medicine Wheel Plaza (Parcel A), at the southeast corner of Main Street and Higgins Avenue, consists of mainly vacant, previously developed property. Bunzy's Autobody and the Royal Hotel, situated near the northeast corner of the site are scheduled to remain. Fill is present at the site to a depth of 2.5 m below grade. Piles from former structure foundations may be buried at the site. The proposed Neeginan Park site (Parcel B) consists of vacant grassland and is located at the northeast corner of Main Street and Higgins Avenue. Fill is present at the site to a depth of 2.5 m below grade. Piles from former structure foundations may be buried at the site.

INVESTIGATIVE METHODOLOGY

A combined total of six boreholes were drilled at the sites, as shown on Figure 1, attached. Boreholes (BH)-1 to BH-3 were drilled to refusal. The boreholes were drilled to refusal in the till or on bedrock. The remaining BH-4 to BH-6 were drilled to a depth of 6 m below grade. The boreholes were drilled on January 12 and 13, 1999, using a piling rig, equipped with a 460 mm diameter, auger bit. The auger bit was advanced and removed at 0.6 m intervals, with soil samples taken every 1.5 m. Upon completion of the boreholes, they were left open for a minimum of 10 minutes to allow the ground water level to stabilize. BH-1 was left open for approximately one hour. After the ground water levels were recorded, the boreholes were backfilled with drill cuttings and bentonite to grade. Any remaining soil was spread out on the site and the area was restored to initial conditions.

During drilling, the soils encountered were logged according to the Unified Soils Classification (USC) system, which includes descriptions regarding soil colour, composition, density, and moisture content. Pocket penetrometer and torvane tests were performed on the ends of the Shelby tubes, to determine the relative undrained shear strength of the clay. Detailed borehole logs are attached.

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Both disturbed and undisturbed soil samples were recovered during drilling. Disturbed soil samples were recovered from the auger bit and sealed in plastic bags. The undisturbed soil samples were recovered using thin walled Shelby tubes pressed into the bottom of the borehole as the hole progressed. The Shelby tube samples were sealed with paraffin wax and wrapped with plastic to prevent moisture loss. Selected soil samples were forwarded to the laboratory for moisture content, and unconfined compression testing. The laboratory's report is attached.

SITE GEOLOGY

The regional soil stratigraphy of the Winnipeg area generally comprises an upper complex zone of fissured clay and silt, overlying lake bottom clays, which overlies glacial till. The overburden is deposited on predominately carbonate (limestone and dolomite) bedrock.

Within the sites, a layer of gravelly clay fill was found at grade. The upper complex zone of silty clays was encountered under the fill at a depth of 2.5 m below grade. A thin layer of topsoil was found at grade, at the Parcel B site.

The lake bottom clay was encountered at a depth ranging from 2.5 to 13.3 m (on average) below grade. This clay is typically highly plastic, stiff-to-firm, moist, and contains silt inclusions and lenses. The upper portion of the clay is weathered light brown, and is typically more fractured than the lower grey or olive-grey clay. The colour difference of the clay is attributed to past weathering. Sulphate pockets (gypsum) were observed within the clay in all of the boreholes. The transition to the olive-grey clay was observed at a depth of between 7.0 and 8.2 m below grade. The olive-grey clay becomes soft to very soft with depth as the till is approached. A silty clay till containing some gravel was encountered at a depth ranging from 11.9 and 14.6 m below grade.

The unconfined compression tests indicated that the clay has an undrained shear strength of between 21 and 31 kPa, which is within the typical range for the clays within Winnipeg (Baracos, et-al, 1983). The torvane and pocket penetrometer tests indicated average undrained shear strengths of 24 and 54 kPa, respectively. The results of the torvane and pocket penetrometer are not considered accurate and are used as a relative indication of strength. The moisture content of the clay varied from 31 to 60.6%, which is typically between the plastic and liquid limits.

The clayey gravel till was encountered at an average depth of 15.3 m below grade. The till is a heterogenous mixture of silt, sand, clay, and gravel, with occasional cobbles and boulders. The till found during drilling was described as wet and medium dense. The moisture content of the till is between 8.0 and 15.6%, which is typical for the upper loose or soft till. Drilling refusal (possible bedrock) occurred at a depth of between 15.5 and 18.9 m below grade.

Ground water was present within the till in BH-2 and the ground water level in the borehole rose rapidly when the till was encountered. That static ground water level in the till (or bedrock) was recorded to be 15.5 m below grade.

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DESIGN CONSIDERATIONS

We understand that the Round House will be a 6000 ft^2 single-storey structure with a structural floor slab and crawlspace.

Assuming that the Round House crawlspace will not be heated and that the foundation will be subject to frost action, some movements are anticipated if a slab foundation is used. We understand that large movement will not be tolerated and do not recommend the use of a slab foundation. We recommend the use of driven precast piles, or alternatively end bearing caissons, or cast-in-place friction piles.

Driven Precast Concrete Piles

The design capacity of hexagonal precast concrete piles driven to practical refusal in the till or bedrock are as follows:

Pile Diameter	Maximum Design Capacity	Practical Refusal
305 mm (12")	445 kN (100 kips)	5 blows per 25 mm
356 mm (14")	625 kN (140 kips)	8 blows per 25 mm
406 mm (16")	800 kN (180 kips)	12 blows per 25 mm

The piles must be driven with a properly sized hammer of at least 40 kJ to practical refusal, as defined above. An experienced inspector should be present during the installation of the piles.

The pile spacing should be a minimum of 2.5 pile diameters, centre to centre, and all piles within groups should be restruck to counter the effects of pile heave. Preboring up to a depth of 6 m from existing grade should be undertaken in order to aid in pile alignment during driving and to reduce the impact of driving on adjacent structures and underground utilities. Monitoring of any underground hallways in adjacent structures for vibrations should be considered during driving.

Caissons on Till or Bedrock

Cast-in-place caissons, end bearing on dense till or bedrock may be an alternative to driven precast piles. Design bearing pressures of 720 kPa can be used for caissons founded on dense till and 2800 kPa for caissons founded on sound competent bedrock. The quality of the bedrock must be confirmed by coring a drill hole at the caisson locations. The minimum shaft diameter would be 760 mm to allow manual entry and inspection.

The rapid rise in the ground water in one of the boreholes upon drilling into the till, indicates that ground water inflows may be large. The concrete may have to be placed by the tremie method. The anticipated high ground water inflows limits the viability of the use of caissons, and for this reason driven precast piles are recommended over caissons.

Cast-in-Place Concrete Friction Piles

Cast-in-place concrete friction piles may be designed using the allowable skin friction values as follows:

Depth Below Basement Level	Skin Friction
0 to 3.0 m	0 kPa
3.0 to 5.0 m	8.5 kPa
5.0 to 11.5 m	7.0 kPa

Pile spacing should be a minimum of 3 pile diameters, measured centre to centre. Piles should contain adequate reinforcement and be of adequate structural design. Piles beneath the parkade or unheated areas should be reinforced along their entire length. Pile lengths exceeding 13 m, as measured from the existing grade, are not recommended, as they may encounter ground water seepage from the underlying till. The minimum pile length should be 6 m under a heated floor and 8 m under a nonheated floor, as measured from the base of the excavation. The minimum pile diameter is 350 mm.

The silt which was occasionally encountered in the test holes may be saturated, and sloughing and/or seepage may occur during pile drilling. Temporary steel casing should be available on site and utilized as required to maintain a clean and dry pile hole.

Excavation Considerations

The excavation for the Round House should extend approximately 3 m below grade and below any fill. The Manitoba Labour Guideline for Excavation Works must be followed. Should the excavation extend beyond approximately 5 m below grade, the potential for basal heave, due to aquifer artesian pressures, must be reviewed.

The excavation will extend through the surficial silts and clays into the highly plastic clay. The silt layers may produce water. The excavation must not be flooded, and any water accumulating within the excavation should be pumped out.

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Frost action and swelling of the clay is expected beneath the Round House structural floor slab, and a minimum of 0.4 m of void form is recommended.

A subdrainage system consisting of weeping tiles should be installed around or beneath the Round House crawlspace to control the inflow of water. The basement walls should be damp proofed and designed to resist lateral earth pressures. Figure 2 outlines the lateral earth pressures anticipated on the basement walls, and assumes a free draining sand and gravel backfill and a subdrainage system.

It is likely that the clays beneath the former buildings have lost moisture following construction. These clays may swell with the addition of moisture, and heaving of the excavation floor may occur. It is recommended that a vapour barrier be used to minimize changes in moisture conditions. A 50 mm layer of sand should be placed over the vapour barrier as protection.

Concrete

Water soluble sulphates (gypsum crystals) were observed during drilling. Sulphate resistant Portland cement (Type 50) should be used for all concrete in contact with the soil. Concrete exposed to freeze-thaw cycles should be adequately air entrained to improve freeze-thaw durability.

LIMITATIONS

The scope of this assessment report is limited to the matters expressly covered and is intended solely for the client to whom it is addressed. Wardrop makes no warranties, expressed or implied, including without limitation, as to the marketability of the site, or fitness for a particular use. The assessment was conducted using standard engineering and scientific judgement, principles and practices within a practical scope and budget. It is based on the observations of the assessor during the time of the site visit, in conjunction with archival information obtained from a number of sources, which is assumed to be correct. Except as provided, Wardrop has made no independent investigations to verify the accuracy or completeness of the information obtained from secondary sources or personal interviews. Generally, the findings, conclusions and recommendations are based on a limited amount of data (e.g., the number of boreholes drilled and samples analyzed), and that actual conditions on the property may vary from those described above. Any findings regarding site conditions different from those described above, upon which this report is based, will consequently change Wardrop's conclusions and recommendations.

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DISCLAIMER

This document has been prepared in response to a specific request from the client to whom it is addressed. The contents of this document are not intended for use of, nor is it intended to be relied upon, by any person, firm, or corporation other than that client of Wardrop Engineering Inc. to whom it is addressed. Wardrop Engineering Inc. denies any liability whatsoever to other parties who may obtain access to this document, or for damages or injury suffered by such third parties arising from the use of this document by them, without the express prior written authorization of Wardrop Engineering Inc. and its client who has commissioned this document.

We trust that this letter report meets with your requirements, and would be pleased to discuss the results with you.

Sincerely,

WARDROP ENGINEERING INC.

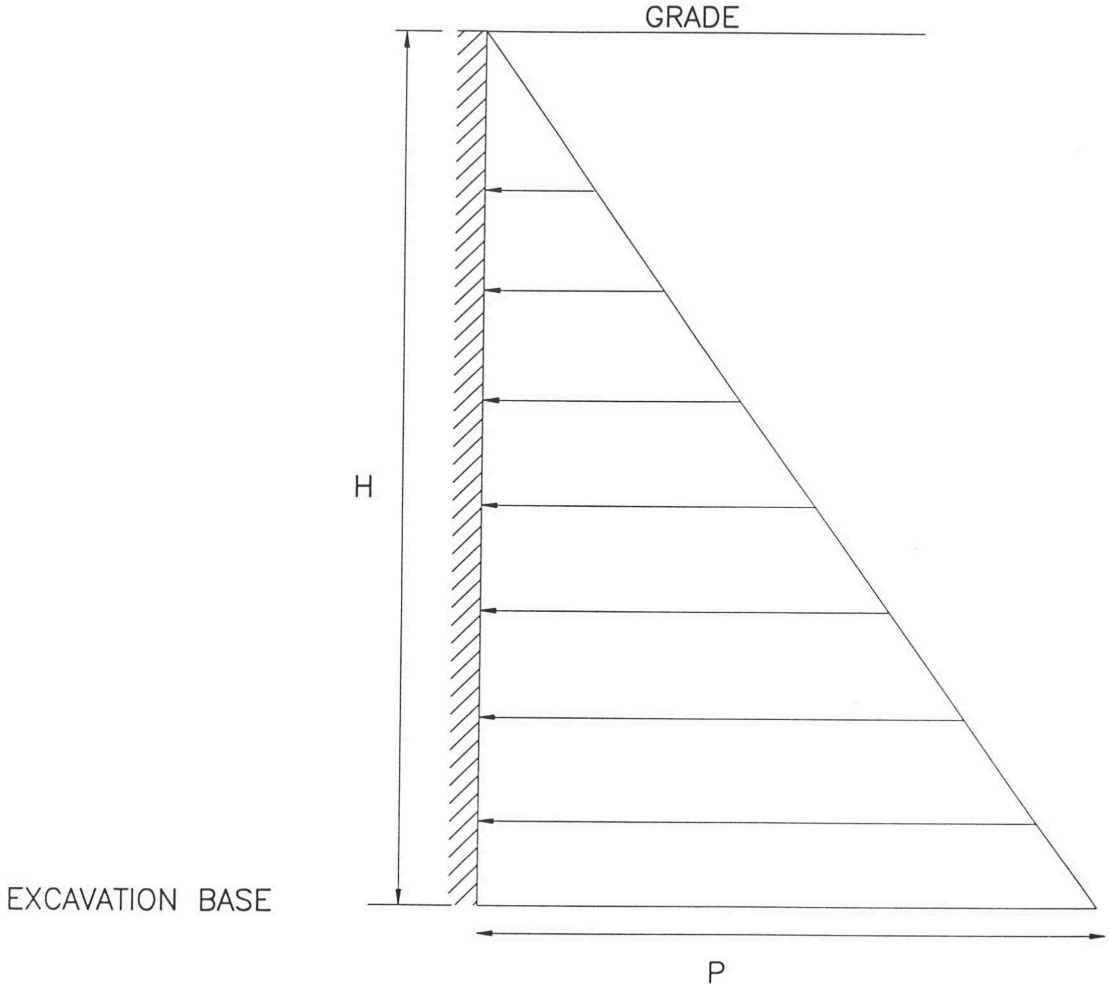
M. Wiebe

M.P. (Mike) Wiebe, P.Eng.
Project Manager
Environmental Earth Sciences

MPW/pp

Attachments

Copy Mr. Ed Wolowich, Wardrop Engineering
Mr. Joe Hyrich, Wardrop Engineering



$$P = K_0 \gamma H$$

WHERE,

P = DESIGN LATERIAL EARTH PRESSURE

K_0 = AT REST EARTH PRESSURE COEFFICIENT (0.5)

γ = UNIT WEIGHT OF SOIL (20 kN/m FOR GRANULAR SOIL)

H = DEPTH OF EXCAVATION

CITY OF WINNIPEG

GEOTECHNICAL STUDY
NEEGINAN ROUND HOUSE AND PARK

Drawn:	P.C. Finnbogason
Designed:	M.P. Wiebe
Checked:	M.P. Wiebe
Date:	99/02/03
Scale:	As Noted
PROJ.	991556-01-00

LATERAL EARTH PRESSURE DISTRIBUTION
BASEMENT WALLS

FIGURE 2

0
REV.

WARDROP Engineering Inc.
Winnipeg • Toronto • Thunder Bay • Saskatoon

WARDROP ENGINEERING INC.

BOREHOLE LOG: BH-1

Logged By: J.D.H.
Drawn By: J.D.H
Checked By: M.P.W.

Client: City of Winnipeg
Project No: 991556-01-00
Borehole Location: Northwest Corner of Parcel A

Project: Neeginan Round House
Date Drilled: January 12, 1999

SUBSURFACE PROFILE					SAMPLE		SHEAR STRENGTH (kPa)			Moisture Content (%)
Elevation	Depth (m)	Well Data	USC Symbol	Description	Number	Type	Pocket Pen.	Field Vane	Lab Comp.	Wp |----O----| Wl (20 40 60 80 100)
232	0			Ground Surface						
230	0–2.5			**Gravelly Clay Fill** brown; slightly moist; loose - soil frozen to around 0.7 m below grade - some concrete rubble at 1.8 m below grade	1, 2, 3					
225	2.5–7			**Clay (CH)** silty with gypsum pockets, brown; moist; medium stiff - soft-to-medium stiff, very moist at around 6 m below grade	4, 5, 6					
220	7–12			**Clay (CH)** silty, grey; very moist; soft	7, 8, 9					
217	12–15.5			**Clay Till (CL)** clay with silt and gravel inclusions, grey; very moist; very soft - less gravel inclusions at 13.7 m below grade	10, 11, 12					
	15.5–			**Clayey Gravel Till (GC)** light grey clay and course gravel; wet; medium dense - refusal on rock at 15.5 m below grade	13					
				End of Borehole						

Sample	Pocket Pen.	Field Vane	Lab Comp.
3	172	56.4	51.7
8	221	4.9	41.8

Drilling Contractor: Subterranean (Manitoba) Limited

Drill Type: Piling Rig (0.46 m diameter augers)

Datum: Geodetic

Well Casing Elevation: Well Not Installed

Water Table Elevation: Hole Dry

Date Measured: January 12, 1999

Well Materials: Hole Backfilled

WARDROP ENGINEERING INC.

BOREHOLE LOG: BH-2

Logged By: J.D.H.
Drawn By: J.D.H
Checked By: M.P.W.

Client: City of Winnipeg
Project No: 991556-01-00
Borehole Location: Near Centre of Parcel A

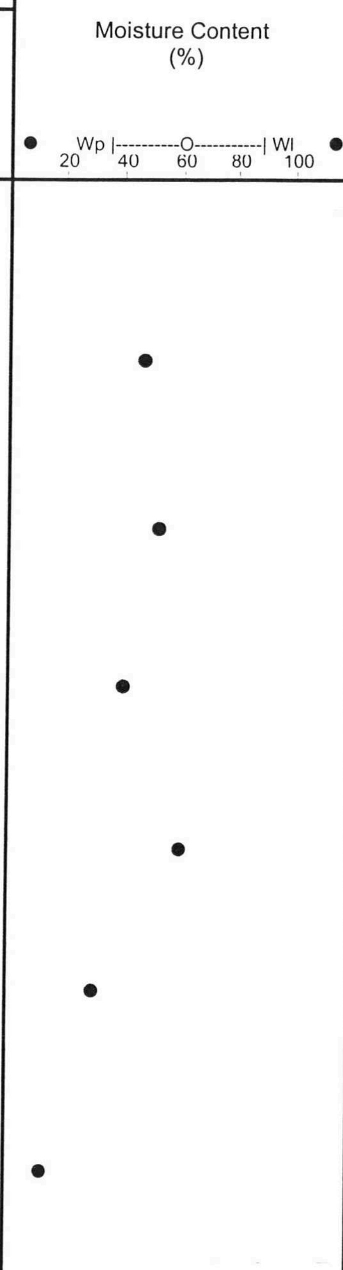
Project: Neeginan Round House
Date Drilled: January 12, 1999

SUBSURFACE PROFILE

Elevation	Depth (m)	Description
232	0	Ground Surface
		Gravelly Clay Fill gravel and silty clay, brown-black; dry; relatively loose - soil frozen to around 0.4 m below grade - some topsoil in upper portion of layer
229		**Clay (CH)** silty with gypsum pockets, brown; moist; medium stiff - soft-to-medium stiff, very moist at around 5 m below grade
224		**Clay (CH)** silty, grey; very moist; soft
218		**Clay Till (CL)** clay with silt and fine gravel inclusions, grey; very moist; very soft - light grey silt pocket at 14.9 m below grade
216		**Clayey Gravel Till (GC)** light grey clay and silt and course gravel; wet; medium dense - saturated at around 16 m below grade - refusal on rock at 18.9 m below grade
213		

SAMPLE / SHEAR STRENGTH (kPa)

Number	Type	Pocket Pen.	Field Vane	Lab Comp.
1				
2				
3				
4				
5				
6		66	44	NT
7				
8				
9				
10				
11		12.3	24.5	NT
12				
13				
14				
15				
16				



Drilling Contractor: Subterranean (Manitoba) Limited

Drill Type: Piling Rig (0.46 m diameter augers)

Datum: Geodetic

Well Casing Elevation: Well Not Installed
Water Table Elevation: 15.5 m below grade
Date Measured: January 12, 1999
Well Materials: Hole Backfilled

WARDROP ENGINEERING INC.

BOREHOLE LOG: BH-3

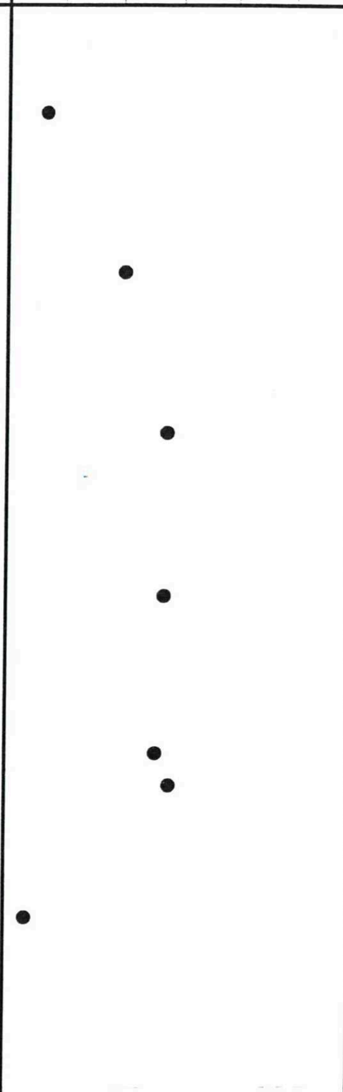
Logged By: J.D.H.
Drawn By: J.D.H
Checked By: M.P.W.

Client: City of Winnipeg
Project No: 991556-01-00
Borehole Location: Near Centre of Parcel B

Project: Neeginan Round House
Date Drilled: January 12, 1999

Elevation	Depth (m)	Description	Sample Number	Sample Type	Pocket Pen. (kPa)	Field Vane (kPa)	Lab Comp. (kPa)
232	0	Ground Surface					
	0–1.2	**Silty Clay Fill (Topsoil)** black; frozen	1				
230	1.2–2.0	**Gravel Fill** light brown; dry; loose					
230	2.0–3.0	**Crushed Limestone Fill** crushed limestone; dry	2				
224	3.0–8.0	**Clay (CH)** silty with gypsum pockets, brown; moist; medium stiff - pocket of grey clayey silt; soft-to-medium stiff, from 3.35 to 3.8 m - soft-to-medium stiff, very moist at around 6 m below grade	3, 4, 5, 6		147	118	NT
218	8.0–14.5	**Clay (CH)** silty, grey; very moist; soft	7, 8, 9, 10, 11		24.5	34.3	62.3
217	14.5–15.3	**Clay Till (CL)** clay with silt and fine gravel inclusions, grey; very moist; very soft	12				
215	15.3–18.9	**Gravelly Clay Till (CL)** mixture of light grey clay and course gravel; wet; soft - gravel content increasing with depth - refusal on rock at 18.9 m below grade	13, 14				
		End of Borehole					

Moisture Content (%): Wp |----O----| Wl; 20 40 60 80 100



Drilling Contractor: Subterranean (Manitoba) Limited

Drill Type: Piling Rig (0.46 m diameter augers)

Datum: Geodetic

Well Casing Elevation: Well Not Installed
Water Table Elevation: Hole Dry
Date Measured: January 12, 1999
Well Materials: Hole Backfilled

Wardrop Engineering Inc.
Soil Sample Analyses
Neeginan Round House #991556-01-00

January 25, 1999
WX-06552
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TABLE 1 SOIL SAMPLE ANALYSIS PROJECT NO. 991556-01-00		
Test Hole No.	Sample Depth (ft)	Moisture Content (%)
BH1	4	21.8
BH1	15	56.4
BH1	25	47.4
BH1	35	46.5
BH1	45	41.8
BH1	51	15.6
BH2	9.5	47.1
BH2	20	52.5
BH2	40	60.6
BH2	49	31.0
BH2	60	13.2
BH2	30	40.5
BH3	5	13.9
BH3	15	41.4
BH3	25	56.0
BH3	35	55.0
BH3	45	52.3
BH3	55	8.0
BH4	8.5	26.0
BH4	20	52.6
BH5	5	20.9
BH5	15	57.1
BH6	13	52.2
BH6	20	57.1

TABLE 2 UNCONFINED COMPRESSION TESTS PROJECT NO. 991556-01-00					
Test Hole No.	Sample Depth (ft)	Moisture Content (%)	Unit Weight (wet-kg/cu.m)	Unit Weight (dry-kg/cu.m)	Unit Load (Kpa)
BH1	32	44.9	1699	1172	41.8
BH1	11.5	51.4	1663	1098	51.7
BH3	47	57.2	1731	1101	62.3