Addendum #1

Consultant: Scatliff + Miller + Murray
Date: June 2, 2025
Owner: City of Kenora
Project Number: 711-001-24
Title: ITT #711-001-24 Central Park Greenspace Construction

The following answers, clarifications, and changes to the Invitation to Tender documents form part of the ITT process and shall be included in all considerations relating to the Proponent's submission.

Item #	Reference	Answer/Clarification/Change
1	Appendix E – Geotechnical Report	Attached to this addendum is Appendix E containing a geotechnical report for the Central Community Club property. It shall be incorporated into the end of the Invitation to Tender.

CITY OF KENORA
INVITATION TO TENDER

ITT #711-001-24

Appendix E
Geotechnical Report

TREK GEOTECHNICAL Quality Engineering | Valued Relationships

Solid Construction Inc.

Central Community Club, Kenora, ON
Geotechnical Report

Prepared for:
Nigel Grammer
Lead Estimator
Solid Construction Inc.
61 Tailleau Road, Kenora, ON
P9N 3W8

Project Number: 0814-001-00

Date: November 4, 2021

TREK GEOTECHNICAL
Quality Engineering | Valued Relationships

November 4, 2021

Our File No. 0814-001-00

Nigel Grammer
Lead Estimator
Solid Construction Inc.
61 Tailleau Road, Kenora, ON
P9N 3W8

RE: Central Community Club, Kenora, ON
Geotechnical Report

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

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

RBelbas

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

Encl.

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Revision History

Revision No.	Author	Issue Date	Description
0	Matt Klymochko	November 4, 2021	Final Report

Authorization Signatures

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1.0 Introduction

This report provides geotechnical design recommendations for Solid Construction Inc. (Solid) prepared by TREK Geotechnical Inc. (TREK) for the proposed Central Community Centre development, located at 730 1st Street South in Kenora, ON. The terms of reference for this work are included in our contract dated September 10, 2021. The scope of work includes a sub-surface investigation, laboratory testing, and provision of geotechnical design and construction recommendations for the proposed development.

2.0 Background Information

2.1 Project Description

The proposed development consists of a new clubhouse, outdoor ice rink, volleyball court, bocce ball courts, and parking area. The clubhouse is anticipated to be in the order of 190 m^2 (2,050 ft^2) in size. TREK understands that a thickened-edge slab and helical piles are the preferred foundations for the clubhouse. Foundation loads are unknown but are anticipated to be relatively light. TREK also understands that it is preferred to have the outdoor ice rink placed on a concrete slab.

2.2 Existing Information

A site development plan was provided by Solid and used in development of our geotechnical program.

3.0 Key Geotechnical Considerations

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

- A shallow foundation system (footings, thickened-edge slab) is deemed to be unsuitable to support the clubhouse due to the presence of compressible peat and clay soils within the practical depth of construction.
- Decomposition of organics (peat) below the new club house is expected to produce methane gas during degradation. A methane mitigation system may be required to eliminate toxic gases, or methane monitoring may be required following construction.
- Sloping bedrock may be present within the footprint of the proposed clubhouse which will impact the installation of helical piles and possibly pile integrity and capacity. If sloping bedrock is encountered, pipe piles socketed into bedrock may be required to replace helical piles.

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

A sub-surface investigation was completed on September 30, 2021 under the supervision of TREK personnel to determine the soil stratigraphy and groundwater conditions at the site. Seven test holes (TH21-01 to 07) were drilled and sampled to depths ranging between 1.5 and 12.7 m below ground surface as part of the investigation at the locations shown on Figure 01. The test holes were drilled by Paddock Drilling Ltd. using an Acker MP5-T geotechnical drill rig mounted on a Morooka MST 1500 track-mounted carrier equipped with 125 mm diameter solid stem augers and 170 mm diameter hollow stem augers. The test holes were backfilled with auger cuttings and bentonite chips.

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

Test hole locations were determined by handheld GPS. Test hole elevations were surveyed using a rod and level relative to a temporary benchmark assigned an arbitrary elevation of 100.0 m. The temporary benchmark selected was the top nut of a fire hydrant (denoted as TBM-1 on Figure 01). The UTM coordinates of each test hole are provided on the test hole logs. The test hole logs also include a description of the soil units encountered and other pertinent information such as groundwater and sloughing conditions and a summary of the laboratory testing results. Laboratory test results are included in Appendix A.

4.2 Stratigraphy

Brief descriptions of the soil units encountered at the test hole locations are provided below. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the attached test hole logs.

The sub-surface stratigraphy consists of surficial fill soils overlying organics (peat), silty clay and sand. The fill soils consist of organic clay (topsoil), silt or sand, extending to approximately 0.5 m depth, with the exception of TH21-01, where fill soils extended to 1 m depth. Peat was encountered below the fill in all test holes, extending to a depth ranging between 2.3 m and 3 m. The peat is typically orange to dark brown containing trace to some rootlets and is moist to wet with different degrees of humifaction, ranging from fibrous material (H2-H3), to amorphous material, (H5-H6), based on the von Post classification. The clay underlying the peat is silty, moist, soft to firm and of high plasticity, becoming very soft with depth. Sand was encountered in TH21-01 and TH21-02 at a depth of 5.5 m, extending to 12.7 m, the maximum depth of exploration. The sand contains trace gravel, is wet, compact, poorly graded, and typically coarse grained. Boulders or sloping bedrock were suspected below the clay at depths of 4.6 and 4.9 m below ground surface in TH21-04 and 07, respectively.

However, this could not be verified due to the drilling method used. Additionally, soil samples could not be recovered below these depths.

4.3 Power Auger Refusal

Power auger refusal was not observed during drilling. However, the augers began move significantly out of plumb during drilling of TH21-04 and TH21-07 at depths of 4.6 m and 4.9 m below ground surface, respectively. It is considered likely that the auger tip was sliding along sloping bedrock at each of these locations. Holes were terminated shortly after observing this to prevent damage to the augers.

4.4 Groundwater Conditions

Seepage and sloughing conditions were encountered at depths of 3.0, 2.9 m, 3.0 m, and 3.7 m, in TH21-01, 02, 03, and 04 respectively.

These groundwater measurements should not be considered reflective of (static) long-term groundwater levels, which would require monitoring over an extended period to determine. It is important to recognize that groundwater conditions may change seasonally, annually, or due to construction activities.

5.0 New Clubhouse

5.1 Foundation Recommendations

Helical piles end bearing in compact sand and pipe piles socketed into bedrock are suitable foundations to support the new club house based on the sub-surface and anticipated loading conditions. Recommendations for these pile types in accordance with the National Building Code of Canada (NBCC, 2015) are provided in the following section. A shallow foundation system (footings, thickened-edge slab) was evaluated but deemed to be unsuitable to support the clubhouse due to the presence of compressible peat and clay soils within practical depth of construction.

5.2 Limit States Design (NBCC, 2015)

Limit states design recommendations for deep foundations in accordance with the National Building Code of Canada (2015) 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 (2015) 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, 2015)

Resistance to Axial Loads for Deep Foundations (Analysis Methods)	ϕ
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

5.3 Helical Piles

Installation of helical piles may be difficult or not feasible if sloping bedrock or boulders are encountered. Installing helical piles on sloping bedrock or boulders may result in misalignment of piles, pile damage, or low bearing capacity all of which will impact foundation performance. Sloping bedrock may have been encountered within TH21-04, in this regard, the selection of this pile type should be carefully considered based on the associated increased risk. It may be more cost effective to plan for installation of helical piles with the understanding that rock socketed pipe piles may be required at select locations if installation of helical piles is unsuccessful.

5.3.1 Compressive Capacity

Helical piles installed in compact sand will derive their resistance primarily from end bearing with a relatively small contribution from shaft friction. The design and selection of pile and helix dimensions, depth, and capacity should be performed by an experienced supplier/contractor, familiar with installing helical piles in Kenora, and reviewed by TREK. For preliminary design purposes, the factored ULS and SLS axial capacity of helical piles installed in compact sand can be approximated by the formulas provided below. Piles designed based on the SLS resistances are expected to exhibit less than 25 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.

1. **Nominal End Bearing Capacity (kN) = $(N_{q*}\ \gamma' H) \times \pi \times (D_{helix}^2 - D_{shaft}^2)/4$**
2. **SLS End Bearing Capacity (kN) = $1/3 \times (N_{q*}\ \gamma' H) \times \pi \times (D_{helix}^2 - D_{shaft}^2)/4$**
3. **ULS End Bearing Capacity (kN) = $\Phi_r \times (N_{q*}\ \gamma' H) \times \pi \times (D_{helix}^2 - D_{shaft}^2)/4$**

Where:

N_{q*} = Bearing capacity factor (a value of 20 should be used at this site).

γ' = Effective unit weight (a value of 7 kN/m^3 should be used at this site).

H = Helix embedment depth below final grade (m).

D_{helix} = Helix diameter (m)

D_{shaft} = Pile shaft (pipe) outer diameter (m)

Φ_r = ULS resistance factor (a factor of 0.4 should be used unless a static pile load test is performed at the project site).

The above equation assumes that the groundwater level is 2 m below ground surface, a conservative assumption based on the limited data available.

TREK has provided preliminary SLS and factored ULS capacities for commonly available helical piles installed to depths of 9 and 12 m below existing ground surface within compact sand in Table 2.

Table 2: Recommended ULS and SLS Pile Capacities for Common Helical Pile Sizes

Pile Size – Shaft Diameter (m) x Helix Diameter (m)	Factored ULS Capacity (kN) $\Phi_r = 0.4$		SLS Capacity (kN)	
	9 m	12 m	9 m	12 m
0.089 x 0.305	35	45	28	37
0.166 x 0.458	72	96	60	80

5.3.2 *Uplift Capacity*

The uplift capacity of helical piles at both the factored ULS and SLS can be taken as 75% of the factored ULS capacity as outlined above.

5.3.3 *Additional Design and Construction Recommendations*

1. The weight of the embedded portion of the pile may be neglected in the design.
2. The pile must be designed to withstand all design loads and handling stresses during installation.
3. 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.
4. Piles should be installed under the supervision of TREK Geotechnical personnel to observe static load testing and installation.

5. Torque should be measured and recorded during installation to verify proper installation as established by static load testing; however, torque should not be used as a direct measurement of pile capacity.

5.4 Steel Pipe Piles Socketed into Bedrock

Steel pipe piles socketed into sound, un-weathered, intact bedrock are a suitable foundation system. The depth to bedrock was not verified during the sub-surface investigation and is expected to vary across the site since sloping bedrock was suspected during drilling of TH21-04 and 07. In this regard, it may be warranted to perform an additional sub-surface investigation consisting of bedrock coring to increase certainty of pile lengths and minimize the risk of cost overruns during construction if this alternative is preferred.

The piles can be installed by lowering the pipe into the bottom of a pre-drilled and grout-filled hole or by using rotary and percussion hammer methods and injecting grout through the bottom of the pile. These methods are commonly used to install steel pipes into bedrock. Other methods for installing the pipe piles may be considered but must be reviewed and approved by TREK prior to pile installation. It is important that an experienced contractor be retained as proper installation and grouting methods can affect performance. Bearing resistances (compressive and uplift) are provided for this pile type in the following sections.

5.4.1 *Compressive Capacity*

Steel pipe piles socketed into bedrock will derive a majority of their compressive resistance in end bearing with a relatively small contribution form shaft friction. The factored ULS axial capacity of a steel pipe pile socketed into bedrock is based on the structural strength of the steel section and can be calculated using the following formula, which includes application of a resistance factor of 0.4:

$$0.4f'_yA_p$$

Where,

f'_y = yield stress of the steel

A_p = cross-sectional area of the pipe

Pile settlements under service loads are expected to be less than 5 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.

5.4.2 *Uplift Capacity*

The uplift capacity of pipe piles socketed and grouted into bedrock will depend on the bond strength between the grout and steel surface of the pile or the grout and bedrock surface, whichever is lower. The bond strength between the grout and the pile can be calculated based on a factored ULS uplift bond stress 185 kPa. The bond strength between the grout and the bedrock can be calculated based on a factored ULS bond stress of 0.03f'$_c$ (f'$_c$ = compressive strength of the grout). For calculation of uplift capacity, the bond stress is to be applied only to surface area of the pipe embedded within the bedrock.

5.4.3 Additional Design and Construction Recommendations

1. The weight of the embedded portion of the pile may be neglected in the design.
2. Piles should be socketed to a minimum depth of 0.5 m or three socket diameters (whichever is greater) into sound, un-weathered, intact bedrock.
3. Temporary steel casings (sleeves) must be installed to the top of competent bedrock to install pipe piles in pre-drilled and grout-filled holes to protect against sloughing of the pile hole and/or to control groundwater seepage. The casing may be removed once the pile has been installed into the rock and grouted, provided it can be removed without disturbing the pile. It may be required to delay casing removal until the grout has achieved sufficient strength to maintain pile alignment and avoid damage during casing withdrawal.
4. Pipe piles installed in a pre-drilled and grout-filled hole must be free of soil or rock cuttings and any other deleterious material prior to grout placement.
5. Pipe piles installed in a pre-drilled and grout-filled hole must be placed in the centre of the hole and securely on the base of the socket.
6. Proper measurements should be taken during grouting to verify that the complete filling of the drill hole has occurred.
7. Grouting should be completed as soon as possible after drilling.
8. Pile verticality (plumbness) should be measured on all piles 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. Piles should be grouted to ground surface to ensure compliance with surrounding soils along the entire pile length, in particular if lateral resistance is required

5.5 Lateral Resistance

The soil response (sub-grade 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 for preliminary design of pile foundations. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral sub-grade reaction modulus (K_s) as provided in Table 3. 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.

Table 3: Recommended Values for Lateral Sub-grade Reaction Modulus

Depth Below Existing Site Grade (m)	Soil Type	Lateral Subgrade Reaction Modulus Ks [kN/m³]
0 to 3	Fill	$\frac{4400z}{d}$
0.5 to 3.0	Peat	-
3.0 to 5.5	Clay	$\frac{870z}{d}$
5.5 to 12.7	Sand	$\frac{4400z}{d}$

Note 1: d is pile diameter in metres
Note 2: z = depth in metres

It should be understood that using the lateral sub-grade 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 cycling), and
- pile material behaves linear elastically (does not reach yield conditions).

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 preliminary design, a lateral pile analysis that incorporates 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 should be carried out by TREK to confirm the lateral load capacity of the piles.

5.6 Ad-freezing Effects

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, concrete structures may be subject to an ad-freeze bond stress of 65 kPa within the depth of frost penetration and steel structures may be subject to an ad-freeze bond stress of 100 kPa. Ad-freeze forces will be resisted by structural dead loads and uplift resistance provided by the length of the pile below the depth of frost penetration (2.5 m).

The following design recommendations apply to piles subject to ad-freeze forces:

1. An ad-freeze bond stress of 65 kPa for concrete and 100 kPa for steel 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 factored ULS condition based on the following nominal geotechnical resistances:
    a. Helical Piles – 75% of the nominal end bearing capacity (formula 1 in Section 5.3)
    b. Pipe Piles – Ultimate bond strength of 460 kPa between the grout and the pile or the ultimate bond strength 0.1f'c (f'c = compressive strength of the grout) between the grout and the bedrock within the rock-socketed portion of the pile, whichever is less.

4. Resistance to ad-freezing within the depth of frost penetration should be neglected from design
5. Structural dead loads should be added to the resistance.
6. The calculated geotechnical resistance plus the structural dead loads must be greater than the factored ad-freezing forces.
7. Measures such as flat lying rigid polystyrene insulation could be considered to reduce frost penetration depths and thereby ad-freezing and uplift forces.

5.7 Negative Skin Friction

The effects of negative skin friction will need to be assessed if the site is raised or existing fill soils are replaced with new compacted fills. New fill could result in consolidation settlement of the underlying peat and clay soils and development of negative skin friction along pile shafts causing dragload on the piles. Dragload may result in excessive forces within the piles. TREK should be contacted to evaluate the potential of effects of negative skin friction once the site grades are finalized.

5.8 Pile Caps and Grade Beams

A minimum void of 150 mm should be provided underneath all grade beams and pile caps 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 selected such that they can deform a minimum of 150 mm without transferring intolerable stresses to the structure. Excavations for pile caps and grade beams should be backfilled with non-frost susceptible granular fill compacted to a minimum of 98% of the Standard Proctor Maximum Dry Density (SPMDD).

5.9 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 and seasonal movements. Further, all concrete should be designed in accordance with CSA A23.1-14 (Concrete Materials and Methods of Construction). Sulphate testing for water soluble sulphate content to assess the degree of exposure for concrete subjected to sulphate attack was not completed, however based on past experience in the area, and previous investigations at this site, sulphate resistant concrete is not required.

5.10 Floor Slabs

5.10.1 Structural Slabs

The peat will result in poor performance of grade supported floor slabs. Structural floor slabs are therefore recommended for the new clubhouse to allow for volumetric changes in the underlying sub-grade soils. The void can consist of a compressible layer (*e.g.* void form) to permit sub-grade soil movements without engaging the floor slab, or alternatively, a crawl space. Void forms should be selected such that they can deform a minimum of 150 mm with minimal transfer of stresses to the structure. A vapour barrier should be placed between the floor slab and the void form (if present).

5.11 Methane Gas Mitigation

Urban development areas within proximity of swamplands are known to contain methane gas from decomposition of organic material (e.g. peat). Methane gas is combustive and asphyxiating at high concentrations and poses a threat to the safety of commercial and residential building occupants. Although it is a relatively low risk scenario for this site compared to developments over landfills, it is something to be considered. In this regard, a methane mitigation system may be required to eliminate toxic gases. The following are options to help mitigate methane gases.

1. Place coarse granular fill over the peat to help dissipate methane vapours around the slab. The granular fill should extend beyond the footprint of the building. The thicker and coarser the granular layer is, the more effective it will be at dissipating the vapours. There is risk however that the granular fill will not be sufficient to effectively dissipate vapours which could lead further mitigation after construction of the clubhouse which would likely be very costly.
2. Install a PVC membrane directly below the floor slab or below new granular fill. An experienced supplier/contractor should be consulted for design of a liner. Care must be taken when placing fill over the membrane to protect against damage to the liner. Utility trenches (e.g. water, sewer, electrical, fibre, etc.) and connections into the slab would need to be properly sealed with the membrane. This approach is probably the most appropriate and cost-effective solution given the relatively low risk site conditions.
3. Install a passive or active ventilation system consisting of perforated pipes installed in the granular fill below the slab to collect and ventilate the methane vapours. Methane monitors should be installed in the building to evaluate the effectiveness of the ventilation system to determine if additional action is required. An environmental engineer should be consulted to develop an appropriate ventilation system for the site.

6.0 New Outdoor Ice Rink

TREK understands that a grade-supported concrete slab is preferred for the proposed outdoor ice rink. A grade-supported slab will be subject to settlement (total and differential) due to consolidation of the underlying peat and clay soils. Although difficult to predict, these settlements could be in the order of 500 mm. Movements of this magnitude will result in poor performance and damage to the slab which we assume is unacceptable. A grade-supported slab will also be subject to seasonal movements associated with freeze/thaw cycles of the frost susceptible soils underlying the slab. In this regard, we recommend the following to mitigate slab movements:

- install a structurally supported slab,
- remove all peat and replace with compacted granular fill, or
- leave the existing fill in place, preload the peat and clay in a staged construction approach. Complete the slab construction once settlement monitoring indicates it is acceptable to do so (could require more than a year of settlement)

More details of each option are provided below.

A more cost-effective approach may consist of eliminating the concrete slab and installing a grade beam supported by helical piles as described in the preceding section of this report. In this case, the rink boards would be supported by the grade beam and piles and the interior portion of the rink would consist of a granular pad. Seasonal maintenance would be required however to maintain a level rink surface. The granular pad should consist of at least 300 mm of addition granular fill and consist of Ontario Provincial Standards Specifications (OPSS) Granular A or B materials.

6.1 Structurally Supported Slabs

Foundations for a structurally supported slab should consist of helical piles bearing on compact sand as described in the preceding section of this report. A minimum void of 150 mm beneath structural floor slabs is recommended to allow for volumetric changes in the underlying sub-grade soils. The void can consist of a compressible layer (*e.g.* void form) to permit sub-grade soil movements without engaging the floor slab. Void forms should be selected such that they can deform a minimum of 150 mm with minimal transfer of stress to the slab. A vapour barrier should be placed between the slab and the void form (if present).

6.2 Peat Removal

Complete removal of peat is expected to require excavation of up to 3 m of soils. Site grades can be restored using compacted granular fill (OPSS Granular A or B) placed in maximum lifts of 150 mm and compacted to 100% of the SPMDD. Even with this level of compaction the granular fill can still be expected to settle 0.5% to 1% of the fill thickness. Long-term consolidation settlement of the very soft clay due to the added weight of the granular fill (compared to existing peat soils) should also be expected and could be in the order of 50 to 100 mm.

6.3 Preloading and Staged Construction

A grade-supported slab over the peat soils is an alternative if preloading with settlement monitoring is completed. The purpose of this approach would be to consolidate the underlying peat and very soft clay soils prior to slab construction to reduce the risk of post-construction settlement of the slab. In this case, settlement monitoring equipment would be installed within the peat and clay and 1 to 2 m of granular fill (OPSS Granular A or B) placed and compacted over the entire footprint of the ice rink. Settlement would be monitored over a period of 1 to 2 years and once the settlement has stopped, the granular fill would be stripped to the design sub-grade and the concrete slab constructed above. A high strength non-woven geotextile or geogrid should be placed on top of existing fill soils prior to placement of new granular fill to help to mitigate impacts from differential settlement. Granular fill should be placed in lifts no greater than 150 mm and compacted to 100% of the SPMDD. Some differential settlement and maintenance of the granular fill should be expected while consolidation of peat and clay occurs. To minimize seasonal movements associated with freeze/thaw of the sub-grade soils, insulation should be installed to provide frost protection to an equivalent depth of 2.5 m below grade.

TREK should be contacted to develop a preloading and monitoring program which is not included in our current scope of work.

7.0 Pavements

This section provides recommendations for asphalt pavements. Recommended pavement sections for parking areas are provided in Table 4. If the granular fill materials provided in Table 4 are not available, alternative materials capable of providing equivalent performance may be proposed for approval by TREK.

Table 4: Recommended Asphalt Pavement Sections

Material	Layer Thickness		Compaction Requirements
	Car Parking Areas	Heavy Vehicular Loads	Compaction Requirements / Comments
Asphalt	100 mm	100 mm	Mix design and compaction requirements by others
OPSS Granular A	75 mm	100 mm	100% of the SPMDD
OPSS Granular B	250 mm	350 mm	98% of the SPMDD
Non-Woven Geotextile (Titan Environmental TE-8 or equivalent)	Required	Required	Install as per manufacturer's recommendations

Additional Pavement Recommendations:

1. For best performance, all organics, fill, silt, and any other deleterious material should be completely removed such that the sub-grade consists of native clay. It is anticipated however that this will require removal of up to 3.0 m of fill and organic materials. Assuming that this will not be practical from a cost or constructability perspective and provided the potential for significant settlement due to compression of peat soils is considered acceptable, the sub-grade may consist of existing granular fill materials. Removal of existing fill is not recommended in this case, however, it should be scarified, moisture conditioned, and recompacted to 98% of the SPMDD.
2. Excavations for pavement sub-grade 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 minimize disturbance to the sub-grade at all times.
3. After excavation, the sub-grade should be inspected by TREK personnel. Silt and soft areas identified should be repaired as per directions provided by TREK. This will likely consist of excavating an additional 150 to 300 mm and backfilling with a 50 mm down granular fill (OPSS Granular B) 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, inundation with water or disturbance. 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. A non-woven geotextile should be placed in accordance with the manufacturer's recommendations on the prepared subgrade prior to placement of granular fill. Titan Environmental TE-8 or equivalent would be appropriate for use.
6. The granular sub-base and base materials should be placed in lifts not exceeding 150 mm thick and compacted to as per the recommendations in Table 4.

7. The granular base course materials should consist of a well graded, durable, crushed rock, in accordance with Ontario Provincial Standards Specifications.

8.0 Site Drainage

Positive site drainage around the perimeter of the structure should be provided at a gradient of at least 2%. A minimum gradient of about 2% should be used for both landscaped and paved areas and maintained throughout the life of the structures.

9.0 Temporary Excavations

Excavations must be carried out in compliance with the Occupational Health and Safety Act Ontario regulation 213/91 *Construction Projects* and other applicable safety regulations or codes. Any open-cut excavation greater than 3 m deep must be designed and sealed by a professional engineer and reviewed by the geotechnical engineer of record (TREK). If space is limited or the stability of adjacent structures may be endangered by an excavation, a shoring system may be required to prevent damage to, or movement of, any part of adjacent structures, and the creation of a hazard to workers and the public.

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

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

10.0 Seismic Site Classification

The site classification for seismic site response was determined based on Section *4.1.8 Earthquake Load and Effects* of the NBCC (2015). Site Class E may be applied to this site.

11.0 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:

a) continuous basis during:
    i. the construction of all deep foundation units with all pertinent information recorded for each *foundation unit*,
    ii. during the installation and removal of retaining structures and related backfilling operations,
    iii. during the placement of engineered fills that are to be used to support the *foundation units*, and
b) as-required, unless otherwise directed by the *authority having jurisdiction*,
    i. in the construction of all *shallow foundation units,* and
    ii. in excavating, dewatering and other related works

In accordance with Engineers and Geoscientists of Manitoba, a Professional Engineer or delegated staff responsible to them must perform site reviews for the work presented in the documents they've sealed.

For conformance with the NBCC and EGM requirements, TREK should be retained on a full-time basis to observe and document the installation of all caisson foundations, shoring or engineered fills supporting the structure, and on an as-required basis for other components such as sub-grade inspections and compaction testing. TREK is familiar with the geotechnical conditions present and the underlying design assumptions of our foundation recommendations. TREK is therefore solely qualified to evaluate any design modifications deemed to be necessary should altered subsurface conditions be encountered.

12.0 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.

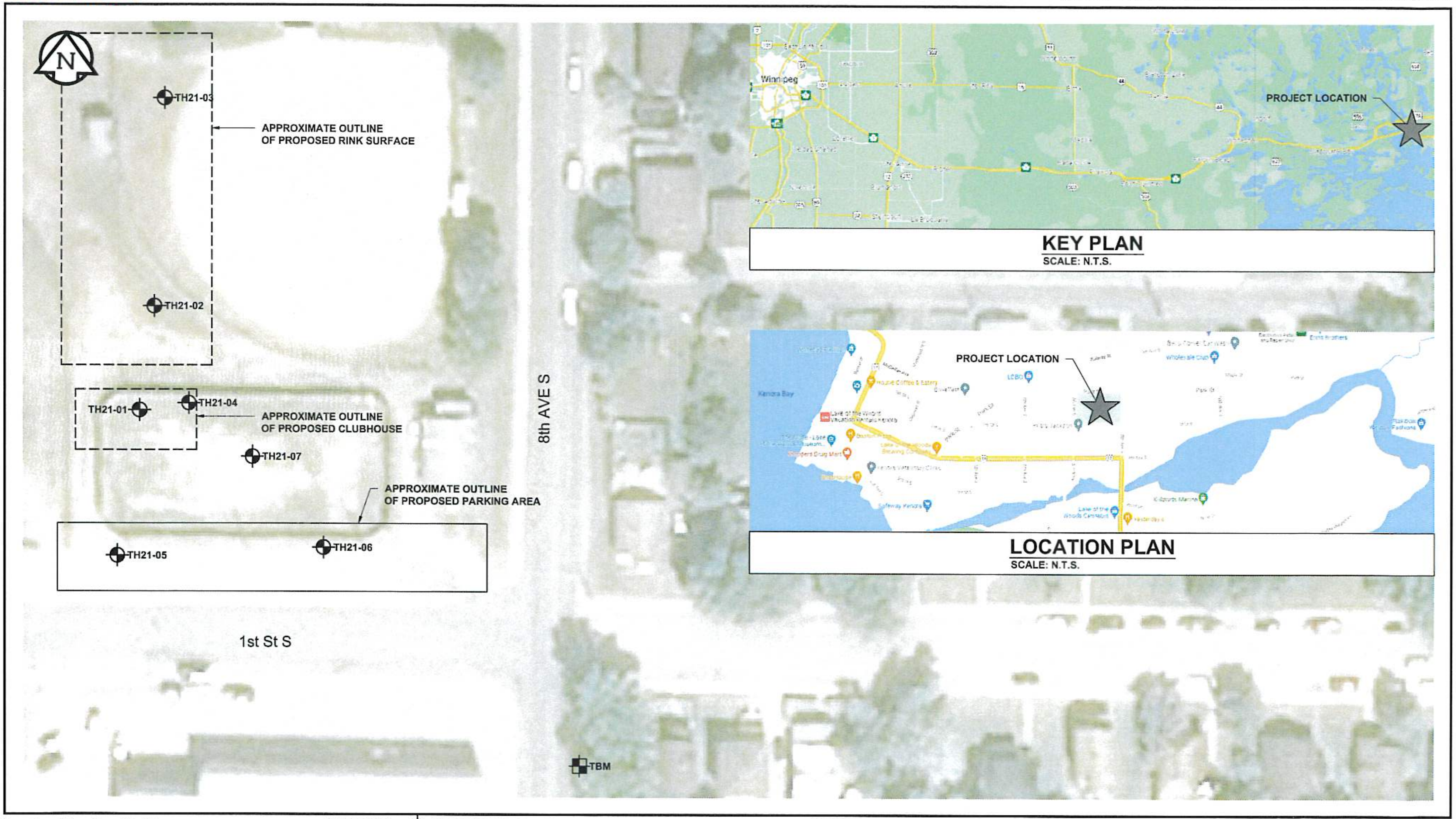
This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of Solid Construction Inc. (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third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

Figure

TREK GEOTECHNICAL

0814 001 00
Solid Construction Inc.
Kenora Central Community Club Sub-surface Investigation

Z:\Projects\0814 Solid Construction\0814 001-00 Central Community Club\3 Survey and Dwg\3.4 CAD\3 4 3 Working Folder\0814-001-00 Kenora Central CC Fig 01 CT.dwg, 2021-11-04 9:37:23 AM



0 5 10 15 20 25 m
SCALE = 1 : 600 (279 mm x 432 mm)

LEGEND:
- ⊕ TEST HOLE (TREK, 2021)
- ▚ TEMPORARY BENCHMARK

NOTES:
1. AERIAL IMAGERY FROM BING MAPS, (2021).
2. TEMPORARY BENCHMARK IS LOCATED AT THE TOP OF EXISTING FIRE HYDRANT AT THE SOUTHEAST CORNER OF 1st STREET SOUTH AND 8th AVENUE SOUTH.

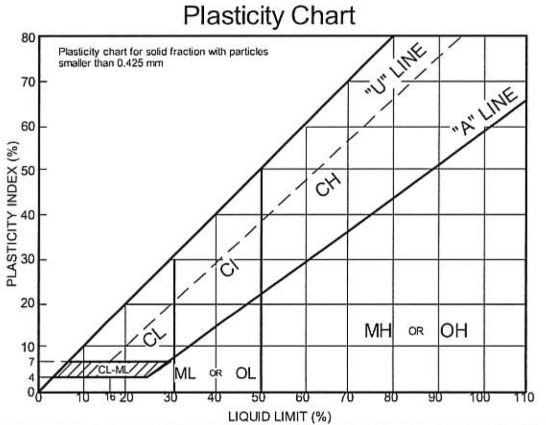
Figure 01
Test Hole Location Plan

Test Hole Logs

EXPLANATION OF FIELD AND LABORATORY TESTING

GENERAL NOTES

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

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

Material		Particle Size	
		mm	ASTM Sieve sizes
Sand	Coarse	2.00 to 4.75	#10 to #4
	Medium	0.425 to 2.00	#40 to #10
	Fine	0.075 to 0.425	#200 to #40
Silt or Clay		< 0.075	< #200

Material		Particle Size	
		mm	ASTM Sieve Sizes
Boulders		> 300	> 12 in.
Cobbles		75 to 300	3 in. to 12 in.
Gravel	Coarse	19 to 75	3/4 in. to 3 in.
	Fine	4.75 to 19	#4 to 3/4 in.

* 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

TREK GEOTECHNICAL

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

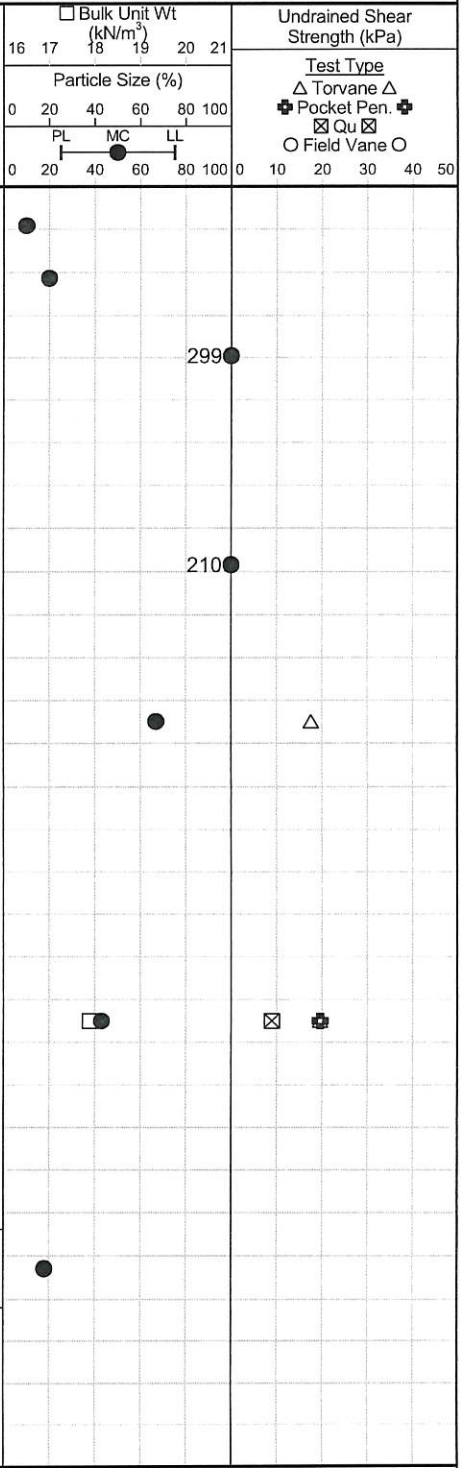
Sub-Surface Log

Client: Solid Construction Inc.
Project Name: Central Community Club, Kenora, ON
Contractor: Paddock Drilling Ltd.
Method: 170 mm Hollow Stem Auger, Acker MP5-T Track Mount
Project Number: 0814 001 00
Location: UTM 14N: 5513737.084 N, 393521.7324 E
Ground Elevation: 99.42 m
Date Drilled: September 30, 2021

Sample Type: ◣ Grab (G) ▮ Shelby Tube (T) ☒ Split Spoon (SS) / SPT ⧓ Split Barrel (SB) / LPT ▯ Core (C)

Particle Size Legend: ▦ Fines ▨ Clay ▥ Silt ⁘ Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
			SAND (FILL) - trace silt, trace gravel (<10 mm diam.) - brown, moist, loose - poorly graded, fine grained	◣	G01	
99.0	0.5		SILT (FILL) - trace to some fine sand, trace to some clay - grey, moist, soft - low plasicity	◣	G02	
98.5	1.0		PEAT - amorphous, some rootlets - brown - moist - H5-H6 Von Post classification	◣	G03	
	1.5					
	2.0		- grey to dark brown, moist to wet below 2.1 m	◣	G04	
	2.5					
96.5	3.0		CLAY - silty - grey - moist, soft - high plasicity	◣	G05	
	3.5					
	4.0					
	4.5		- very soft below 4.6 m	▮	T06	
	5.0					
93.9	5.5		SAND - trace fine gravel <25 mm diam.) - brown - wet, compact - angular - poorly graded, coarse grained			
	6.0			☒	S07	21
	6.5					
	7.0					

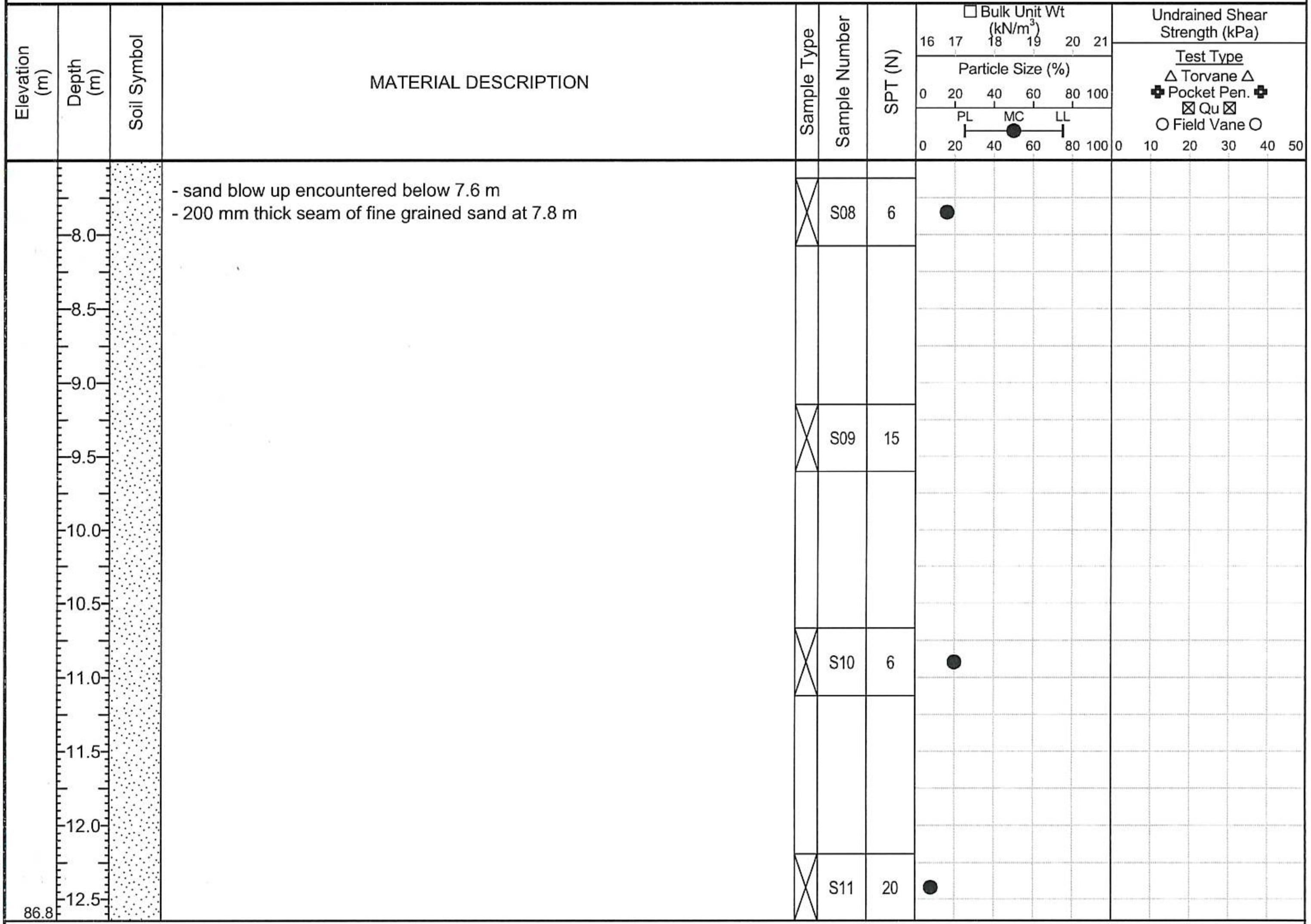


Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

Sub-Surface Log

Test Hole TH21-01

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
			- sand blow up encountered below 7.6 m - 200 mm thick seam of fine grained sand at 7.8 m		S08	6
					S09	15
					S10	6
86.8					S11	20



END OF TEST HOLE AT 12.7 m DEPTH IN SAND

Notes:

1. Seepage and sloughing observed below 3.0 m depth.
2. Test hole drilled with 125 mm diam. solid stem auger to 4.6 m depth.
3. Switched to hollow stem below 4.6 m depth due to seepage and sloughing conditions.
4. Water level not measured after completion of drilling due to drilling method used.
5. Test hole open to 3.0 m depth immediately after completion.
6. Test hole backfilled to surface with cuttings and bentonite chips.
7. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH21-02

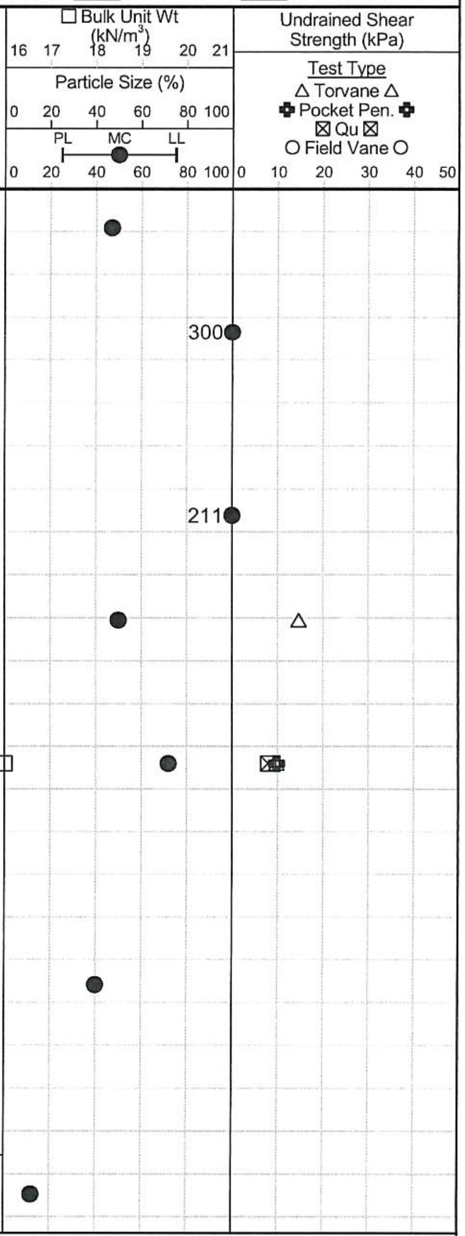
1 of 1

Client: Solid Construction Inc.
Project Name: Central Community Club, Kenora, ON
Contractor: Paddock Drilling Ltd.
Method: 125 mm Solid Stem Auger, Acker MP5-T Track Mount
Project Number: 0814 001 00
Location: UTM 14N: 5513754.419 N, 393524.1316 E
Ground Elevation: 99.39 m
Date Drilled: September 30, 2021

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

Particle Size Legend: ▦ Fines ▨ Clay ▥ Silt ⁙ Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
99.2		ORGANIC CLAY (Topsoil) - some organics, dark brown to black, moist, intermediate plasticity	Grab (G)	G12	
98.9		CLAY (FILL) - silty, trace fine sand, trace fine gravel (<15 mm diam.), grey, moist, firm, high plasicity			
	0.5	PEAT - amorphous, some rootlets - brown - moist - H5-H6 Von Post classification	Grab (G)	G13	
	1.0				
	1.5				
		- grey to dark brown, moist to wet below 1.8 m	Grab (G)	G14	
	2.0				
97.0 ▼					
	2.5	CLAY - silty - grey - moist, soft - high plasicity	Grab (G)	G15	
	3.0	- very soft below 3.0 m			
			Shelby Tube (T)	T16	15.5
	3.5				
	4.0				
	4.5		Grab (G)	G17	
	5.0				
93.9	5.5	SAND - trace fine gravel <25 mm diam.) - brown - wet, compact - angular, poorly graded, coarse grained			
93.3	6.0		Split Spoon (SS) / SPT	S18	9



END OF TEST HOLE AT 6.1 m DEPTH IN SAND

Notes:

1. Seepage and sloughing observed below 2.9 m depth.
2. Water level measured at 2.4 m depth immediately after completion of drilling.
3. Test hole open to 2.9 m depth immediately after completion of drilling.
4. Test hole backfilled to surface with cuttings and bentonite chips.
5. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH21-03

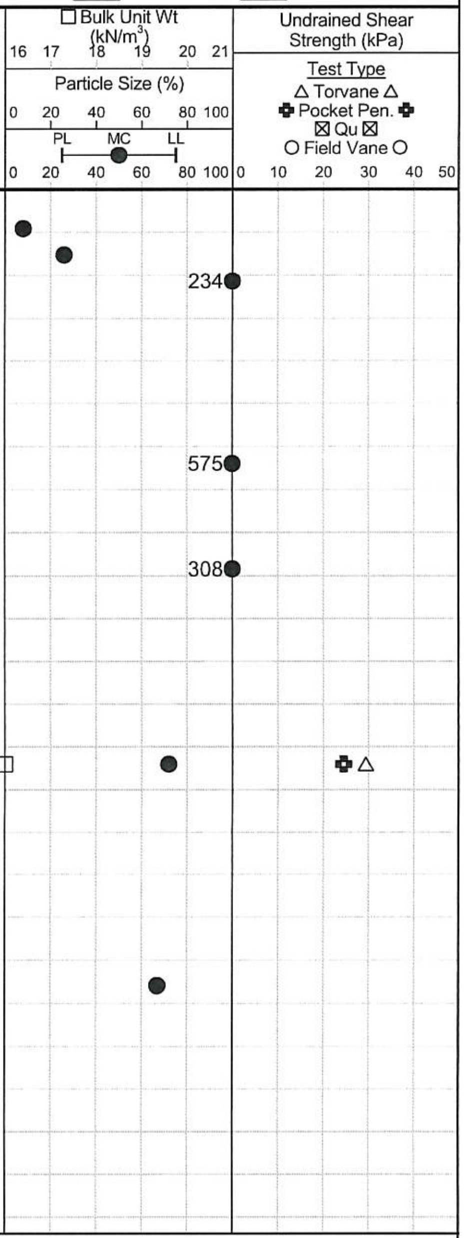
1 of 1

Client: Solid Construction Inc.
Project Number: 0814 001 00
Project Name: Central Community Club, Kenora, ON
Location: UTM 14N: 5513788.953 N, 393525.8167 E
Contractor: Paddock Drilling Ltd.
Ground Elevation: 99.45 m
Method: 125 mm Solid Stem Auger, Acker MP5-T Track Mount
Date Drilled: September 30, 2021

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

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt ⁙ Sand ● Gravel ◌ Cobbles ■ Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)	Plotted value
		SAND (FILL) - trace silt, trace gravel (<10 mm diam.) - brown, moist, loose, poorly graded, fine grained	G	G19		
99.1		SILT (FILL) - trace to some fine sand, trace to some clay, grey, moist, soft, low plasicity	G	G20		
99.0	0.5	PEAT - amorphous, trace rootlets - dark brown - moist - H5-H6 Von Post classification	G	G21		234
	1.5	- fibrous, orange to brown, moist, H2-H3 Von Post classification below 1.5 m	G	G22		575
	2.0	- amorphous, grey to dark brown, moist to wet, H5-H6 Von Post classification below 1.8 m	G	G23		308
96.4	3.0	CLAY - silty - grey - moist, firm - high plasicity	T	S24	15.4 □	
	4.6	- very soft below 4.6 m	G	G25		
93.4	6.0					



END OF TEST HOLE AT 6.1 m DEPTH IN CLAY

Notes:

1. Seepage and sloughing observed below 3.0 m depth.
2. Water level measured at 2.4 m depth immediately after completion of drilling.
3. Test hole open to 3.0 m depth immediately after completion of drilling.
4. Test hole backfilled to surface with cuttings and bentonite chips.
5. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

TREK GEOTECHNICAL

Sub-Surface Log

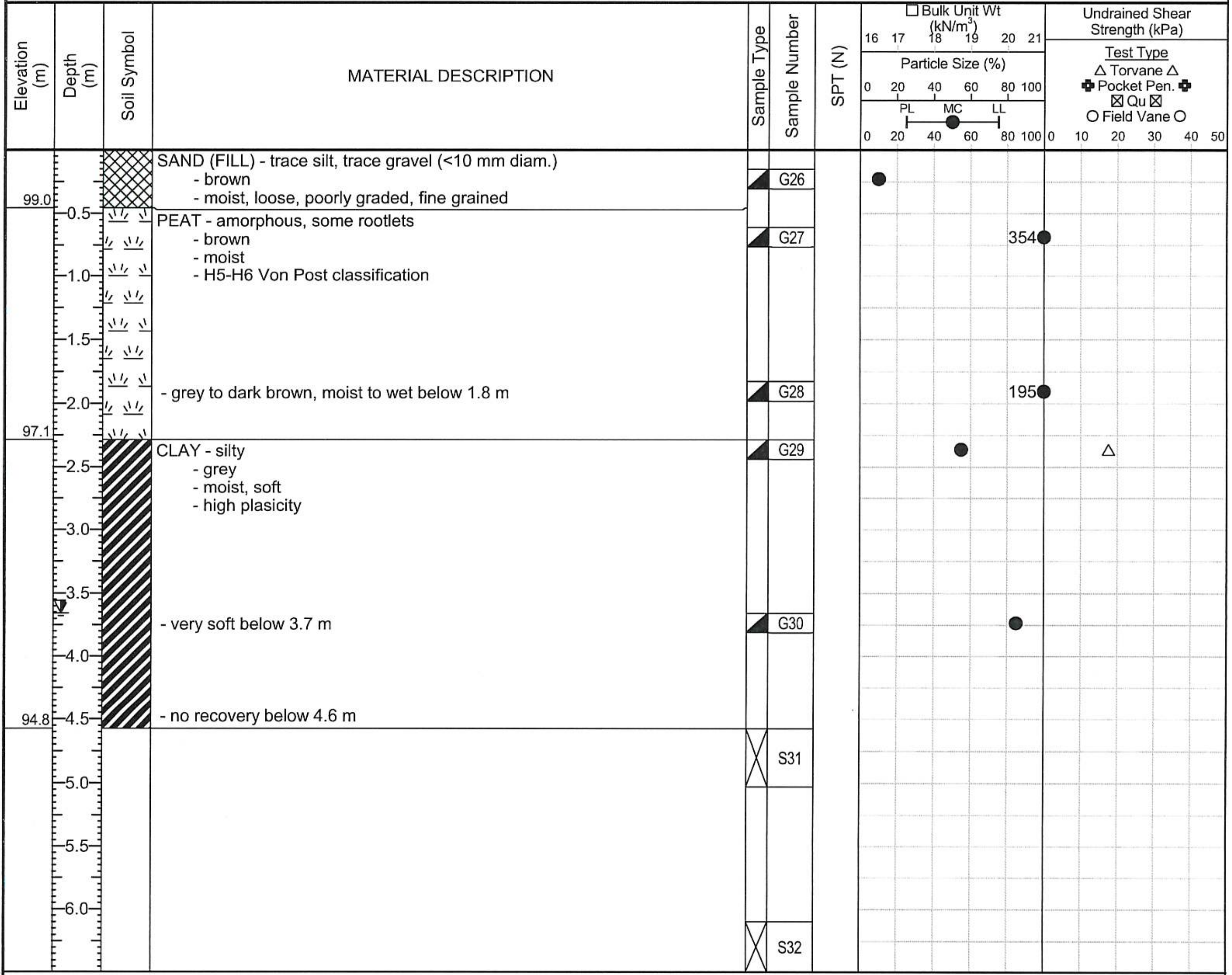
Test Hole TH21-04

1 of 1

Client:	Solid Construction Inc.	**Project Number:**	0814 001 00
Project Name:	Central Community Club, Kenora, ON	**Location:**	UTM 14N: 5513738.258 N, 393529.8225 E
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	99.41 m
Method:	125 mm Solid Stem Auger, Acker MP5-T Track Mount	**Date Drilled:**	September 30, 2021

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

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt Sand Gravel Cobbles Boulders



SAND (FILL) - trace silt, trace gravel (<10 mm diam.)
- brown
- moist, loose, poorly graded, fine grained

PEAT - amorphous, some rootlets
- brown
- moist
- H5-H6 Von Post classification

- grey to dark brown, moist to wet below 1.8 m

CLAY - silty
- grey
- moist, soft
- high plasicity

- very soft below 3.7 m

- no recovery below 4.6 m

END OF TEST HOLE AT 6.5 m DEPTH

Notes:

1. Test hole terminated due to augers going out of plumb below 4.6 m depth.
2. Seepage and sloughing observed below 3.7 m depth.
3. Water level measured at 3.7 m depth immediately after completion of drilling.
4. Test hole open to 4.9 m depth immediately after completion of drilling.
5. Test hole backfilled to surface with cuttings and bentonite chips.
6. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

GEOTECHNICAL

Sub-Surface Log

Test Hole TH21-05
1 of 1

Client:	Solid Construction Inc.	**Project Number:**	0814 001 00
Project Name:	Central Community Club, Kenora, ON	**Location:**	UTM 14N: 5513712.909 N, 393518.086 E
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	99.38 m
Method:	125 mm Solid Stem Auger, Acker MP5-T Track Mount	**Date Drilled:**	September 30, 2021

Sample Type: ◩ Grab (G) ▮ Shelby Tube (T) ☒ Split Spoon (SS) / SPT ⧓ Split Barrel (SB) / LPT ▯ Core (C)

Particle Size Legend: ▦ Fines ▨ Clay ▥ Silt ⛆ Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)	☐ Bulk Unit Wt (kN/m^3) 16 17 18 19 20 21 Particle Size (%) 0 20 40 60 80 100 PL MC LL 0 20 40 60 80 100	Undrained Shear Strength (kPa) Test Type △ Torvane △ ✚ Pocket Pen. ✚ ☒ Qu ☒ ○ Field Vane ○ 0 10 20 30 40 50
		▩	SAND (FILL) - trace clay, trace silt, trace gravel (<15 mm diam.) - black, moist, compact - poorly graded, fine grained	◩	G32		●	
98.9	0.5		PEAT - fibrous, some rootlets - orange to brown - moist - H2-H3 Von Post classification	◩	G33		230●	
	1.0							
97.9	1.5							

END OF TEST HOLE AT 1.5 m DEPTH IN PEAT
Notes:
1. Seepage and sloughing were not observed during drilling.
2. Test hole dry upon completion of drilling.
3. Test hole open to 1.5 m depth immediately after completion of drilling.
4. Test hole backfilled to surface with cuttings and bentonite chips.
5. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH21-06

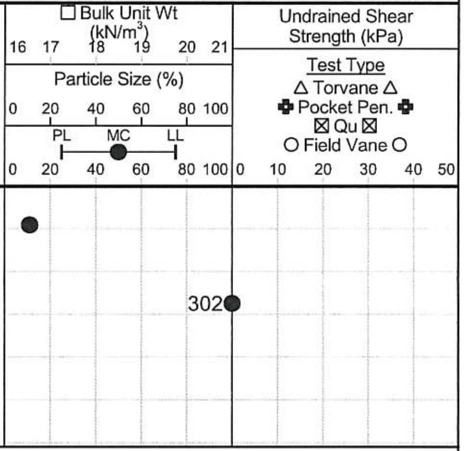
1 of 1

Client: Solid Construction Inc.
Project Number: 0814 001 00
Project Name: Central Community Club, Kenora, ON
Location: UTM 14N: 5513714.279 N, 393552.2624 E
Contractor: Paddock Drilling Ltd.
Ground Elevation: 99.36 m
Method: 125 mm Solid Stem Auger, Acker MP5-T Track Mount
Date Drilled: September 30, 2021

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

Particle Size Legend: ▩ Fines ▨ Clay ▥ Silt ⛆ Sand ● Gravel ◌ Cobbles ■ Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)
99.1		⊠	SAND (FILL) - trace clay, trace silt, trace gravel (<15 mm diam.) - black, moist, compact, poorly graded, fine grained	◩	G34	
97.8	0.5 1.0 1.5	ᐡᐡ	PEAT - fibrous, some rootlets - orange to brown - moist - H2-H3 Von Post classification	◩	G35	



END OF TEST HOLE AT 1.5 m DEPTH IN PEAT
Notes:
1. Seepage and sloughing were not observed during drilling.
2. Test hole dry upon completion of drilling.
3. Test hole open to 1.5 m depth immediately after completion of drilling.
4. Test hole backfilled to surface with cuttings and bentonite chips.
5. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko
Reviewed By: Kent Bannister
Project Engineer: Ryan Belbas

Sub-Surface Log

Test Hole TH21-07

1 of 1

Client: Solid Construction Inc.

Project Number: 0814 001 00

Project Name: Central Community Club, Kenora, ON

Location: UTM 14N: 5513729.369 N, 393540.4555 E

Contractor: Paddock Drilling Ltd.

Ground Elevation: 99.40 m

Method: 125 mm Solid Stem Auger, Acker MP5-T Track Mount

Date Drilled: September 30, 2021

Sample Type: ◩ Grab (G) ▮ Shelby Tube (T) ☒ Split Spoon (SS) / SPT ⧓ Split Barrel (SB) / LPT ◫ Core (C)

Particle Size Legend: ▩ Fines ▨ Clay ▥ Silt ⛆ Sand ● Gravel ◌ Cobbles ■ Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	SPT (N)	☐ Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 Particle Size (%) 0 20 40 60 80 100 PL MC LL 0 20 40 60 80 100	Undrained Shear Strength (kPa) Test Type △ Torvane △ ✚ Pocket Pen. ✚ ☒ Qu ☒ ○ Field Vane ○ 0 10 20 30 40 50
			Test hole drilled in attempt to verify presence of bedrock or boulders. Augers in test hole began sloping below 4.9 m.					
	0.5							
	1.0							
	1.5							
	2.0							
	2.5							
	3.0							
	3.5							
	4.0							
	4.5							
94.5								

END OF TEST HOLE AT 6.5 m DEPTH

Notes:

1. Test hole terminated due to augers going out of plumb below 4.9 m depth.
2. Seepage and sloughing conditions not measured after completion of drilling.
3. Test hole backfilled to surface with cuttings and bentonite chips.
4. Test hole surveyed relative to TBM located at the top of existing fire hydrant at the southeast corner of 1st St South and 8th Ave South. An elevation of 100.0 m was assigned to the TBM.

Logged By: Matt Klymochko **Reviewed By:** Kent Bannister **Project Engineer:** Ryan Belbas

Appendix A

Laboratory Testing

TREK GEOTECHNICAL
www.trekgeotechnical.ca
1712 St. James Street
Winnipeg, MB R3H 0L3
Tel: 204.975.9433 Fax: 204.975.9435

Moisture Content Report
ASTM D2216-10

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON

Sample Date	05-Oct-21
Test Date	12-Oct-21
Technician	JN

Test Hole	TH21-01	TH21-01	TH21-01	TH21-01	TH21-01	TH21-01
Depth (m)	0.2 - 0.3	0.5 - 0.6	0.9 - 1.1	2.1 - 2.3	3.0 - 3.2	6.1 - 6.6
Sample #	G01	G02	G03	G04	G05	S07
Tare ID	AC14	N28	W48	PB28	K18	N85
Mass of tare	6.9	8.3	8.4	8.6	8.5	8.4
Mass wet + tare	252.7	220.2	161.7	251.4	224.7	267.7
Mass dry + tare	230.6	185.8	46.8	86.8	138.2	227.4
Mass water	22.1	34.4	114.9	164.6	86.5	40.3
Mass dry soil	223.7	177.5	38.4	78.2	129.7	219.0
Moisture %	9.9%	19.4%	299.2%	210.5%	66.7%	18.4%

Test Hole	TH21-01	TH21-01	TH21-01	TH21-02	TH21-02	TH21-02
Depth (m)	7.6 - 8.1	10.7 - 11.1	12.2 - 12.6	0.2 - 0.3	0.8 - 0.9	1.8 - 2.0
Sample #	S08	S10	S11	G12	G13	G14
Tare ID	D27	W85	AB75	AC05	W47	W44
Mass of tare	8.4	8.6	6.9	6.8	8.5	8.6
Mass wet + tare	304.7	238.9	253.4	331.9	159.6	189.0
Mass dry + tare	263.4	200.6	234.4	228.6	46.2	66.6
Mass water	41.3	38.3	19.0	103.3	113.4	122.4
Mass dry soil	255.0	192.0	227.5	221.8	37.7	58.0
Moisture %	16.2%	19.9%	8.4%	46.6%	300.8%	211.0%

Test Hole	TH21-02	TH21-02	TH21-02	TH21-03	TH21-03	TH21-03
Depth (m)	2.4 - 2.6	4.6 - 4.7	5.6 - 6.1	0.2 - 0.3	0.3 - 0.5	0.5 - 0.6
Sample #	G15	G17	S18	G19	G20	G21
Tare ID	E47	E16	W01	Z05	Z07	W34
Mass of tare	8.7	8.5	8.4	8.4	8.8	8.7
Mass wet + tare	222.1	380.9	355.0	213.5	246.3	200.1
Mass dry + tare	150.6	274.9	319.2	198.8	196.8	66.0
Mass water	71.5	106.0	35.8	14.7	49.5	134.1
Mass dry soil	141.9	266.4	310.8	190.4	188.0	57.3
Moisture %	50.4%	39.8%	11.5%	7.7%	26.3%	234.0%

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Moisture Content Report
ASTM D2216-10

Project No. 0814-001-00
Client Solid Construction Inc.
Project Central Community Club, Kenora, ON

Sample Date 05-Oct-21
Test Date 12-Oct-21
Technician JN

Test Hole	TH21-03	TH21-03	TH21-03	TH21-04	TH21-04	TH21-04
Depth (m)	1.5 - 1.7	2.1 - 2.3	4.6 - 4.7	0.2 - 0.3	0.6 - 0.8	1.8 - 2.0
Sample #	G22	G23	G25	G26	G27	G28
Tare ID	H54	F66	H55	W59	D49	W99
Mass of tare	8.5	8.6	8.5	8.7	8.5	8.5
Mass wet + tare	201.0	221.4	398.8	220.5	123.3	246.3
Mass dry + tare	37.0	60.8	241.6	200.8	33.8	89.2
Mass water	164.0	160.6	157.2	19.7	89.5	157.1
Mass dry soil	28.5	52.2	233.1	192.1	25.3	80.7
Moisture %	575.4%	307.7%	67.4%	10.3%	353.8%	194.7%

Test Hole	TH21-04	TH21-04	TH21-05	TH21-05	TH21-06	TH21-06
Depth (m)	2.3 - 2.4	3.7 - 3.8	0.2 - 0.3	0.6 - 0.8	0.2 - 0.3	0.6 - 0.8
Sample #	G29	G30	G32	G33	G34	G35
Tare ID	A103	D56	Z12	E80	A34	W15
Mass of tare	8.6	8.8	8.5	8.6	8.3	8.5
Mass wet + tare	267.8	249.7	225.4	182.9	272.5	148.0
Mass dry + tare	175.8	139.4	183.2	61.4	247.4	43.2
Mass water	92.0	110.3	42.2	121.5	25.1	104.8
Mass dry soil	167.2	130.6	174.7	52.8	239.1	34.7
Moisture %	55.0%	84.5%	24.2%	230.1%	10.5%	302.0%

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Shelby Tube Visual

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON

Test Hole	TH21-01
Sample #	T06
Depth (m)	4.6 - 5.2
Sample Date	05-Oct-21
Test Date	12-Oct-21
Technician	JN

Tube Extraction

Recovery (mm) 580

Bottom - 5.2 m	5.14 m	4.94 m	4.89 m	4.76 m	**Top - 4.6 m**
Toss	Qu Bulk	Moisture Content PP/TV Visual	Keep	Toss	
40 mm	200 mm	50 mm	130 mm	160 mm	

Visual Classification

Material	CLAY
Composition	silty

trace silt inclusions (<10 mm diam.)
trace gravel (<10 mm diam.)

Color	dark brown
Moisture	moist
Consistency	very soft to soft
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.20
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	19.6

Pocket Penetrometer

Reading	**1**	0.40
	2	0.40
	3	0.50
	Average	0.43
Undrained Shear Strength (kPa)		21.2

Moisture Content

Tare ID	W13
Mass tare (g)	8.4
Mass wet + tare (g)	411.3
Mass dry + tare (g)	291.2
Moisture %	42.5%

Unit Weight

Bulk Weight (g)		1008.4
Length (mm)	**1**	145.87
	2	146.01
	3	145.85
	4	146.20
Average Length (m)		0.146
Diam. (mm)	**1**	68.49
	2	68.72
	3	70.50
	4	70.29
Average Diameter (m)		0.070
Volume (m^3)		5.54E-04
Bulk Unit Weight (kN/m^3)		17.9
Bulk Unit Weight (pcf)		113.7
Dry Unit Weight (kN/m^3)		12.5
Dry Unit Weight (pcf)		79.8

Unconfined Compressive Strength

ASTM D2166

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON

Test Hole	TH21-01
Sample #	T06
Depth (m)	4.6 - 5.2
Sample Date	21-Jun-21
Test Date	28-Jun-21
Technician	JN

Unconfined Strength

	kPa	ksf
Max q_u	18.7	0.4
Max S_u	9.3	0.2

Specimen Data

Description CLAY - silty, trace silt inclusions (<10 mm diam.), trace gravel (<10 mm diam.), dark brown, moist, very soft to soft, high plasticity

Length	146.0	(mm)	**Moisture %**	42%	
Diameter	69.5	(mm)	**Bulk Unit Wt.**	17.9	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	12.5	(kN/m^3)
Initial Area	0.00379	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

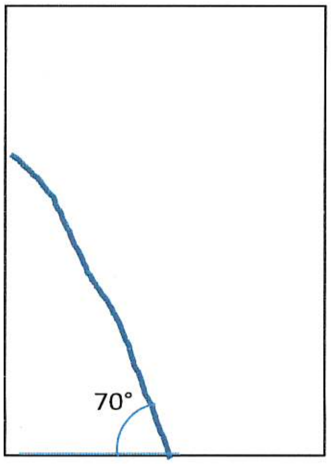
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.20	19.6	0.41
Vane Size		
m		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	0.40	19.6	0.41
	0.40	19.6	0.41
	0.50	24.5	0.51
Average	**0.43**	**21.3**	**0.44**

Failure Geometry

Sketch:



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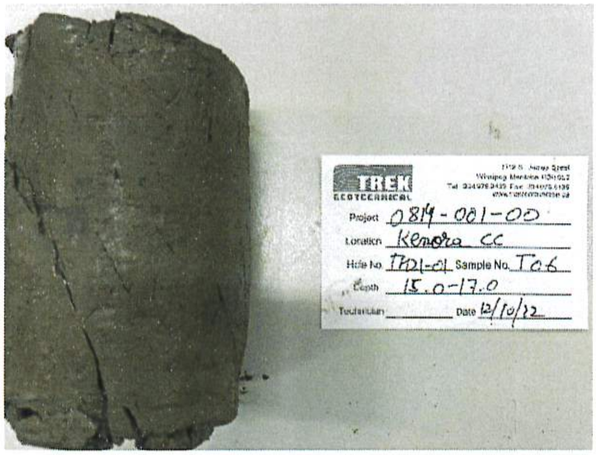
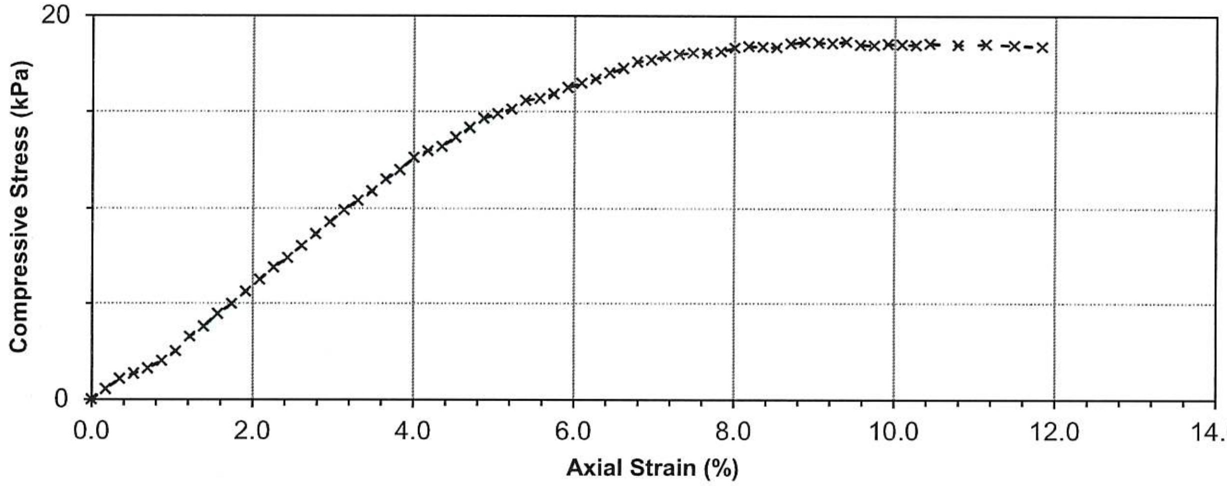

TREK GEOTECHNICAL
Project 0814-001-00
Location Kenora CC
Hole No. TH21-01 Sample No. T06
Depth 15.0-17.0
Technician Date 12/10/22

Unconfined Compressive Strength

ASTM D2166

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON

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	-0.11	0.0000	0.00	0.003794	0.0	0.00	0.00
10	-0.07	0.2540	0.17	0.003800	2.0	0.53	0.27
20	-0.03	0.5080	0.35	0.003807	4.0	1.06	0.53
30	-0.01	0.7620	0.52	0.003814	5.0	1.32	0.66
40	0.01	1.0160	0.70	0.003820	6.0	1.58	0.79
50	0.04	1.2700	0.87	0.003827	7.6	1.98	0.99
60	0.08	1.5240	1.04	0.003834	9.6	2.50	1.25
70	0.14	1.7780	1.22	0.003840	12.6	3.28	1.64
80	0.18	2.0320	1.39	0.003847	14.6	3.80	1.90
90	0.23	2.2860	1.57	0.003854	17.1	4.45	2.22
100	0.27	2.5400	1.74	0.003861	19.2	4.96	2.48
110	0.32	2.7940	1.91	0.003868	21.7	5.60	2.80
120	0.37	3.0480	2.09	0.003875	24.2	6.24	3.12
130	0.42	3.3020	2.26	0.003881	26.7	6.88	3.44
140	0.46	3.5560	2.44	0.003888	28.7	7.39	3.69
150	0.51	3.8100	2.61	0.003895	31.2	8.02	4.01
160	0.56	4.0640	2.78	0.003902	33.8	8.65	4.33
170	0.61	4.3180	2.96	0.003909	36.3	9.28	4.64
180	0.66	4.5720	3.13	0.003916	38.8	9.91	4.95
190	0.70	4.8260	3.31	0.003923	40.8	10.41	5.20
200	0.74	5.0800	3.48	0.003930	42.8	10.90	5.45
210	0.79	5.3340	3.65	0.003938	45.4	11.52	5.76
220	0.83	5.5880	3.83	0.003945	47.4	12.01	6.01
230	0.88	5.8420	4.00	0.003952	49.9	12.63	6.31

Unconfined Compressive Strength

ASTM D2166

Project No. 0814-001-00
Client Solid Construction Inc.
Project Central Community Club, Kenora, ON

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	0.91	6.0960	4.18	0.003959	51.4	12.99	6.49
250	0.93	6.3500	4.35	0.003966	52.4	13.22	6.61
260	0.97	6.6040	4.52	0.003973	54.4	13.70	6.85
270	1.01	6.8580	4.70	0.003981	56.5	14.18	7.09
280	1.05	7.1120	4.87	0.003988	58.5	14.66	7.33
290	1.07	7.3660	5.05	0.003995	59.5	14.89	7.44
300	1.09	7.6200	5.22	0.004003	60.5	15.11	7.56
310	1.13	7.8740	5.39	0.004010	62.5	15.59	7.79
320	1.14	8.1280	5.57	0.004017	63.0	15.68	7.84
330	1.16	8.3820	5.74	0.004025	64.0	15.90	7.95
340	1.19	8.6360	5.92	0.004032	65.5	16.25	8.13
350	1.21	8.8900	6.09	0.004040	66.5	16.47	8.23
360	1.23	9.1440	6.26	0.004047	67.5	16.69	8.34
370	1.26	9.3980	6.44	0.004055	69.1	17.03	8.52
380	1.28	9.6520	6.61	0.004062	70.1	17.25	8.62
390	1.31	9.9060	6.79	0.004070	71.6	17.59	8.79
400	1.32	10.1600	6.96	0.004077	72.1	17.68	8.84
410	1.34	10.4140	7.13	0.004085	73.1	17.89	8.95
420	1.35	10.6680	7.31	0.004093	73.6	17.98	8.99
430	1.36	10.9220	7.48	0.004100	74.1	18.07	9.03
440	1.36	11.1760	7.66	0.004108	74.1	18.04	9.02
450	1.37	11.4300	7.83	0.004116	74.6	18.12	9.06
460	1.39	11.6840	8.00	0.004124	75.6	18.33	9.17
470	1.40	11.9380	8.18	0.004132	76.1	18.42	9.21
480	1.40	12.1920	8.35	0.004139	76.1	18.39	9.19
490	1.40	12.4460	8.53	0.004147	76.1	18.35	9.18
500	1.42	12.7000	8.70	0.004155	77.1	18.56	9.28
510	1.43	12.9540	8.87	0.004163	77.6	18.64	9.32
520	1.43	13.2080	9.05	0.004171	77.6	18.61	9.30
530	1.43	13.4620	9.22	0.004179	77.6	18.57	9.29
540	1.44	13.7160	9.40	0.004187	78.1	18.66	9.33
550	1.43	13.9700	9.57	0.004195	77.6	18.50	9.25
560	1.43	14.2240	9.74	0.004203	77.6	18.47	9.23
570	1.44	14.4780	9.92	0.004211	78.1	18.55	9.28
580	1.44	14.7320	10.09	0.004219	78.1	18.52	9.26
590	1.44	14.9860	10.27	0.004228	78.1	18.48	9.24
600	1.45	15.2400	10.44	0.004236	78.6	18.56	9.28
620	1.45	15.7480	10.79	0.004252	78.6	18.49	9.25
640	1.46	16.2560	11.14	0.004269	79.1	18.54	9.27
660	1.46	16.7640	11.48	0.004286	79.1	18.46	9.23
680	1.46	17.2720	11.83	0.004303	79.1	18.39	9.20

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Shelby Tube Visual

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON
Test Hole	TH21-02
Sample #	T16
Depth (m)	3.0 - 3.7
Sample Date	05-Oct-21
Test Date	12-Oct-21
Technician	JN

Tube Extraction

Recovery (mm) 620 (overpush)

Bottom - 3.7 m	3.58 m	3.38 m	3.18 m	3.10 m	**Top - 3 m**

Toss	Qu Bulk	Keep	Moisture Content PP/TV Visual	Slough
50 mm	200 mm	190 mm	80 mm	100 mm

Visual Classification

Material	CLAY
Composition	silty
Color	dark brown
Moisture	moist
Consistency	very soft
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.38
Vane Size (s,m,l)	l
Undrained Shear Strength (kPa)	7.5

Pocket Penetrometer (large 24 mm diam.)

Reading	**1**	1.50
	2	1.50
	3	1.50
	Average	1.50
Undrained Shear Strength (kPa)		4.6

Moisture Content

Tare ID	AC25
Mass tare (g)	6.8
Mass wet + tare (g)	326.8
Mass dry + tare (g)	193.0
Moisture %	71.9%

Unit Weight

Bulk Weight (g)		932.4
Length (mm)	**1**	150.36
	2	149.55
	3	151.11
	4	150.97
Average Length (m)		0.150
Diam. (mm)	**1**	69.57
	2	70.77
	3	71.33
	4	70.98
Average Diameter (m)		0.071
Volume (m^3)		5.90E-04
Bulk Unit Weight (kN/m^3)		15.5
Bulk Unit Weight (pcf)		98.6
Dry Unit Weight (kN/m^3)		9.0
Dry Unit Weight (pcf)		57.4

Unconfined Compressive Strength
ASTM D2166

Project No. 0814-001-00
Client Solid Construction Inc.
Project Central Community Club, Kenora, ON

Test Hole TH21-01
Sample # T16
Depth (m) 3.0 - 3.7
Sample Date 21-Jun-21
Test Date 28-Jun-21
Technician JN

Unconfined Strength

	kPa	ksf
Max q_u	15.6	0.3
Max S_u	7.8	0.2

Specimen Data

Description CLAY - silty, dark brown, moist, very soft, high plasticity

Length	150.5	(mm)	**Moisture %**	72%	
Diameter	70.7	(mm)	**Bulk Unit Wt.**	15.5	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	9.0	(kN/m^3)
Initial Area	0.00392	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

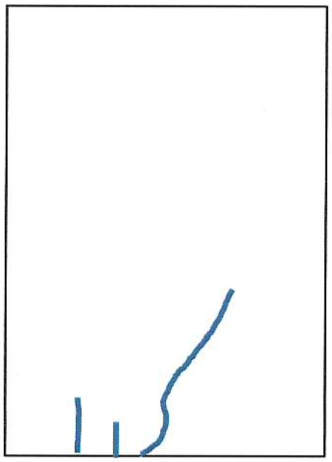
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.38	7.5	0.16
Vane Size		
I		

Pocket Penetrometer

	Reading	**Undrained Shear Strength**	
	tsf	**kPa**	**ksf**
	1.50	4.6	0.10
	1.50	4.6	0.10
	1.50	4.6	0.10
Average	**1.50**	4.6	**0.10**

Failure Geometry

Sketch:



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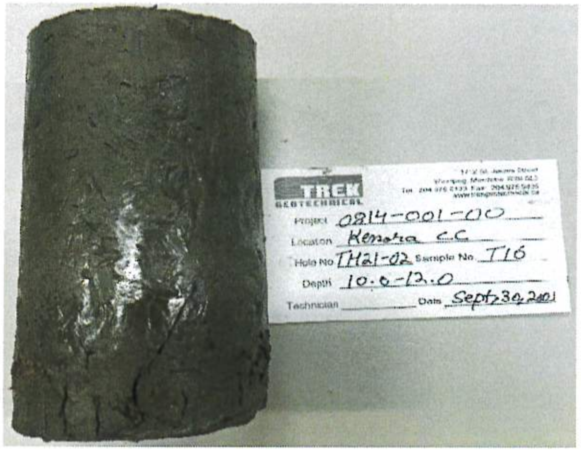
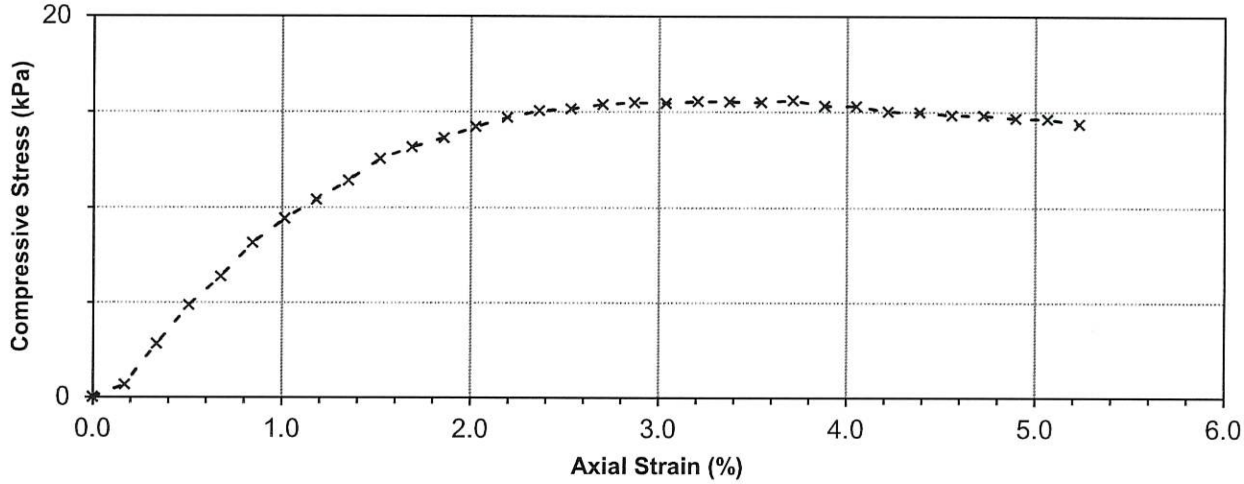


Unconfined Compressive Strength

ASTM D2166

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON

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	-0.11	0.0000	0.00	0.003922	0.0	0.00	0.00
10	-0.06	0.2540	0.17	0.003928	2.5	0.64	0.32
20	0.11	0.5080	0.34	0.003935	11.1	2.82	1.41
30	0.27	0.7620	0.51	0.003942	19.2	4.86	2.43
40	0.39	1.0160	0.68	0.003948	25.2	6.38	3.19
50	0.53	1.2700	0.84	0.003955	32.3	8.16	4.08
60	0.63	1.5240	1.01	0.003962	37.3	9.41	4.71
70	0.71	1.7780	1.18	0.003969	41.3	10.41	5.21
80	0.79	2.0320	1.35	0.003975	45.4	11.41	5.71
90	0.88	2.2860	1.52	0.003982	49.9	12.53	6.27
100	0.93	2.5400	1.69	0.003989	52.4	13.14	6.57
110	0.97	2.7940	1.86	0.003996	54.4	13.62	6.81
120	1.02	3.0480	2.03	0.004003	57.0	14.23	7.11
130	1.06	3.3020	2.19	0.004010	59.0	14.71	7.35
140	1.09	3.5560	2.36	0.004017	60.5	15.06	7.53
150	1.10	3.8100	2.53	0.004024	61.0	15.16	7.58
160	1.12	4.0640	2.70	0.004030	62.0	15.38	7.69
170	1.13	4.3180	2.87	0.004037	62.5	15.48	7.74
180	1.13	4.5720	3.04	0.004045	62.5	15.45	7.73
190	1.14	4.8260	3.21	0.004052	63.0	15.55	7.78
200	1.14	5.0800	3.38	0.004059	63.0	15.52	7.76
210	1.14	5.3340	3.54	0.004066	63.0	15.50	7.75
220	1.15	5.5880	3.71	0.004073	63.5	15.59	7.80
230	1.13	5.8420	3.88	0.004080	62.5	15.32	7.66

Project No. 0814-001-00
Client Solid Construction Inc.
Project Central Community Club, Kenora, ON

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	1.13	6.0960	4.05	0.004087	62.5	15.29	7.65
250	1.11	6.3500	4.22	0.004094	61.5	15.02	7.51
260	1.11	6.6040	4.39	0.004102	61.5	14.99	7.50
270	1.10	6.8580	4.56	0.004109	61.0	14.84	7.42
280	1.10	7.1120	4.73	0.004116	61.0	14.82	7.41
290	1.09	7.3660	4.89	0.004123	60.5	14.67	7.33
300	1.09	7.6200	5.06	0.004131	60.5	14.64	7.32
310	1.07	7.8740	5.23	0.004138	59.5	14.37	7.19

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Shelby Tube Visual

Project No.	0814-001-00
Client	Solid Construction Inc.
Project	Central Community Club, Kenora, ON

Test Hole	TH21-03
Sample #	T24
Depth (m)	3.0 - 3.7
Sample Date	05-Oct-21
Test Date	12-Oct-21
Technician	JN

Tube Extraction

Recovery (mm) 630 (overpush)

Bottom - 3.7 m 3.55 m 3.46 m 3.24 m **Top - 3 m**

Keep	Moisture Content PP/TV Visual	Toss	Slough
150 mm	90 mm	220 mm	160 mm

Visual Classification

Material	CLAY
Composition	silty
Color	dark brown
Moisture	moist
Consistency	soft to firm
Plasticity	intermediate plasticity
Structure	-
Gradation	-

Torvane

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

Pocket Penetrometer

Reading	**1**	0.50
	2	0.50
	3	0.40
	Average	0.47
Undrained Shear Strength (kPa		22.9

Moisture Content

Tare ID	AB35
Mass tare (g)	6.8
Mass wet + tare (g)	262.6
Mass dry + tare (g)	155.2
Moisture %	72.4%

Unit Weight

Bulk Weight (g)		913.8
Length (mm)	**1**	146.58
	2	145.55
	3	144.59
	4	146.60
Average Length (m)		0.146
Diam. (mm)	**1**	71.52
	2	71.99
	3	70.71
	4	70.61
Average Diameter (m)		0.071
Volume (m^3)		5.81E-04
Bulk Unit Weight (kN/m^3)		15.4
Bulk Unit Weight (pcf)		98.2
Dry Unit Weight (kN/m^3)		9.0
Dry Unit Weight (pcf)		57.0