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APPENDIX ‘A’

GEOTECHNICAL REPORT

APPENDIX ‘A’ - GEOTECHNICAL REPORT

The geotechnical report is provided to aid in the Contractor’s evaluation of the existing pavement structure and/or soil conditions. The information presented is considered accurate at the locations shown on the Drawings and at the time of drilling. However, variations in pavement structure and/or soil conditions may exist between test holes and fluctuations in groundwater levels can be expected seasonally and may occur as a result of construction activities. The nature and extent of variations may not become evident until construction commences.

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Dillon Consulting Ltd.

Geotechnical Investigation Report for City of Winnipeg Community Resource Recovery Centres – Pacific Avenue Site

Prepared for:

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TREK GEOTECHNICAL Quality Engineering | Valued Relationships

January 17, 2014

Our File No. 0022 010 00

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RE: Final Geotechnical Investigation Report for City of Winnipeg Community Resource Recovery Centres – Pacific Avenue Site

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

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

Sincerely,

TREK Geotechnical Inc.
Per:

Michael Van Helden, Ph.D., P.Eng.
Geotechnical Engineer, Principal
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/mvh
Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	BSH	Dec 23, 2013	DRAFT Final Report
1	BSH	Jan 17, 2014	Final Report

Authorization Signatures

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1.0 Introduction

This report summarizes the results of the geotechnical investigation completed by TREK Geotechnical Inc. (TREK) for the proposed City of Winnipeg Community Resource Recovery Centre (4R Depots) located on Pacific Avenue just west of McPhillips Street. The terms of reference for the investigation are included in the short-term consulting services agreement issued by Dillon on November 12, 2013. The scope of work includes a sub-surface investigation, laboratory testing, and the provision of recommendations for the design and construction for suitable foundation systems including piled foundations, spread footings, at-grade floor slabs (including exterior slabs), retaining walls and pavement sections.

2.0 Background and Existing Information

The construction of Community Resource Recovery Centres is a part of the City of Winnipeg's Comprehensive Integrated Waste Management Plan (CIWMP). The sites are open to the public to receive all manner of residential materials for disposal with a focus on source separation and division. At the time of the investigation, the site was relatively flat with scattered construction debris around the entrance off Pacific Ave. Surrounding the debris area, the site is largely covered with grass, having a few large trees along the east and west edges of the site. Abandoned railway lines exist on the east and west sides of the site. The site location is shown on Drawing 01 (attached). The proposed development consists of access roads and parking areas, a weigh scale and associated scale house and waste disposal bin areas. The waste disposal bins will be situated at the base of a retaining wall and founded on at-grade concrete slabs. A future re-use centre will also be developed at the north west corner of the main site. An overview of the proposed site development (provide by Dillon) is shown on Drawing 01 (attached) and is based on the information provided by Dillon in Appendix A. It is our understanding that the attendant shack, equipment garage and future re-use centre may be founded on shallow foundations or deep (piled) foundations, depending on the tolerance for seasonal movement.

Existing information provided to TREK was reviewed and is included in Appendix A:

- **Final Concept Drawings for the Pacific Avenue 4R Depot (Dillon, November 2013):** Provides overview of the proposed development features and layout.

3.0 Field Program

3.1 Sub-Surface Investigation

A subsurface investigation was undertaken on October 29th, 2013 under the supervision of TREK personnel to determine the soil stratigraphy and groundwater conditions at the site. Test holes were drilled using a Soilmec STM-20 truck mounted piling rig equipped with 508 mm diameter augers.

Subsurface soils observed during the drilling were visually classified based on the Unified Soil Classification System (USCS). Other pertinent information such as drilling, groundwater and backfill conditions was also recorded. Samples retrieved during drilling included disturbed grab samples and relatively undisturbed Shelby tubes, and were transported to TREK's laboratory in Winnipeg, Manitoba for further analysis. Laboratory testing consisted of moisture content determination on all samples. Undrained shear strength testing (pocket penetrometer, torvane and unconfined compression), unit weight determination and Atterberg Limits were also completed on select samples.

Nine test holes (TH13-11 to TH13-19) were drilled at the locations shown on Drawing 01. Test holes TH13-11 & 14 were advanced to Power Auger Refusal (PAR) at depths of 14.0 and 15.2 m below ground surface, respectively. The remaining test holes were relatively shallow (3.0 to 3.3 m depth) and completed to evaluate near surface conditions.

Test hole logs are attached in Appendix B and include soil descriptions, the elevation of soil units encountered and other pertinent information such as groundwater levels and sloughing conditions. Test hole locations (UTM Coordinates) and elevations were provided by Dillon and are presented on the test hole logs. Laboratory test results are included in Appendix C.

3.2 Sub-Surface Conditions

The subsurface stratigraphy in descending order from ground surface consists of clay (fill), silt, silty clay and silt (till). A brief description of the soil units encountered at the test hole locations is provided below. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed test hole logs in Appendix B.

Clay (Fill)

Clay (fill) was encountered at surface in all test holes. The clay fill extends to depths ranging from 0.6 m to 2.1 m. The clay fill is silty, contains trace sand, trace gravel, trace rootlets, is brown to dark brown, moist, stiff to very stiff and of high plasticity. Moisture contents range from 22 % to 56 %, with an average of 34 %.

Silt

Silt was encountered underlying clay fill in all test holes extending to depths ranging from 1.7 to 3.0 m below surface with an average of 2.5 m. The silt generally contains trace clay, trace fine grained sand, trace gravel, is light brown, moist, firm and of low plasticity. The moisture content of the silt ranged from 20 % to 33 % with an average of 26 %. Atterberg limit results from one sample indicate a plastic limit of 15 % and a liquid limit of 28 %.

Silty Clay

Silty clay was observed in all test holes below the silt. The clay extended to 11.9 m and 14.0 m below surface (Elev. 220.3 and 218.3 m) in TH13-11 & 14, respectively. The remaining eight test holes were terminated within the silt or silty clay. The silty clay contains trace silt inclusions, is grey, moist and of high plasticity. The moisture content of the clay was generally consistent with depth, ranging from 33 % to 60 %, with an average of 51 %. Atterberg limit results from one sample indicate a plastic limit of 26 % and a liquid limit of 101 %.

Six unconfined compressive strength tests were performed resulting in undrained shear strengths ranging from 14.8 to 40.5 kPa. Based on these tests and the results of torvane and pocket penetrometer tests, the consistency of the clay is considered firm to stiff, generally becoming softer with depth. Bulk unit weights of the clay range from 16.3 to 17.0 kN/m^3 with an average of 16.5 kN/m^3 based on six samples.

Silt (Till)

Silt (till) underlies the silty clay at approximately 11.9 to 14.0 m depth based on test holes TH13-11 & 14. Power auger refusal was reached at depths between 14.0 and 15.2 m (Elev. 218.2 and 217.1 m) in test holes TH13-11 & 14. The silt (till) contains trace clay, trace sand to being sandy, trace gravel, trace cobbles is moist, light grey and is of no to low plasticity. Moisture contents of the silt (till) are 9.0, 9.2, 10.6 and 12.7 % having no clear trend with depth.

3.3 Seepage and Sloughing Conditions

Sloughing was not observed in any of the test holes. Seepage was observed in test hole TH13-11 at 4.6 m in the silty clay and in TH13-14 at 14.3 m in the silt (till). Water levels were measured within the test holes upon completion of drilling at 4.1 and 4.3 m below surface in TH13-11 & 14, respectively. These observations are short term and should not be considered reflective of (static) groundwater levels in the silty clay, which would require monitoring over an extended period to determine. It is important to recognize that groundwater conditions may change seasonally, annually, or as a result of construction activities.

4.0 Foundation Recommendations

4.1 Limit States Design

Limit states design recommendations according to 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 probabilistic reliability criteria. Two general design scenarios are evaluated corresponding to the serviceability and ultimate capacity requirements.

The **Ultimate Limit State (ULS)** is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation units. The ULS foundation bearing capacity is obtained by multiplying the nominal (ultimate) bearing capacity by a resistance factor (reduction factor), which is then compared to the factored (increased) structural loads. The ULS bearing capacity must be greater or equal to the maximum factored load. Table 1 summarizes the resistance factors that can be used for the design of piles 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 (or unit resistances) are provided that are developed on the basis of limiting settlement to approximately 25 mm or less. A more detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust the SLS foundation capacity if a more stringent settlement tolerance is required.

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

Case	Resistance Factor
Deep Foundation with bearing resistance to axial load based on semi-empirical analysis using laboratory and *in-situ* test data.	0.4
Deep Foundation with analysis using dynamic monitoring results (PDA Testing with CAPWAP Analysis)	0.5
Deep Foundation with analysis using static loading test results	0.6
Shallow Foundations for bearing resistance	0.5

4.2 Foundation Options

It is understood that, among other structures, a reinforced concrete retaining wall with a “saw-tooth” layout in plan view is proposed to provide a dumping area for waste disposal bins and that shallow foundations are preferred, if feasible. Recommendations are provided for shallow foundations (footings) although these should only be used if the structure can tolerate some seasonal differential movements. Alternatives for the retaining wall such as driven sheet piles should be considered and could decrease construction costs (recommendations for lateral earth pressures are provided in Section 6.0). Segmental block (geosynthetic reinforced earth) retaining walls are typically designed by the manufacturer, however TREK can provide a design review and external stability check if required. Deep foundations are recommended for any structures that may be sensitive to seasonal soil movements associated with freeze/thaw and/or wetting/drying cycles.

Site conditions, structure types and anticipated foundation loads make this site best suited for cast-in-place friction piles and driven pre-cast concrete piles as deep foundation options. Cast-in-place piles end-bearing in till may also be a suitable foundation option if increased pile loads are required; recommendations can be provided for this option if required.

4.2.1 Shallow Foundations

Provided seasonal movements relating to moisture changes in the soil are tolerable, a shallow (footing) foundation bearing on undisturbed silty clay would be an appropriate foundation system. To eliminate the effects from freeze/thaw footings should be placed below 2.4 m depth. Alternatively, footings above 2.4 m depth can be insulated to provide an equivalent level of frost protection. TREK can provide recommendations on insulation thickness and limits, if requested. Unrestrained differential soil movements associated with moisture changes can be expected to be up to 50 to 100 mm.

Based on the measured undrained shear strengths and the ULS resistance factors provided in Table 1, the ULS bearing capacity appropriate for design is 125 kPa. The SLS bearing capacity appropriate for design is 85 kPa and is based on settlements of less than 25 mm.

If increased bearing capacity is required beneath the footing, a compacted granular pad may be constructed below the base of the slab or thickened edge to distribute the contact load to maintain a ULS bearing pressure of 125 kPa and SLS bearing pressure of 85 kPa on the clay underlying the granular pad. In plan, the compacted granular pad should extend beyond the edge of the footing by at least the gravel thickness. The allowable bearing pressure on the gravel pad can be calculated using the following formulae:

ULS Bearing Capacity = 125 (w+d)/w

SLS Bearing Capacity = 85 (w+d)/w

where:
w = width of the footing (m)
d = depth of gravel below the footing (m)

The granular pad should be constructed using 50 mm down crushed limestone with the upper 100 mm of the granular pad constructed using 20 mm down crushed limestone as a levelling course. The crushed limestone should be compacted to a minimum of 98% SPMDD.

Additional considerations for the design and construction of footings are provided below.

1. The base width for footings should meet requirements established by the City of Winnipeg.
2. Organics, silts, fill soils, and any other deleterious material should be stripped such that the subgrade consists of native, undisturbed high plastic clay. Based on the exploration this could result in excavation of up to 3.0 m. Excavation should be completed by a backhoe equipped with a smooth bladed bucket in a manner which minimizes disturbance to the exposed subgrade. Care should be taken not to over-excavate and to minimize the subgrade disturbance at all times. Fill required to raise grades or for levelling should consist of a 20 mm down crushed rock compacted to 100% SPMDD.
3. The subgrade should be inspected by qualified geotechnical personnel prior to concrete placement.

4. Where soft or weak subgrade materials are identified by the geotechnical personnel, these areas should be repaired as directed by the geotechnical engineer. This may require excavation and placement/compaction of granular fill. A typical repair for this application would involve excavation to 300 mm below the design subgrade elevation, followed by backfilling and compaction using granular sub-base materials.

4.2.2 *Cast in Place Concrete Friction Piles*

Pile capacities for evaluation of the Ultimate and Service Limit States can be calculated based on the SLS and ULS (factored) unit resistances provided in Table 2. A ULS resistance factor of 0.4 was selected associated with resistances based on field observations and laboratory testing. The pile settlement under applied (unfactored) loads equal to the SLS pile capacity can be expected to be 25 mm or less. If required, a detailed settlement analysis can be provided by TREK once the final pile loads are known.

The SLS pile capacity should be calculated based on the skin friction resistance only, which is consistent with traditional friction pile design. The ULS pile capacity should be calculated based on the total pile capacity at ultimate (plunging) failure, which would consist of both skin friction and end-bearing components; factored ULS resistances for both are provided.

Table 2. Recommended Limit State Unit Resistances for Friction Piles

Soil	Depth (m)		ULS Resistance		SLS Skin Friction
	From	To	End-Bearing	Skin Friction	
Clay Fill / Frost Zone	0.0	2.4	0	0	0
Silty Clay	2.4	11	60 kPa	14 kPa	12 kPa

[1] ULS resistance = A Resistance Factor of 0.4 is applied.

Additional design and construction recommendations for cast-in-place concrete piles are provided below:

1. The weight of the embedded portion of the pile may be neglected.
2. The contribution from end bearing should be ignored.
3. Adhesion within the upper 2.4 m of the pile or fill soils if encountered deeper than 2.4 m should be ignored to take into consideration potential shrinkage and environmental effects such as frost action over that depth. Shaft support within any fill materials should also be ignored. A skin friction of 10 kPa (resistance factor of 0.3 has been applied) should be used for calculating uplift resistance against live loads on the piles. A minimum pile length of 9 m should be used to resist uplift forces due to frost jacking.
4. Friction piles should not extend below Elev. 221.0 m to prevent piles from penetrating the underlying till. Should any piles penetrate the till unit, differential settlement between piles may occur.

5. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre. If pile spacing must be closer than 2.5 pile diameters, TREK should be notified so that an evaluation of pile group effects can be performed.
6. Grade beams and pile caps should be constructed with a minimum 150 mm void space to minimize the effects of soil heave due to swelling or frost action.
7. Any existing structures, foundation components or concrete rubble encountered during construction should be excavated and removed to a depth of at least 0.5 m from the underside of grade beams and pile caps and in their entirety at pile locations.
8. All cast-in-place piles require reinforcement design by a qualified structural engineer for the anticipated axial, lateral and bending loads from the structure.
9. Based on observed conditions, shallow sleeving of pile holes may be required. Seepage conditions at the time of construction may differ from that observed at the time of drilling, in particular from near surface layers (e.g. silt) and if seepage and sloughing conditions are observed during drilling of pile holes, sleeves should be used.
10. Drilling and concrete placement for the piles should be inspected by geotechnical personnel to verify the soil conditions and proper installation of the piles.
11. Prior to casting the pile, any groundwater within the shaft should be removed or controlled. If water is present the concrete should be placed using Tremie methods.
12. Concrete should be placed as soon as possible after drilling of the pile shaft.

4.2.3 Driven Pre-Cast Concrete Piles

If larger load resistance is required at this site consideration could be given to using pre-cast concrete piles driven to refusal in dense silt (till). The depth to silt till measured at 2 test hole locations ranged from 11.9 to 14.0 m, which suggests that the depth to pile refusal may be more variable than what is typically encounterd in the Winnipeg area. Additionally pile lengths may approach the practical limits for a driven pre-cast pile (generally a maximum length of 20 m is fabricated). Both SLS and ULS pile capacities are provided in Table 3 for precast concrete piles driven to practical refusal within the glacial till with the specified hammer and set criteria. Based on field observations and laboratory testing, the use of a resistance factor value of 0.4 has been applied to the estimated nominal end bearing value to arrive at the recommended ULS values provided in Table 3. A resistance factor of 0.6 may only be used for driven piles if a static pile load test is carried out at the site (which we anticipate will not be cost-effective). A resistance factor of 0.5 may only be used for driven piles if dynamic monitoring (e.g. PDA Testing with CAPWAP analysis) is carried out at the site during construction. If desired, TREK can provide recommendations on the number of piles to be used in either static or dynamic testing once a preliminary pile layout has been developed.

The SLS capacity provided in Table 3 will result in settlements of less than 25 mm. If a more stringent settlement criterion is required, a detailed settlement analysis can be provided by TREK once the final pile loads are known.

Table 3. Recommended Limit State Pile Capacities for Driven Precast Concrete Piles

Pile Type	Pile Size	ULS Capacity (kN)	SLS Capacity (kN)	Refusal Criteria (Blows/25 mm)
Driven Precast Piles	300 mm	580	445	5
	360 mm	800	625	8
	405 mm	1040	800	12

**Refusal criteria to be met on three consecutive sets using a hammer with a minimum rated energy of 40 kJ per blow*

Additional design and construction recommendations for driven precast concrete piles are provided below:

1. The weight of the embedded portion of the pile may be neglected.
2. The piles must be designed to withstand design loads, handling stresses, and driving forces during installation.
3. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre. If pile spacing must be closer than 2.5 pile diameters, TREK should be notified so that an evaluation of pile group effects can be performed.
4. The piles should be specified to have cured for at least 7 days prior to driving.
5. To aid in pile alignment, reduce ground vibrations, and reduce pile heave during driving, pre-boring may be undertaken. The pre-bore depth should be less than 3 m and the pre-bore diameter should be no more than 50 mm larger than the pile diameter. If lateral resistance is required in the piles, the annulus surrounding the pre-bore section of the piles should be filled with lean mix concrete for compliance with the surrounding soil.
6. Piles should be driven continuously once driving is initiated to the required refusal criteria.
7. All piles driven within 5 pile diameters of the pile being driven should be monitored for pile heave during installation. If pile heave is observed, all piles should be checked. Piles that have heaved should be re-driven to the refusal criteria.
8. Where a steel follower is required to install piles below the surrounding ground surface, the refusal criteria should be increased by up to 50% in order to account for additional energy losses through the use of the follower. TREK should be contacted to provide recommendations in this regard during construction.
9. Inspection of the driven pile installation should be undertaken by qualified and experienced geotechnical personnel who are familiar with this type of pile installation.
10. Any piles damaged, misaligned an excessive amount or reaching premature refusal may need to be replaced. The structural designer should assess non-conforming piles to determine if they are acceptable.
11. Grade beams and pile caps should be constructed with a minimum 150 mm void space to minimize the effects of soil heave due to swelling or frost action.

12. Any existing structures, foundation components or concrete rubble encountered during construction should be excavated and removed to a depth of at least 0.5 m from the underside of grade beams and pile caps and in their entirety at pile locations.

4.2.4 Lateral Pile Loading

The soil response (subgrade reaction) to lateral loads can be modeled in a simplified manner that assumes the soil around a pile can be simulated by a series of horizontal springs for the preliminary design of pile foundations. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral subgrade reaction modulus (k_s). For clays, the lateral subgrade reaction modulus is typically independent of depth or vertical overburden stress. Table 4 provides the recommended subgrade reaction modulus for the lateral load analysis.

The majority of lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the relative stiffness of the pile and soil units. Void spaces surrounding piles due to pre-boring activities should be in-filled with lean-mix concrete to ensure compliance with the surrounding soil.

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

Soil	Depth (m)		K_s (kN/m^3)
	From	To	
Fill Soils, Silts, and Clays	0.0	11	2400/d[1]

[1] d is the pile diameter in metres.

As part of detailed design, a more rigorous lateral pile and group analysis that incorporates the material and section properties of the pile, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading can be carried by TREK out to confirm the lateral load capacity of the piles and pile group, if required.

5.0 Grade Supported Concrete Floor Slabs

It is understood that the proposed new structures may include grade supported concrete slabs. Some vertical deformation of grade supported slabs should be expected due to moisture and volume changes of the underlying clay soils. Additionally, floor slabs in unheated areas will be subject to additional movements from freeze/thaw of the subgrade soils. The following recommendations are provided to reduce or accommodate potential movements of the slab:

1. Generally it is recommended that organics, silts, fill soils and any other deleterious material be stripped such that the subgrade consists of native high plastic silty clay, however the depth of fills/silts at this site (up to 3 m) will likely make complete removal cost prohibitive. Provided some additional settlement and seasonal movements can be tolerated this material may remain in place. In this case surficial organics should be stripped and the exposed clay (fill) subgrade should be scarified to a depth of 300 mm, moisture conditions and re-compacted to 95% of Standard Proctor Maximum Dry Density (SPMDD).

If the risk of increased seasonal movements and settlement is not tolerable all fill soils and silt should be stripped such that the subgrade consists of native high plastic silty clay. Excavation should be completed with a backhoe equipped with a smooth bladed bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed subgrade. Care should be taken not to over-excavate and to minimize the subgrade disturbance at all times.

2. Any existing structures, foundation components or concrete rubble encountered during construction should be removed to 0.5 m below the design subgrade elevation and backfilled with compacted base course material.
3. The sub-grade should be protected from freezing, drying, or inundation with water. If any of these conditions occur the sub-grade should be moisture conditioned as appropriate, scarified and re-compacted to 95% of Standard Proctor Maximum Dry Density (SPMDD).
4. After excavation, the subgrade should be inspected by qualified geotechnical personnel. The subgrade should be proof-rolled with a fully loaded tandem axle truck to detect weak or soft areas. Soft areas should be repaired by sub-excavating to a maximum depth of 0.3 m below the sub-grade level, covered in geo-textile and backfilled with well graded granular compacted to 98% SPMDD.
5. Fill required to raise grades should consist of well graded granular fill compacted to 98% SPMDD.
6. The granular base should consist of well-graded crushed rock compacted to 98% of SPMDD. The granular section should consist of a minimum of 150 mm of crushed limestone base material (19 mm down) overlying 150 mm of crushed limestone sub-base material (50 mm to 75mm down). In unheated areas, the thickness of granular sub-base material should be increased to 250 mm. The granular base should be placed and compacted in lifts not exceeding 150 mm thickness.
7. To minimize changes in soil moisture beneath grade supported floor slabs, the discharge from roof leaders and run-off from exposed slabs should be directed away from the structures.
8. 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 (e.g. grade beams). Allowances should be made to accommodate vertical movements of light-weight structures (e.g. partitions) bearing on the slab.
9. Consideration should be given to providing a sub-floor drainage system consisting of a perimeter weeping tile drain.

6.0 Lateral Earth Pressure

The magnitude of lateral earth pressures from retained soil against permanent walls will depend on the backfill material type and degree of compaction, method of placement, and the magnitude of horizontal deflection of the wall after the backfill is placed. It is recommended that free draining granular soil be used as backfill against permanent walls to improve drainage properties and minimize the potential of lateral frost heave loading. A sub-drainage system consisting of filter-wrapped drainage pipe backfilled with clean gravel should be used at the base of the wall backfill to prevent the build-up of hydrostatic pressures behind the wall structures. Cohesive soils should not be used as backfill behind permanent walls as these soils could generate excessive lateral earth pressures from swelling. For cantilever retaining walls founded below grade, earth pressures due to native soils would be acting on both sides of the embedded portion of the wall.

Table 5 provides earth pressure coefficients and bulk unit weights for compacted granular backfill as well as native clays and silts. An active pressure coefficient (K_a) should be used to calculate lateral loads from retained soils where structures which are free to translate horizontally by at least 0.2 percent of the retaining wall height. An at-rest earth pressure coefficient (K_o) should be used where structures are not free to translate. An appropriate surface surcharge should also be included in the earth pressure distribution to account for surface loads. The active pressure coefficient (K_a) can be used to calculate the component of lateral loads on wall structures due to surcharge loads. A passive earth pressure coefficient (K_p) can be used for lateral earth pressures acting on the down-slope side of retaining structures to resist lateral wall movement, provided soil strains of 2 to 5% can be mobilized. In this regard, actual earth pressures acting on the down-slope face may be between the at-rest and passive earth pressure conditions and TREK should be involved in the selection of lateral earth pressures for final design. The effective (buoyant) unit weight should be used to calculate the earth pressures due to soils below the groundwater table. In this regard, a groundwater table at original ground surface should be assumed for the purposes of preliminary design.

Table 5. Recommended Parameters for Lateral Earth Pressure Calculations

Soil	Bulk Unit Weight	Earth Pressure Coefficient		
		Active (K_a)	At-Rest (K_o)	Passive (K_p)
Granular Backfill	22 kN/m^3	0.3	0.4	3.0
Native Soils (Silts and Clays)	17 kN/m^3	0.55	0.70	1.8

Over-compaction of the backfill soils adjacent to walls may result in earth pressures that are considerably higher than those predicted in design. Compaction of the granular fills within about 1.5 m of retaining walls should be conducted with a light hand operated vibrating plate compactor and the number of compaction passes should be limited. A maximum compacted density of 92% of Standard Proctor Maximum Dry Density (SPMDD) should be specified for fill placed adjacent to walls. Granular backfill placed downslope of retaining walls and within 3 m of the retaining wall should be compacted to 100% of SPMDD. Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

7.0 Foundation Concrete

Based on TREK's experience with soils in the Winnipeg area the degree of exposure for concrete subjected to sulphate attack is classified as severe according to Table 3, CSA A23.1-09 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56 day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-09 for concrete with severe sulphate exposure (S2). Concrete which may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-09.

8.0 Pavement Design

Recommendations for asphalt pavement structure for residential traffic areas and areas that will be subjected to heavier vehicular loads (operational traffic) such as access roads and loading areas are provided in Table 6.

Table 6. Recommended Sections for Asphalt Pavements

Material	Layer Thickness		Compaction Requirements
	Car Parking Areas	Heavy Vehicular Loads	
Asphalt	75 mm	75 mm	98% Marshall Density
20 mm down limestone or recycled concrete	150 mm	150 mm	100% of SPMDD
50 mm down limestone	250 mm	350 mm	98% of SPMDD
Non-Woven Geotextile (Geotex 801 or equivalent)	Optional	Required	Install as per manufacturer's recommendations

1. Organics, silts, fill soils, and any other deleterious material should be stripped such that the subgrade consists of native high plastic silty clay. Excavation should be completed with a backhoe equipped with a smooth bucket and operating from the edge of the excavation in order to minimize disturbance to the exposed subgrade. Care should be taken not to over-excavate and to minimize the subgrade disturbance at all times.

   As an alternative, consideration could be given to removing surficial silts and organics and leaving the existing clay (fill) in place. In this case additional movement resulting from volume changes in the clay (fill) soils should be expected. To minimize the effects of leaving the fill in place the upper 0.3 m of fill should be scarified, moisture condition and re-compacted to 95% of SPMDD.

2. The subgrade should be protected from freezing, drying, or inundation with water. If any of these conditions occur the subgrade should be moisture conditioned as appropriate, scarified and re-compacted to 95% of SPMDD.
3. After excavation, the subgrade should be inspected by qualified geotechnical personnel. The subgrade should be proof-rolled with a fully loaded tandem axle truck to detect weak or soft areas. Soft areas should be repaired by sub-excavating to a maximum depth of 0.3 m below the sub-grade level, covered in geo-textile and backfilled with 50 or 100 mm down crushed limestone compacted to 98% SPMDD.
4. Fill required to raise grades should consist of well graded granular fill compacted to 98% SPMDD. 100 or 150 mm down well graded granular would be appropriate for use.

9.0 Drainage

Drainage adjacent to site buildings or structures should promote runoff away from the structures. A minimum slope of about 2% should be used for both landscaped and paved areas immediately around structures. All paved areas should be provided with minimum slopes of 2% to improve long-term drainage. All roof leaders should be extended sufficiently away from the building walls.

10.0 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, laboratory testing, geometries). 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.

Drawings

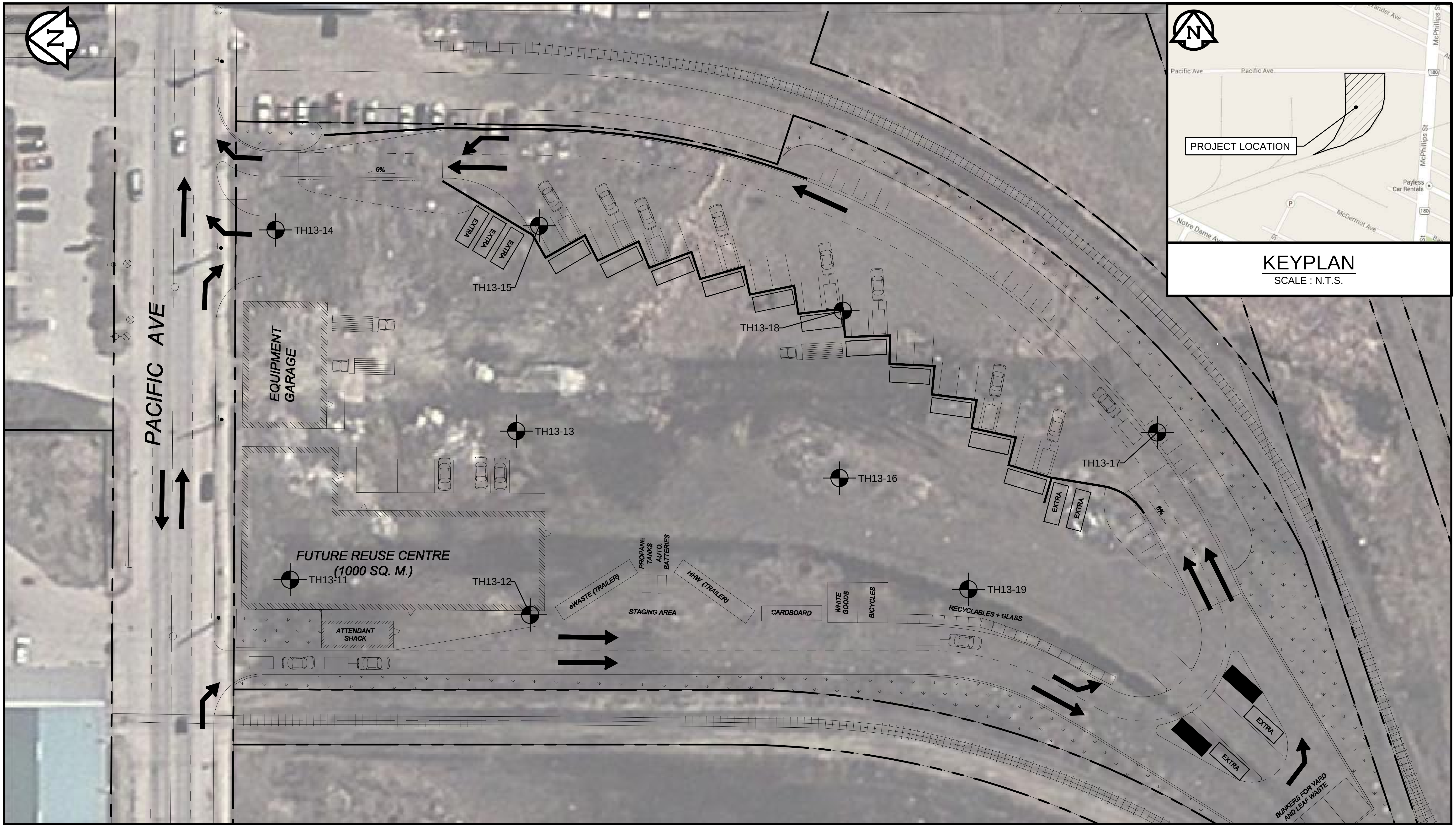
TREK
GEOTECHNICAL

0022 010 00
Dillon Consulting Ltd.
City of Winnipeg Resource Recovery Centres

Tabloid (279mm x 432mm)

PLOT: 16/01/2014 10:58:04 AM

FILE NAME: 0022 010 00_RI.dwg



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

LEGEND :

⌖ TEST HOLE (TREK, OCTOBER 29, 2013)

— - — APPROXIMATE PROPERTY LINE

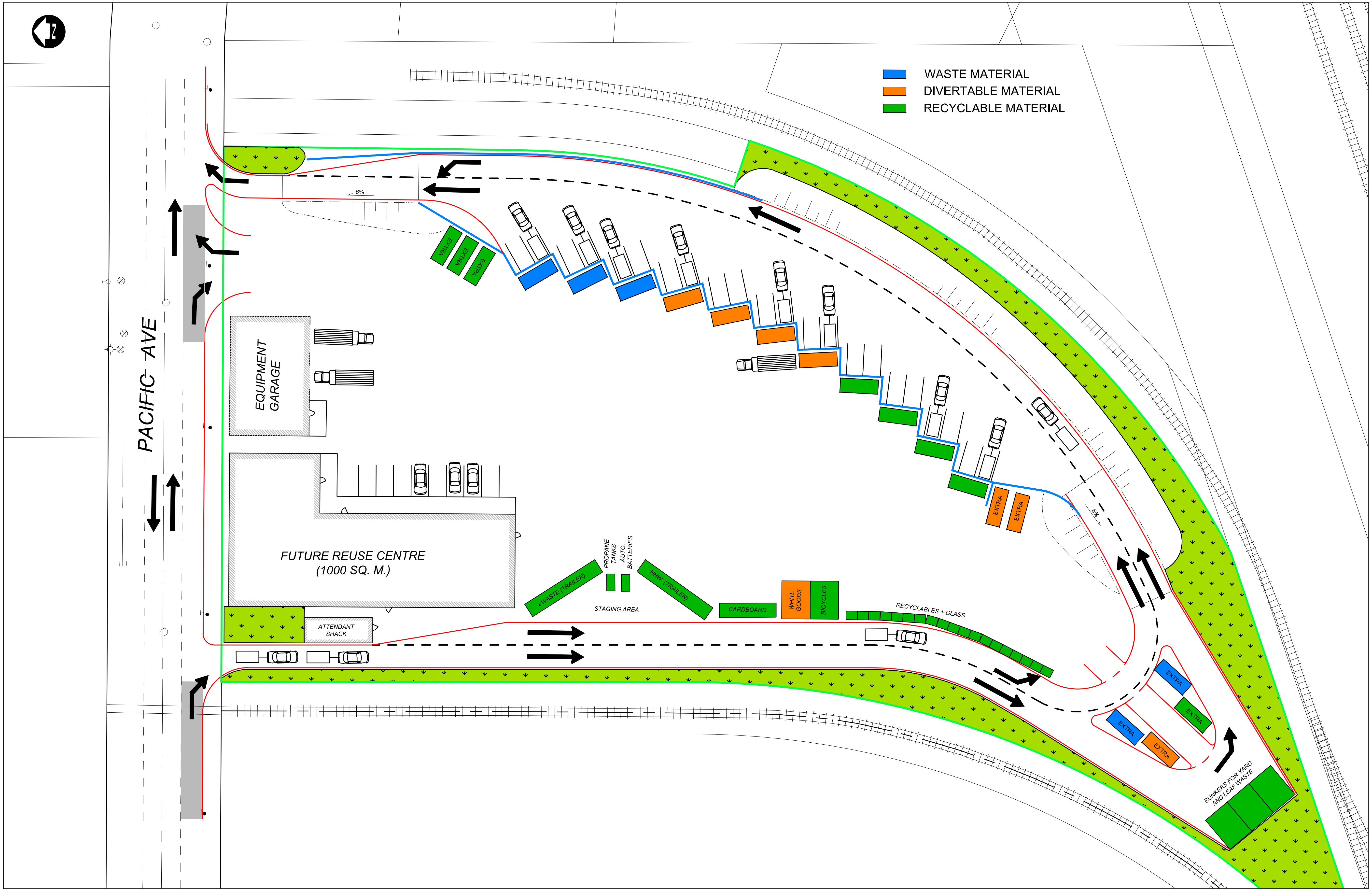
NOTES :

1. AERIAL IMAGE FROM GOOGLE EARTH, MAY 2, 2013

Drawing 01
Test Hole Location Plan
Pacific Avenue

Appendix A

Existing Information



PLAN

DILLON CONSULTING

NOVEMBER 2013

SCALE 1:300

TITLE

4R DEPOT - PACIFIC AVE.
OPTION 4

DRAWING NO.

REF-17

G:\CAD\138224\02-Civil\09-Reference Internal\REF-17_PACIFIC_option5.dwg

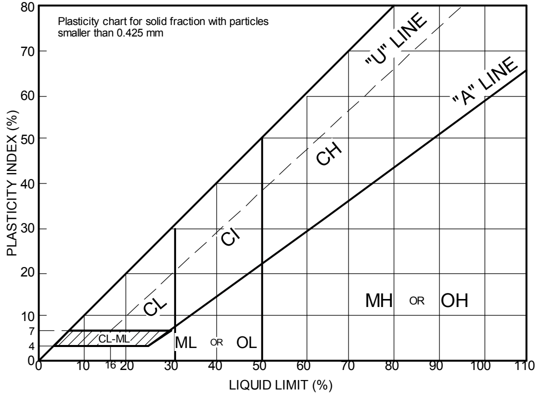
Appendix B

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 Classi-fication	Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW	●	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent...... GW, GP, SW, SP More than 12 percent...... GM, GC, SM, SC 6 to 12 percent...... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	○	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM	▥	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are 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	Particle Size 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	Particle Size 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

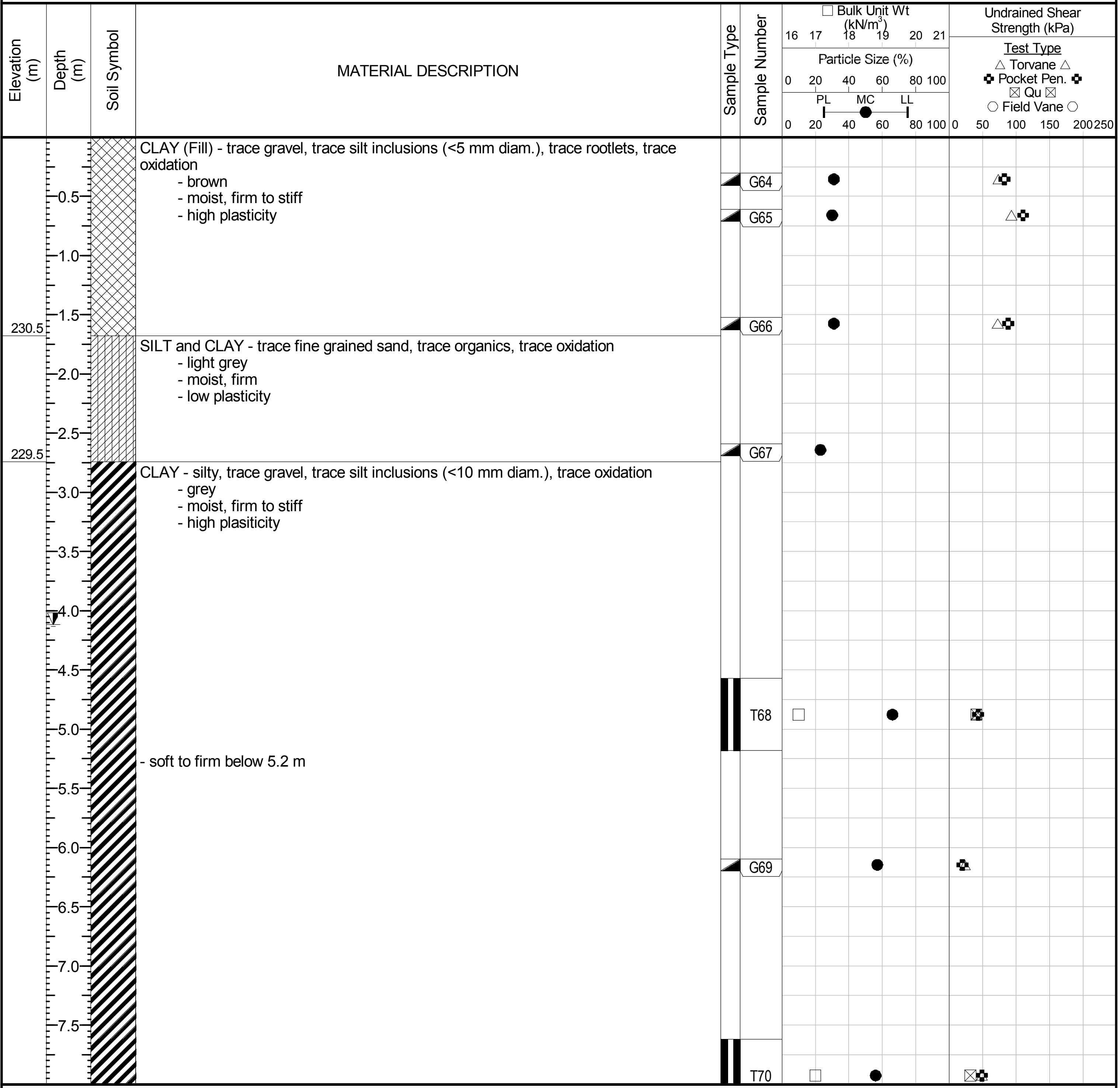
Sub-Surface Log

Test Hole TH13-11

Client:	Dillon Consulting Ltd.	**Project Number:**	0022 010 00
Project Name:	City of Winnipeg Resource Recovery Centres	**Location:**	UTM N-5530425, E631111 (Pacific Industrial)
Contractor:	Subterranean Ltd.	**Ground Elevation:**	232.20 m
Method:	508 mm Auger, Soilmec STM-20 Truck Mount	**Date Drilled:**	29 October 2013

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

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt ░ Sand ● Gravel ▱ Cobbles ■ Boulders



CLAY (Fill) - trace gravel, trace silt inclusions (<5 mm diam.), trace rootlets, trace oxidation
- brown
- moist, firm to stiff
- high plasticity

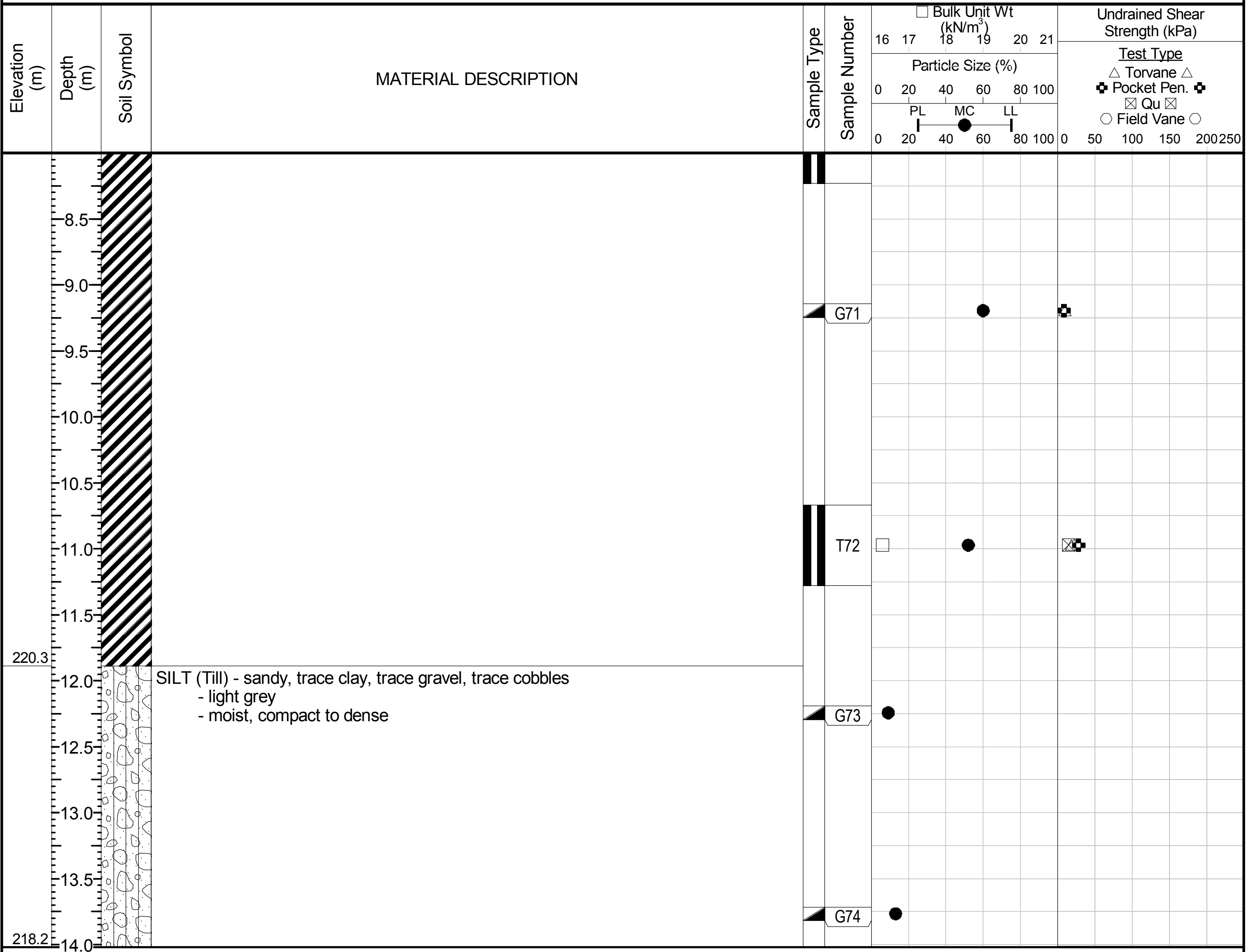
SILT and CLAY - trace fine grained sand, trace organics, trace oxidation
- light grey
- moist, firm
- low plasticity

CLAY - silty, trace gravel, trace silt inclusions (<10 mm diam.), trace oxidation
- grey
- moist, firm to stiff
- high plasticity

- soft to firm below 5.2 m

Logged By: Beta Taryana **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

Sub-Surface Log



SILT (Till) - sandy, trace clay, trace gravel, trace cobbles
- light grey
- moist, compact to dense

POWER AUGER REFUSAL AT 14.0 m in SILT (Till)
Notes:
1) Seepage observed at 4.6 m below surface.
2) No sloughing observed.
3) Water level at 4.1 m below surface upon completion of test hole.
4) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

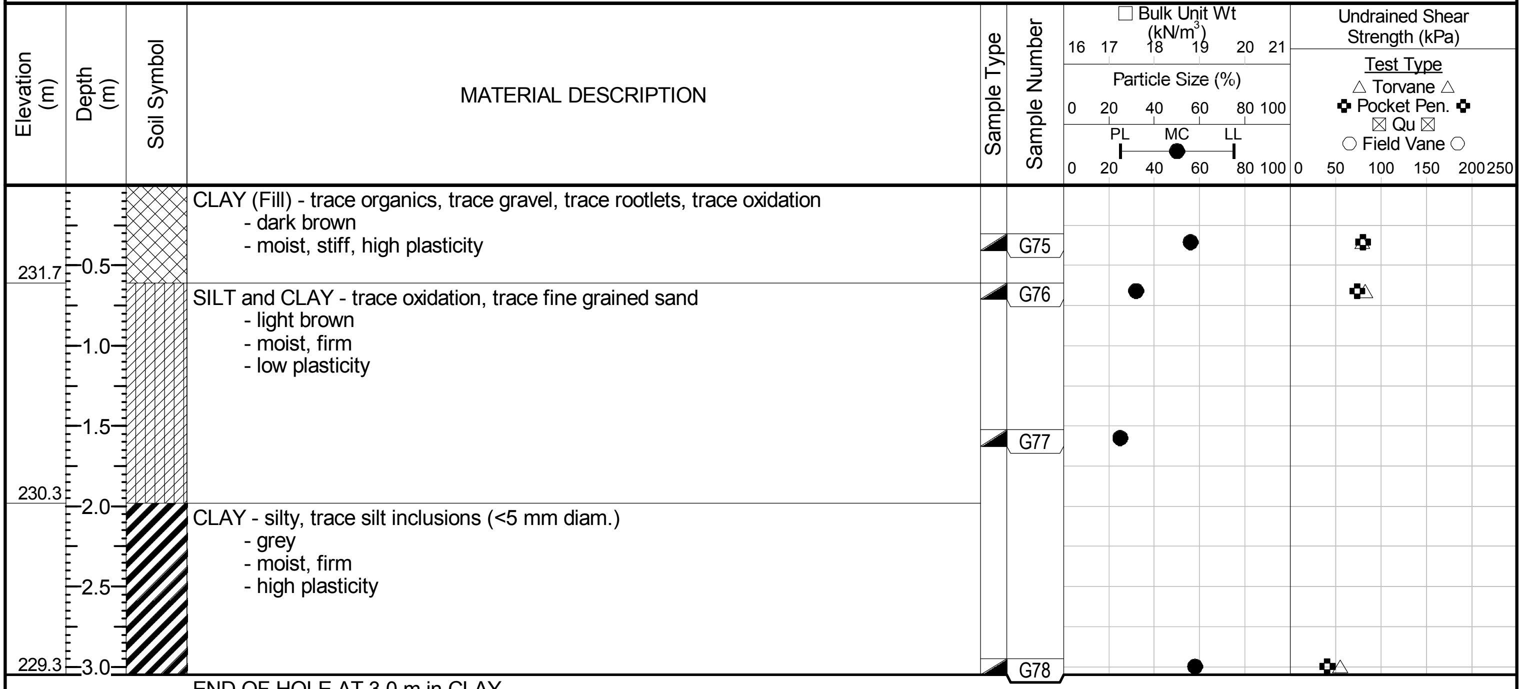
Sub-Surface Log

Test Hole TH13-12

Client: Dillon Consulting Ltd.
Project Number: 0022 010 00
Project Name: City of Winnipeg Resource Recovery Centres
Location: UTM N-5530385, E631107 (Pacific Industrial)
Contractor: Subterranean Ltd.
Ground Elevation: 232.30 m
Method: 508 mm Auger, Soilmec STM-20 Truck Mount
Date Drilled: 29 October 2013

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

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt ▒ Sand ▚ Gravel ▱ Cobbles ■ Boulders



CLAY (Fill) - trace organics, trace gravel, trace rootlets, trace oxidation
- dark brown
- moist, stiff, high plasticity

SILT and CLAY - trace oxidation, trace fine grained sand
- light brown
- moist, firm
- low plasticity

CLAY - silty, trace silt inclusions (<5 mm diam.)
- grey
- moist, firm
- high plasticity

END OF HOLE AT 3.0 m in CLAY
Notes:
1) No seepage or sloughing observed.
2) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana
Reviewed By: Michael Van Helden
Project Engineer: Michael Van Helden

Sub-Surface Log

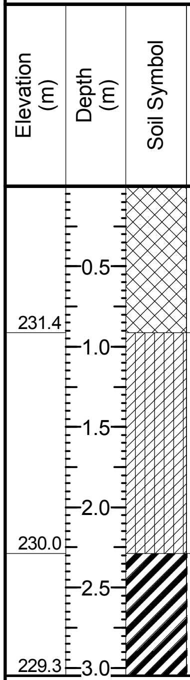
Test Hole TH13-13

1 of 1

Client:	Dillon Consulting Ltd.	**Project Number:**	0022 010 00
Project Name:	City of Winnipeg Resource Recovery Centres	**Location:**	UTM N-5530388, E631137 (Pacific Industrial)
Contractor:	Subterranean Ltd.	**Ground Elevation:**	232.30 m
Method:	508 mm Auger, Soilmec STM-20 Truck Mount	**Date Drilled:**	29 October 2013

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

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt ░ Sand ● Gravel ◌ Cobbles ⬛ Boulders

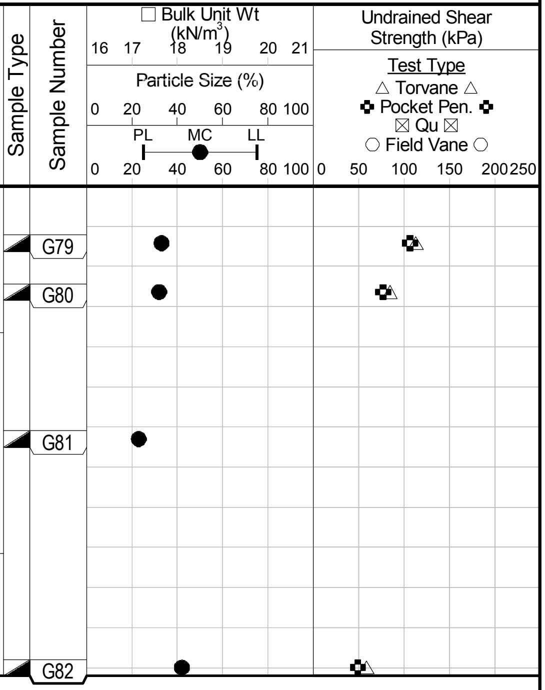


MATERIAL DESCRIPTION

CLAY (Fill) - silty, trace fine grained sand, trace gravel, trace silt inclusions (<5 mm diam.), trace organics, trace oxidation
- dark brown
- moist, stiff
- high plasticity

SILT and CLAY - trace fine grained sand, trace gravel, trace oxidation
- light brown
- moist, firm
- low plasticity

CLAY - silty, trace silt inclusions (<5 mm diam.), trace oxidation
- grey
- moist, stiff
- high plasticity



END OF HOLE AT 3.0 m in CLAY

Notes:

1) No seepage or sloughing observed.

2) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

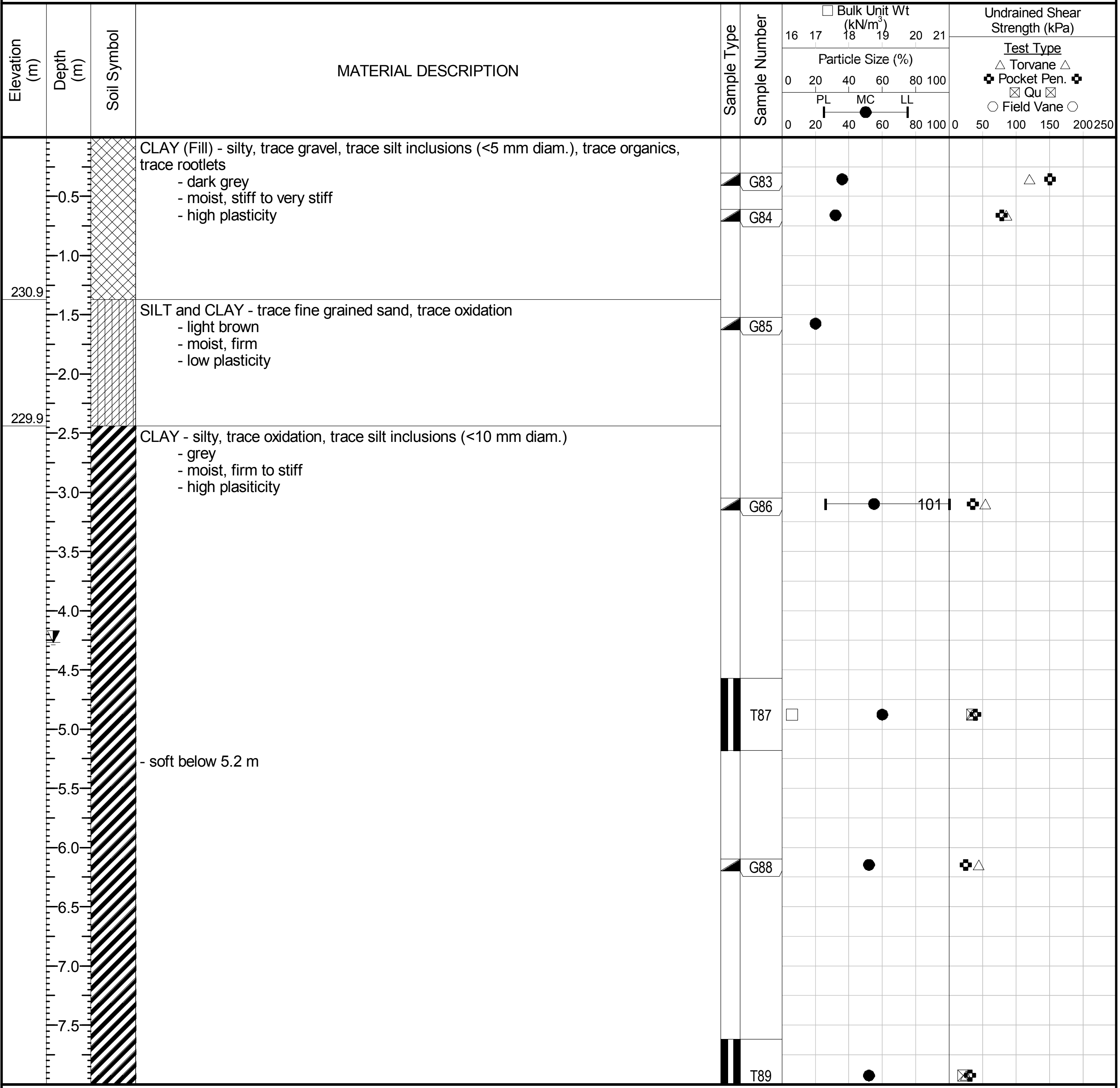
Sub-Surface Log

Test Hole TH13-14

Client: Dillon Consulting Ltd.
Project Number: 0022 010 00
Project Name: City of Winnipeg Resource Recovery Centres
Location: UTM N-5530429, E631169 (Pacific Industrial)
Contractor: Subterranean Ltd.
Ground Elevation: 232.30 m
Method: 508 mm Auger, Soilmec STM-20 Truck Mount
Date Drilled: 29 October 2013

Sample Type: ◢ Grab (G) ▮ Shelby Tube (T) ☒ Split Spoon (SS) ⧗ Split Barrel (SB) ▯ Core (C)

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt ░ Sand ● Gravel ▧ Cobbles ■ Boulders



CLAY (Fill) - silty, trace gravel, trace silt inclusions (<5 mm diam.), trace organics, trace rootlets
- dark grey
- moist, stiff to very stiff
- high plasticity

SILT and CLAY - trace fine grained sand, trace oxidation
- light brown
- moist, firm
- low plasticity

CLAY - silty, trace oxidation, trace silt inclusions (<10 mm diam.)
- grey
- moist, firm to stiff
- high plasiticity

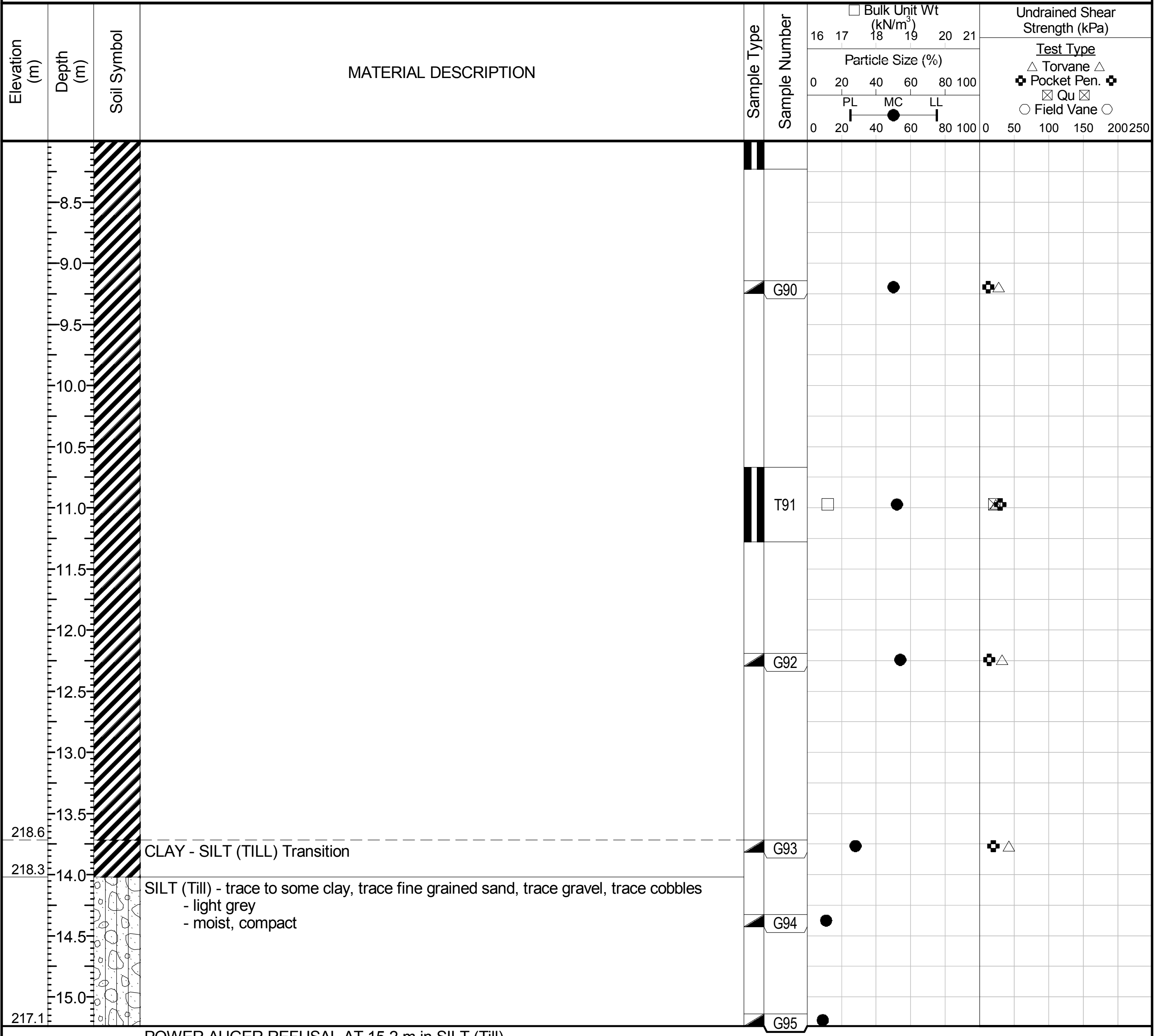
- soft below 5.2 m

Logged By: Beta Taryana **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

Sub-Surface Log

Test Hole TH13-14

2 of 2



CLAY - SILT (TILL) Transition

SILT (Till) - trace to some clay, trace fine grained sand, trace gravel, trace cobbles
- light grey
- moist, compact

POWER AUGER REFUSAL AT 15.2 m in SILT (Till)
Notes:
1) Seepage observed at 14.3 m below surface.
2) No sloughing observed.
3) Water level at 4.3 m below surface upon completion of test hole.
4) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH13-15

1 of 1

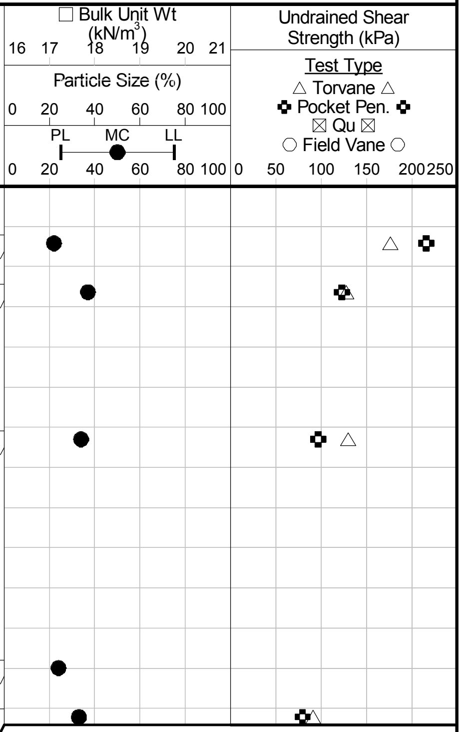
Client: Dillon Consulting Ltd.
Project Number: 0022 010 00
Project Name: City of Winnipeg Resource Recovery Centres
Location: UTM N-5530385, E631171 (Pacific Industrial)
Contractor: Subterranean Ltd.
Ground Elevation: 232.70 m
Method: 508 mm Auger, Soilmec STM-20 Truck Mount
Date Drilled: 29 October 2013

Sample Type: ◪ Grab (G) ◫ Shelby Tube (T) ☒ Split Spoon (SS) ⧗ Split Barrel (SB) ◨ Core (C)

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

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number
	0 – 2.1	▩	CLAY (Fill) - silty, trace fine grained sand, trace gravel, trace silt inclusions (<5 mm diam.), trace rootlets, trace organics, trace debris (nail, glass), trace oxidation - brown - moist, stiff to very stiff - high plasticity - black from 0.6 m to 1.2 m	◪ ◪ ◪	G96 G97 G98
230.6	2.1 – 3.0	▥	SILT and CLAY - trace fine grained sand, trace oxidation - light brown - moist, firm - low plasticity	◪	G99
229.7	3.0 – 3.3	▨	CLAY - silty, trace oxidation, brown, moist, stiff, high plasticity	◪	G100
229.3					

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0



END OF HOLE AT 3.3 m in CLAY
Notes:
1) No seepage or sloughing observed.
2) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

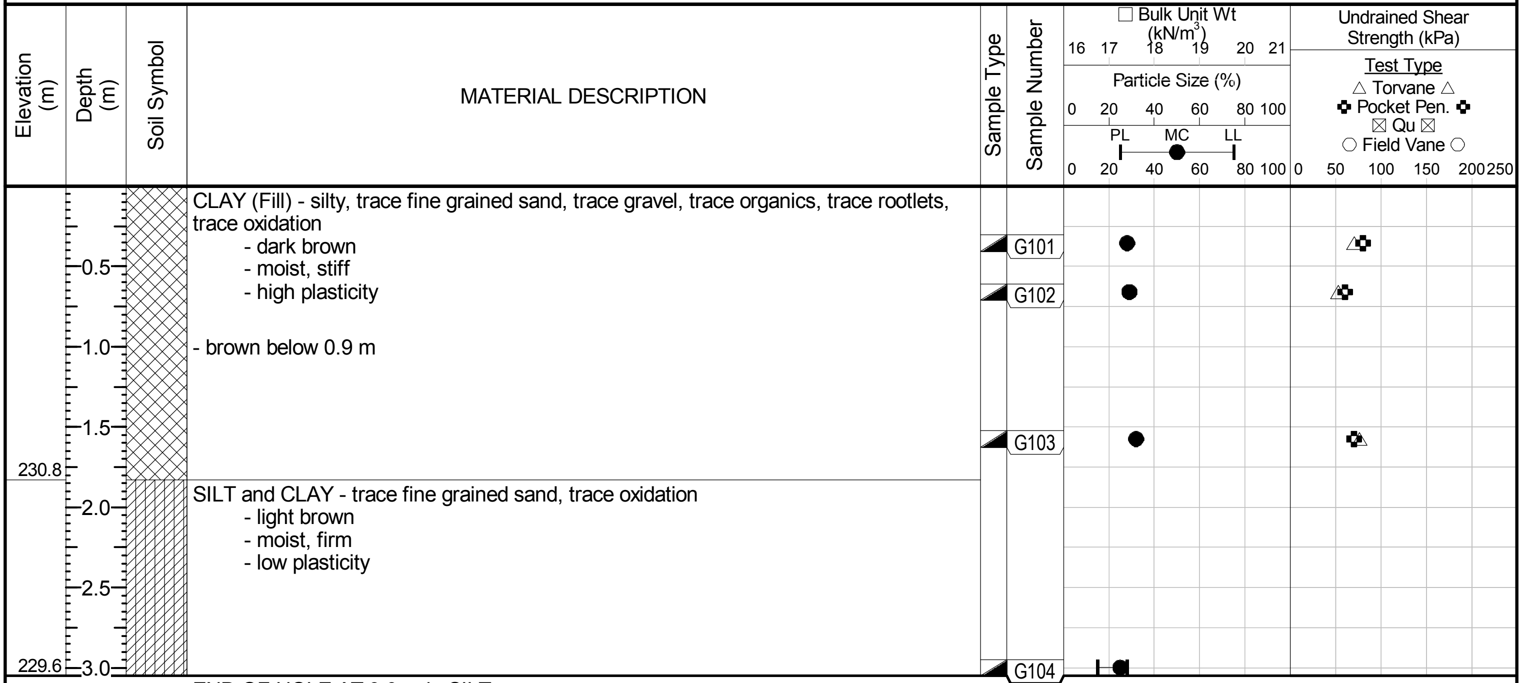
Sub-Surface Log

Test Hole TH13-16

Client: Dillon Consulting Ltd.
Project Name: City of Winnipeg Resource Recovery Centres
Contractor: Subterranean Ltd.
Method: 508 mm Auger, Soilmec STM-20 Truck Mount
Project Number: 0022 010 00
Location: UTM N-5530334, E631130 (Pacific Industrial)
Ground Elevation: 232.60 m
Date Drilled: 29 October 2013

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

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



CLAY (Fill) - silty, trace fine grained sand, trace gravel, trace organics, trace rootlets, trace oxidation
- dark brown
- moist, stiff
- high plasticity

- brown below 0.9 m

SILT and CLAY - trace fine grained sand, trace oxidation
- light brown
- moist, firm
- low plasticity

END OF HOLE AT 3.0 m in SILT
Notes:
1) No seepage or sloughing observed.
2) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana
Reviewed By: Michael Van Helden
Project Engineer: Michael Van Helden

Sub-Surface Log

Test Hole TH13-17

Client: Dillon Consulting Ltd.

Project Name: City of Winnipeg Resource Recovery Centres

Contractor: Subterranean Ltd.

Method: 508 mm Auger, Soilmec STM-20 Truck Mount

Project Number: 0022 010 00

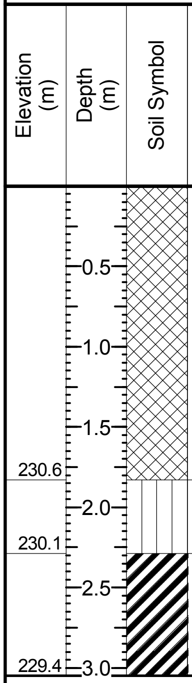
Location: UTM N-5530282, E631140 (Pacific Industrial)

Ground Elevation: 232.40 m

Date Drilled: 29 October 2013

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

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

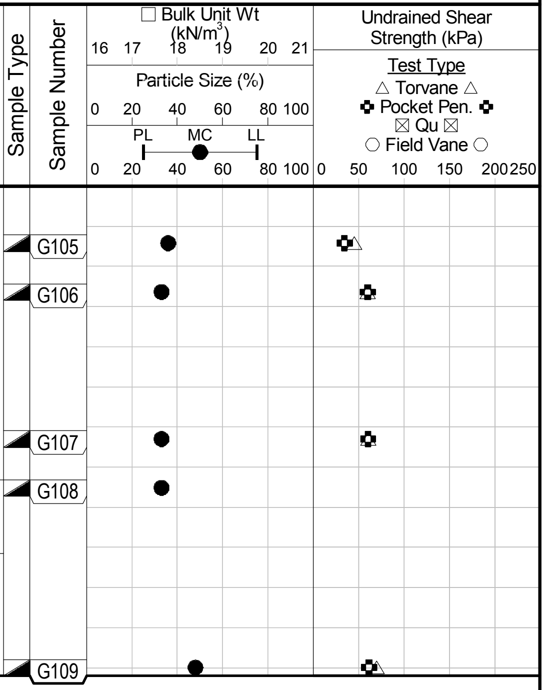


MATERIAL DESCRIPTION

CLAY (Fill) - silty, trace silt inclusions (<10 mm diam.), trace gravel, trace rootlets, trace organics
- dark grey
- moist, firm to stiff
- high plasticity

SILT - trace clay, trace fine grained sand, trace oxidation
- light brown
- moist to wet, soft to firm, low plasticity

CLAY - silty, trace oxidation
- mottled grey and brown
- moist, stiff
- high plasticity



END OF HOLE AT 3.0 m in CLAY
Notes:
1) No seepage or sloughing observed.
2) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana

Reviewed By: Michael Van Helden

Project Engineer: Michael Van Helden

Test Hole TH13-18

1 of 1

Sub-Surface Log

Client: Dillon Consulting Ltd.

Project Number: 0022 010 00

Project Name: City of Winnipeg Resource Recovery Centres

Location: UTM N-5530334, E631158 (Pacific Industrial)

Contractor: Subterranean Ltd.

Ground Elevation: 232.60 m

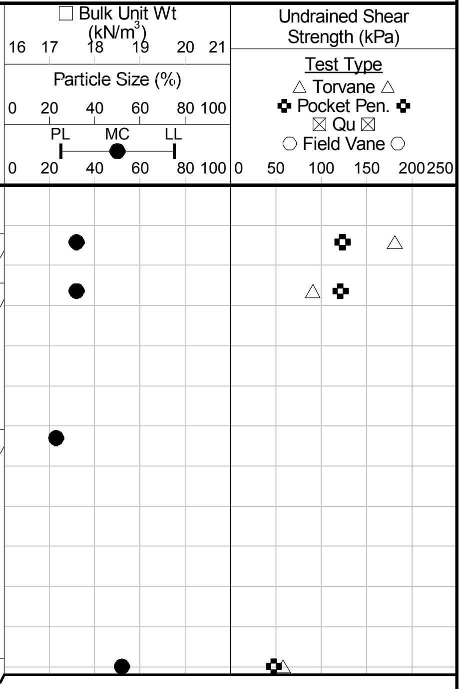
Method: 508 mm Auger, Soilmec STM-20 Truck Mount

Date Drilled: 29 October 2013

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

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt ░ Sand ● Gravel ▱ Cobbles ■ Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number
	0.5 1.0	▩	CLAY (Fill) - silty, trace gravel, trace organics, trace rootlets - black - moist, stiff to very stiff - high plasticity	◩ ◩	G110 G111
231.4	1.5 2.0 2.5	▥	SILT - trace clay, trace fine grained sand - light brown - moist to wet, soft to firm - low plasticity	◩	G112
230.0	3.0	▨	CLAY - silty, trace silt inclusions (<10 mm diam.) - grey - moist, stiff, high plasticity	◩	G113
229.6					



END OF HOLE AT 3.0 m in CLAY

Notes:

1) No seepage or sloughing observed.

2) Test hole backfilled with auger cuttings to surface.

Logged By: Beta Taryana

Reviewed By: Michael Van Helden

Project Engineer: Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH13-19

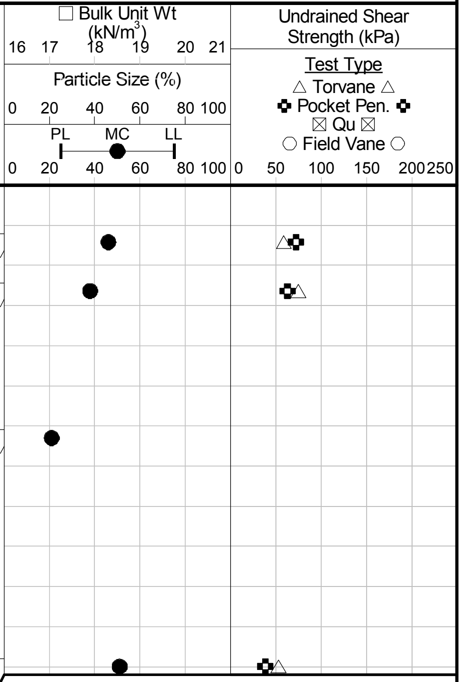
1 of 1

Client: Dillon Consulting Ltd.
Project Number: 0022 010 00
Project Name: City of Winnipeg Resource Recovery Centres
Location: UTM N-5530312, E631112 (Pacific Industrial)
Contractor: Subterranean Ltd.
Ground Elevation: 232.30 m
Method: 508 mm Auger, Soilmec STM-20 Truck Mount
Date Drilled: 29 October 2013

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

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

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number
232.1		CONCRETE - from old structure		
231.5	0.5	CLAY (Fill) - silty, trace gravel, trace organics, trace rootlets - black - moist, firm - high plasticity	◢ ◢	G114 G115
230.6	1.0 1.5	SILT - trace clay, trace fine grained sand, trace oxidation - mottled grey and brown - moist to wet, soft to firm - low plasticity	◢	G116
229.3	2.0 2.5 3.0	CLAY - silty, trace oxidation - grey - moist, firm - high plasticity	◢	G117



END OF HOLE AT 3.0 m in CLAY
Notes:
1) No seepage or sloughing observed.
2) Test hole backfilled with auger cuttings to surface.

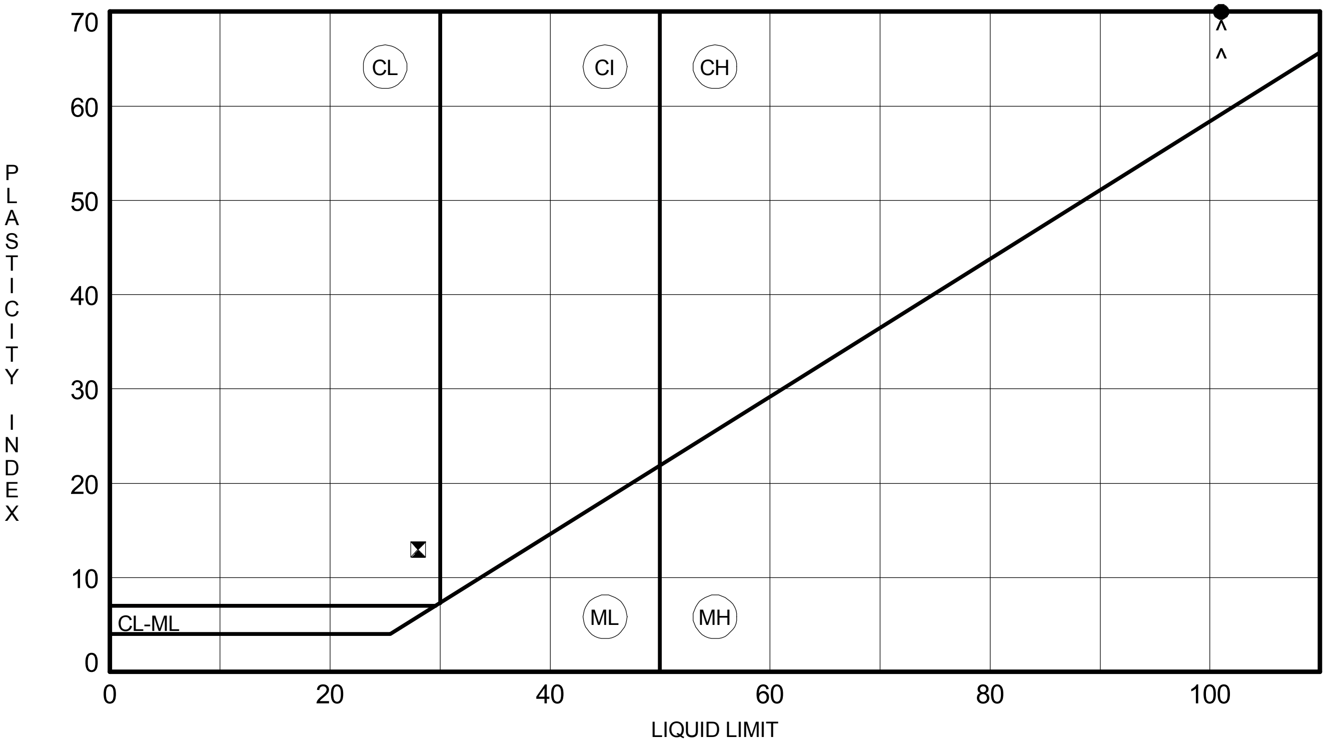
Logged By: Beta Taryana
Reviewed By: Michael Van Helden
Project Engineer: Michael Van Helden

Appendix C

Laboratory Testing Results

A-LINE PLASTICITY CHART

CLIENT Dillon Consulting Ltd.
PROJECT NAME City of Winnipeg Resource Recovery Centres
PROJECT NUMBER 0022 010 00
PROJECT LOCATION



TEST HOLE No.	Sample	Depth (m)	LL	PL	PI	Fines	Classification
● **TH13-14**	**G86**	**3.0 - 3.1**	**101**	**26**	**75**		
⧗ **TH13-16**	**G104**	**2.9 - 3.0**	**28**	**15**	**13**		

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Shelby Tube Visual

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-11
Sample #	T68
Depth (m)	4.6 - 5.2
Sample Date	23-Oct-13
Test Date	13-Nov-13
Technician	Yongnan Sun

Tube Extraction

Recovery (mm) 720

Bottom - 5.2 m **4.6 m - Top**

PP Tv Moisture Visual	Qu γ_{Bulk}			
100 mm	180 mm	110 mm	120 mm	210 mm

Visual Classification

Material	Clay
Composition	silty
trace oxidation	
trace silt inclusions <5mm dia.	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.42
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	41.2

Pocket Penetrometer

Reading	**1**	0.90
	2	0.95
	3	0.80
	Average	0.88
Undrained Shear Strength (kPa)		43.3

Moisture Content

Tare ID	K2
Mass tare (g)	8.3
Mass wet + tare (g)	371.5
Mass dry + tare (g)	226.8
Moisture %	66.2%

Unit Weight

Bulk Weight (g)		1045.90
Length (mm)	**1**	151.15
	2	151.39
	3	151.38
	4	151.42
Average Length (m)		0.151
Diam. (mm)	**1**	72.14
	2	72.20
	3	72.18
	4	72.36
Average Diameter (m)		0.072
Volume (m^3)		6.20E-04
Bulk Unit Weight (kN/m^3)		16.5
Bulk Unit Weight (pcf)		105.3
Dry Unit Weight (kN/m^3)		10.0
Dry Unit Weight (pcf)		63.4

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Unconfined Compressive Strength
ASTM D2166

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-11
Sample #	T68
Depth (m)	4.6 - 5.2
Sample Date	23-Oct-13
Test Date	13-Nov-13
Technician	Youngnan

Unconfined Strength

	kPa	**ksf**
Max q_u	81.0	1.7
Max S_u	40.5	0.8

Specimen Data

Description Clay - silty, trace oxidation, trace silt inclusions <5mm dia., dark grey, moist, firm, high plasticity

Length	151.3	(mm)	**Moisture %**	66%	
Diameter	72.2	(mm)	**Bulk Unit Wt.**	16.5	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	10.0	(kN/m^3)
Initial Area	0.00410	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

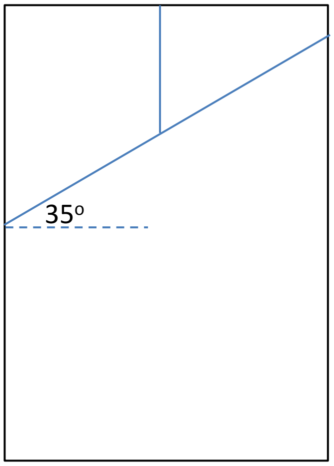
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.42	41.2	0.86
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.90	44.1	0.92
0.95	46.6	0.97
0.80	39.2	0.82
0.88	**43.3**	**0.90**

Failure Geometry

Sketch:



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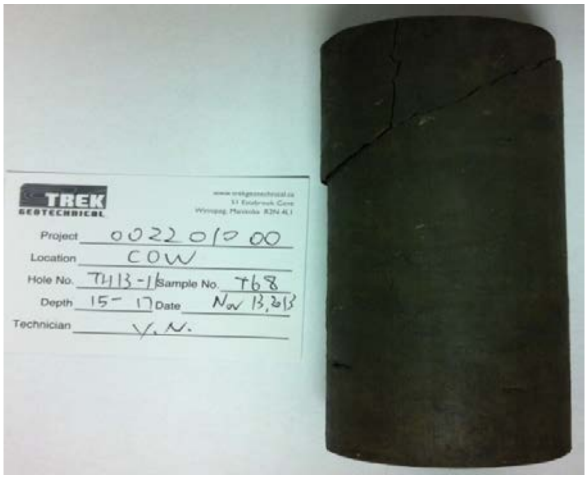


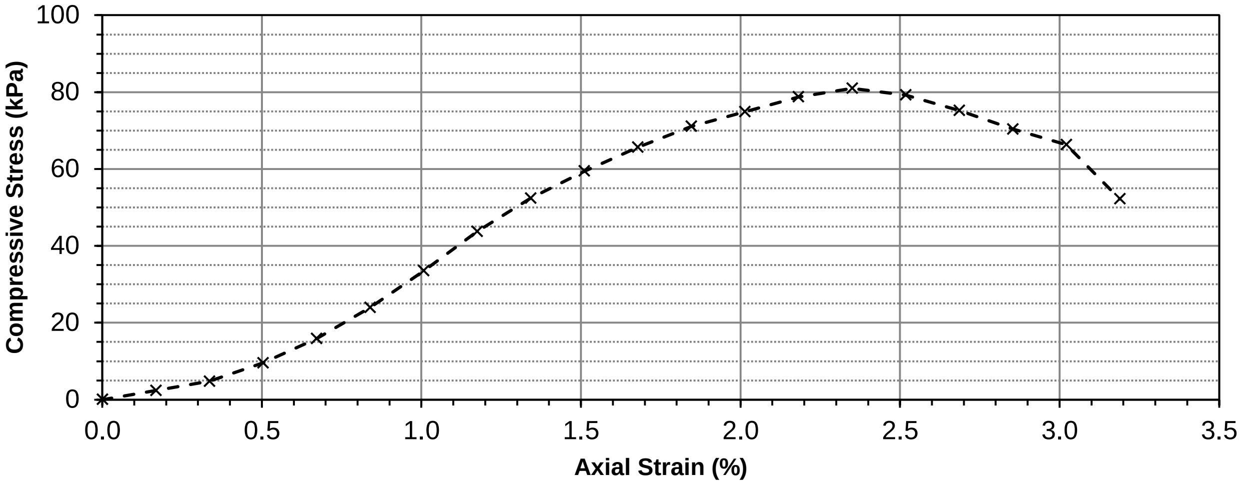
Project 0022 010 00
Location COW
Hole No. TH13-11 Sample No. T68
Depth 15 - 17 Date Nov 13, 2013
Technician Y. N.

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004096	0.0	0.00	0.00
10	3	0.2540	0.17	0.004103	9.8	2.39	1.19
20	6	0.5080	0.34	0.004110	19.6	4.77	2.39
30	12	0.7620	0.50	0.004117	39.3	9.54	4.77
40	20	1.0160	0.67	0.004124	65.5	15.88	7.94
50	30	1.2700	0.84	0.004131	98.9	23.95	11.97
60	42	1.5240	1.01	0.004138	138.5	33.46	16.73
70	55	1.7780	1.17	0.004145	181.4	43.75	21.88
80	66	2.0320	1.34	0.004152	217.6	52.41	26.20
90	75	2.2860	1.51	0.004159	247.3	59.45	29.73
100	83	2.5400	1.68	0.004166	273.7	65.68	32.84
110	90	2.7940	1.85	0.004173	296.7	71.10	35.55
120	95	3.0480	2.01	0.004181	313.2	74.92	37.46
130	100	3.3020	2.18	0.004188	329.7	78.73	39.36
140	103	3.5560	2.35	0.004195	339.8	81.00	40.50
150	101	3.8100	2.52	0.004202	333.1	79.26	39.63
160	96	4.0640	2.69	0.004209	316.5	75.20	37.60
170	90	4.3180	2.85	0.004217	296.7	70.37	35.19
180	85	4.5720	3.02	0.004224	280.2	66.34	33.17
190	67	4.8260	3.19	0.004231	220.9	52.21	26.10

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Shelby Tube Visual

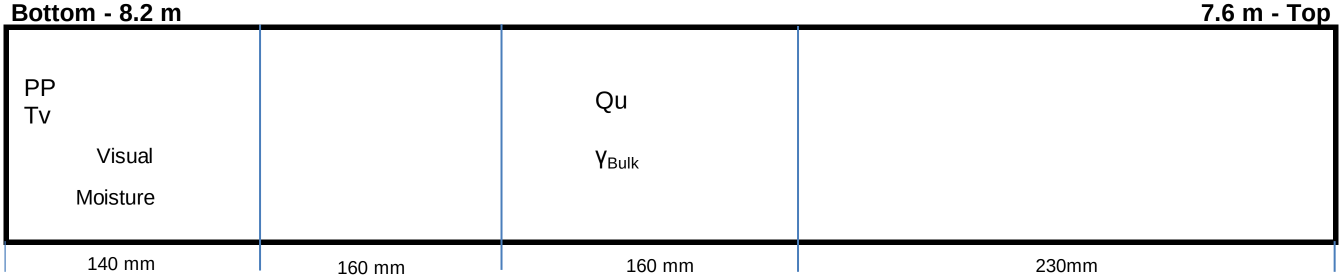
Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-11
Sample #	T70
Depth (m)	7.6 - 8.2
Sample Date	23-Oct-13
Test Date	14-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 690

Bottom - 8.2 m **7.6 m - Top**



Visual Classification

Material	CLAY
Composition	silty

trace silt inclusions <5mm diam.

Color	grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.50
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	49.0

Pocket Penetrometer

Reading	**1**	1.00
	2	0.90
	3	1.10
	Average	1.00
Undrained Shear Strength (kPa)		49.0

Moisture Content

Tare ID	H45
Mass tare (g)	8.4
Mass wet + tare (g)	417.8
Mass dry + tare (g)	270.5
Moisture %	56.2%

Unit Weight

Bulk Weight (g)		1070.90
Length (mm)	**1**	151.27
	2	151.30
	3	151.26
	4	151.72
Average Length (m)		0.151
Diam. (mm)	**1**	72.24
	2	72.15
	3	71.88
	4	72.12
Average Diameter (m)		0.072
Volume (m^3)		6.18E-04
Bulk Unit Weight (kN/m^3)		17.0
Bulk Unit Weight (pcf)		108.2
Dry Unit Weight (kN/m^3)		10.9
Dry Unit Weight (pcf)		69.3

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-11
Sample #	T70
Depth (m)	7.6 - 8.2
Sample Date	23-Oct-13
Test Date	14-Nov-13
Technician	Hachem Ahmed

Unconfined Strength

	kPa	ksf
Max q_u	63.0	1.3
Max S_u	31.5	0.7

Specimen Data

Description CLAY - silty, trace silt inclusions <5mm diam., grey, moist, firm, high plasticity

Length	151.4	(mm)
Diameter	72.1	(mm)
L/D Ratio	2.1	
Initial Area	0.00408	(m^2)
Load Rate	1.00	(%/min)

Moisture %	56%	
Bulk Unit Wt.	17.0	(kN/m^3)
Dry Unit Wt.	10.9	(kN/m^3)
Liquid Limit	-	
Plastic Limit	-	
Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

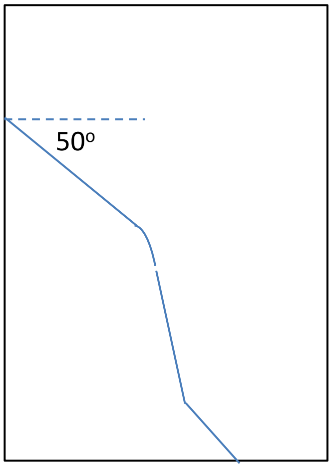
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.50	49.0	1.02
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
1.00	49.1	1.02
0.90	44.1	0.92
1.10	54.0	1.13
1.00	**49.1**	**1.02**

Failure Geometry

Sketch:



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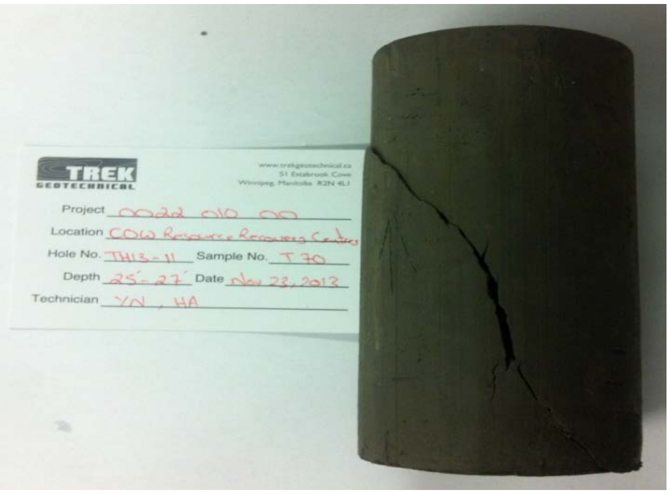


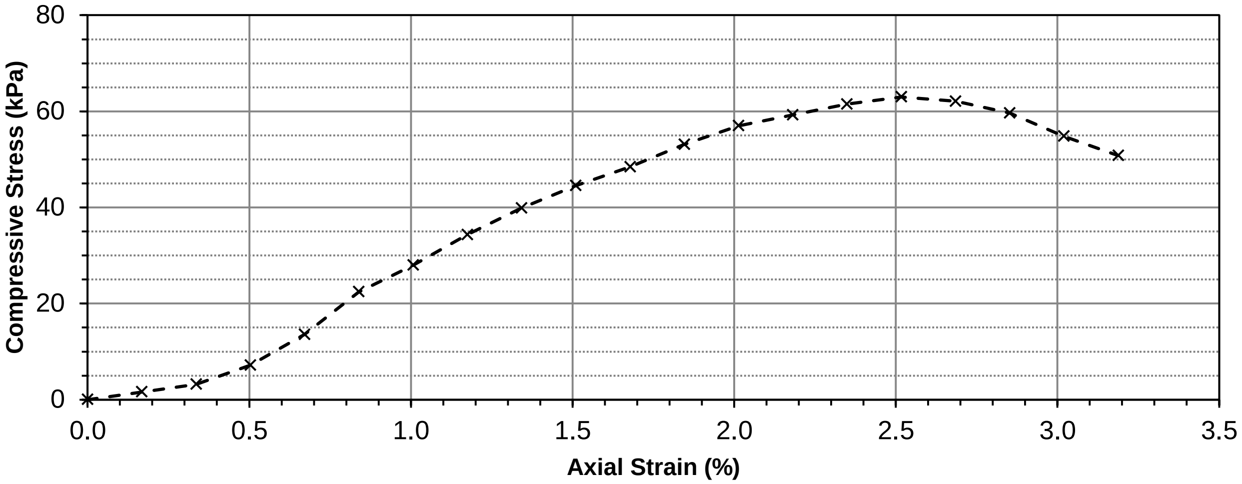
Project 0022 010 00
Location COW Resource Recovery Centres
Hole No. TH13-11 Sample No. T70
Depth 25'-27' Date Nov 23, 2013
Technician YN, HA

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004083	0.0	0.00	0.00
10	2	0.2540	0.17	0.004089	6.5	1.60	0.80
20	4	0.5080	0.34	0.004096	13.1	3.19	1.60
30	9	0.7620	0.50	0.004103	29.4	7.17	3.59
40	17	1.0160	0.67	0.004110	55.7	13.54	6.77
50	28	1.2700	0.84	0.004117	92.3	22.42	11.21
60	35	1.5240	1.01	0.004124	115.4	27.98	13.99
70	43	1.7780	1.17	0.004131	141.8	34.32	17.16
80	50	2.0320	1.34	0.004138	164.9	39.84	19.92
90	56	2.2860	1.51	0.004145	184.6	44.55	22.27
100	61	2.5400	1.68	0.004152	201.1	48.43	24.22
110	67	2.7940	1.85	0.004159	220.9	53.11	26.55
120	72	3.0480	2.01	0.004166	237.4	56.98	28.49
130	75	3.3020	2.18	0.004174	247.3	59.25	29.62
140	78	3.5560	2.35	0.004181	257.2	61.51	30.75
150	80	3.8100	2.52	0.004188	263.8	62.99	31.49
160	79	4.0640	2.68	0.004195	260.4	62.08	31.04
170	76	4.3180	2.85	0.004202	250.6	59.63	29.81
180	70	4.5720	3.02	0.004210	230.8	54.82	27.41
190	65	4.8260	3.19	0.004217	214.3	50.82	25.41

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Shelby Tube Visual

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.
Test Hole	TH13-11
Sample #	T72
Depth (m)	10.7 - 11.3
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Chiran Peiris

Tube Extraction

Recovery (mm) 690

Bottom - 11.3 m				**10.7 m - Top**
PP Tv	Moisture Visual	Qu γ_{Bulk}		
50 mm	100 mm	200 mm	200 mm	150 mm

Visual Classification

Material	Clay
Composition	silty
trace silt inclusions <10mm dia.	
trace gravel inclusions (< 1%)	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.19
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	18.6

Pocket Penetrometer

Reading	**1**	0.60
	2	0.60
	3	0.50
	Average	0.57
Undrained Shear Strength (kPa)		27.8

Moisture Content

Tare ID	F8
Mass tare (g)	8.5
Mass wet + tare (g)	394.3
Mass dry + tare (g)	261.7
Moisture %	52.4%

Unit Weight

Bulk Weight (g)		1039.90
Length (mm)	**1**	151.64
	2	151.54
	3	151.26
	4	151.29
Average Length (m)		0.151
Diam. (mm)	**1**	72.80
	2	72.51
	3	72.47
	4	72.37
Average Diameter (m)		0.073
Volume (m^3)		6.26E-04
Bulk Unit Weight (kN/m^3)		16.3
Bulk Unit Weight (pcf)		103.7
Dry Unit Weight (kN/m^3)		10.7
Dry Unit Weight (pcf)		68.1

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-11
Sample #	T72
Depth (m)	10.7 - 11.3
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Chiran Peiris

Unconfined Strength

	kPa	ksf
Max q_u	29.6	0.6
Max S_u	14.8	0.3

Specimen Data

Description Clay - silty, trace silt inclusions <10mm dia., trace gravel inclusions (< 1%), dark grey, moist, firm, high plasticity

Length	151.4	(mm)	**Moisture %**	52%	
Diameter	72.5	(mm)	**Bulk Unit Wt.**	16.3	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	10.7	(kN/m^3)
Initial Area	0.00413	(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**
0.19	18.6	0.39
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.60	29.4	0.61
0.60	29.4	0.61
0.50	24.5	0.51
0.57	**27.8**	**0.58**

Failure Geometry

Sketch:

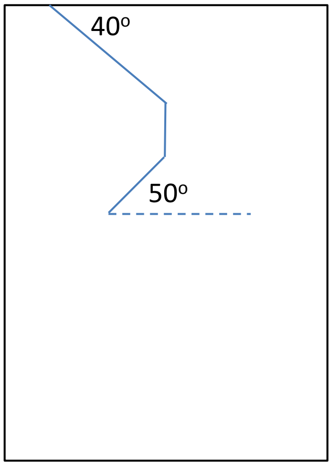


Photo:

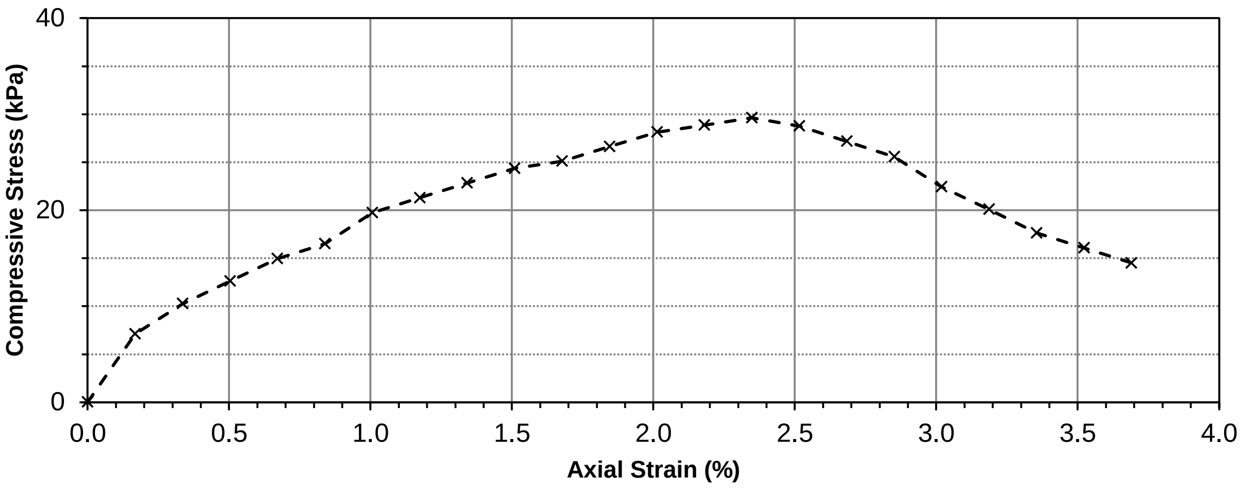
Project 0022 010 00
Location COW RRC
Hole No. TH13-11 Sample No. T-72
Depth 35-37' Date oct-23-13
Technician YN. CP.

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004133	0.0	0.00	0.00
10	9	0.2540	0.17	0.004139	29.4	7.11	3.56
20	13	0.5080	0.34	0.004146	42.5	10.26	5.13
30	16	0.7620	0.50	0.004153	52.4	12.61	6.30
40	19	1.0160	0.67	0.004160	62.2	14.95	7.48
50	21	1.2700	0.84	0.004167	68.8	16.50	8.25
60	25	1.5240	1.01	0.004175	82.4	19.74	9.87
70	27	1.7780	1.17	0.004182	89.0	21.29	10.64
80	29	2.0320	1.34	0.004189	95.6	22.82	11.41
90	31	2.2860	1.51	0.004196	102.2	24.36	12.18
100	32	2.5400	1.68	0.004203	105.5	25.10	12.55
110	34	2.7940	1.85	0.004210	112.1	26.62	13.31
120	36	3.0480	2.01	0.004217	118.7	28.14	14.07
130	37	3.3020	2.18	0.004225	122.0	28.87	14.44
140	38	3.5560	2.35	0.004232	125.3	29.61	14.80
150	37	3.8100	2.52	0.004239	122.0	28.77	14.39
160	35	4.0640	2.68	0.004246	115.4	27.17	13.59
170	33	4.3180	2.85	0.004254	108.8	25.58	12.79
180	29	4.5720	3.02	0.004261	95.6	22.43	11.22
190	26	4.8260	3.19	0.004269	85.7	20.08	10.04
200	23	5.0800	3.35	0.004276	75.3	17.62	8.81
210	21	5.3340	3.52	0.004283	68.8	16.06	8.03
220	19	5.5880	3.69	0.004291	62.2	14.50	7.25

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Shelby Tube Visual

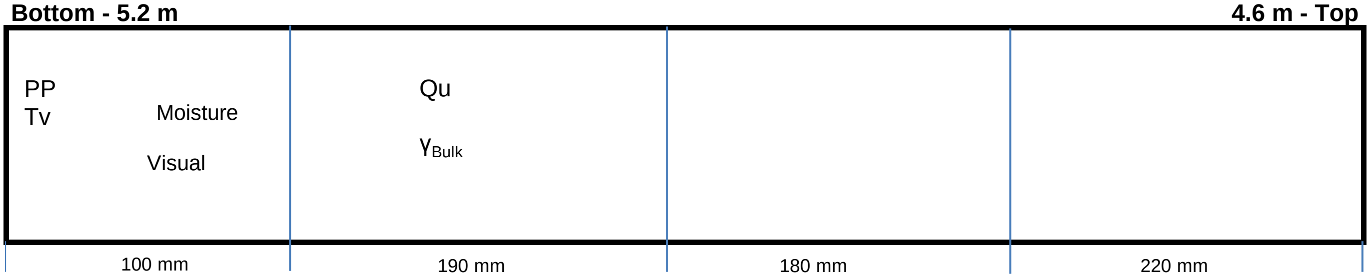
Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-14
Sample #	T87
Depth (m)	4.6 - 5.2
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Yongnan Sun

Tube Extraction

Recovery (mm) 690

Bottom - 5.2 m **4.6 m - Top**



Visual Classification

Material	Clay
Composition	silty
trace silt inclusions <10mm dia.	
trace oxidation	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.35
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	34.3

Pocket Penetrometer

Reading	**1**	0.70
	2	0.90
	3	0.80
	Average	0.80
Undrained Shear Strength (kPa)		39.2

Moisture Content

Tare ID	H55
Mass tare (g)	8.4
Mass wet + tare (g)	297.4
Mass dry + tare (g)	189.4
Moisture %	59.7%

Unit Weight

Bulk Weight (g)		923.20
Length (mm)	**1**	137.77
	2	137.24
	3	137.24
	4	137.36
Average Length (m)		0.137
Diam. (mm)	**1**	71.86
	2	71.75
	3	71.85
	4	71.95
Average Diameter (m)		0.072
Volume (m^3)		5.57E-04
Bulk Unit Weight (kN/m^3)		16.3
Bulk Unit Weight (pcf)		103.4
Dry Unit Weight (kN/m^3)		10.2
Dry Unit Weight (pcf)		64.8

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-14
Sample #	T87
Depth (m)	4.6 - 5.2
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Yongnan Sun

Unconfined Strength

	kPa	ksf
Max q_u	68.1	1.4
Max S_u	34.1	0.7

Specimen Data

Description Clay - silty, trace silt inclusions <10mm dia., trace oxidation, dark grey, moist, firm, high plasticity

Length	137.4	(mm)
Diameter	71.9	(mm)
L/D Ratio	1.9	
Initial Area	0.00405	(m^2)
Load Rate	1.00	(%/min)

Moisture %	60%	
Bulk Unit Wt.	16.3	(kN/m^3)
Dry Unit Wt.	10.2	(kN/m^3)
Liquid Limit	-	
Plastic Limit	-	
Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

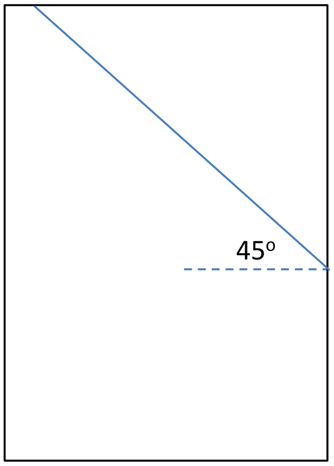
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.35	34.3	0.72
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.70	34.3	0.72
0.90	44.1	0.92
0.80	39.2	0.82
0.80	**39.2**	**0.82**

Failure Geometry

Sketch:



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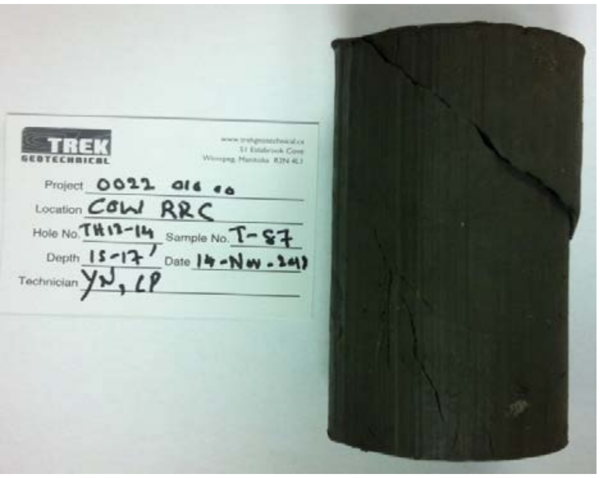
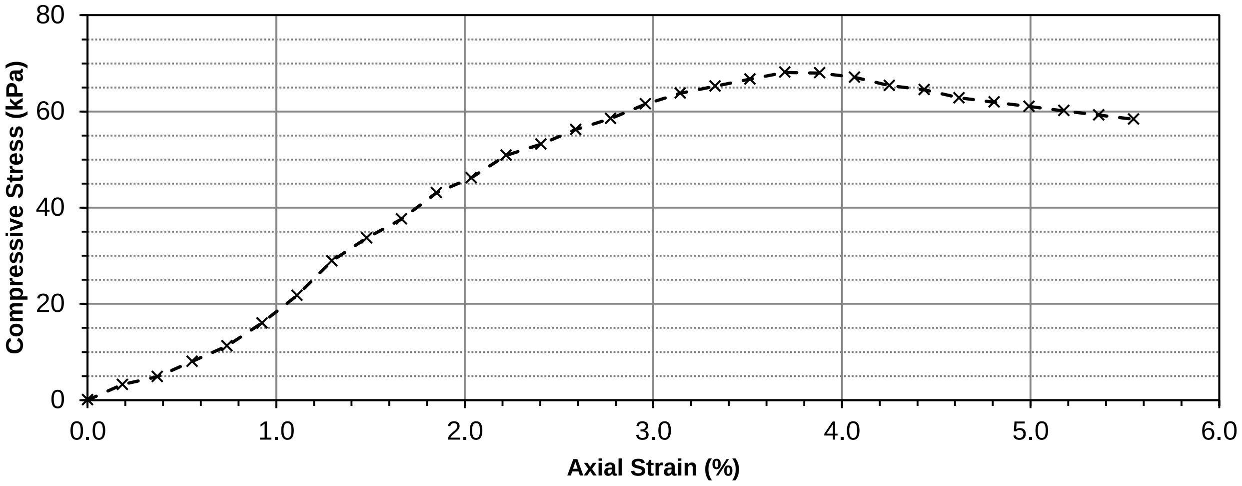


Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004055	0.0	0.00	0.00
10	4	0.2540	0.18	0.004062	13.1	3.22	1.61
20	6	0.5080	0.37	0.004070	19.6	4.82	2.41
30	10	0.7620	0.55	0.004077	32.7	8.02	4.01
40	14	1.0160	0.74	0.004085	45.8	11.21	5.61
50	20	1.2700	0.92	0.004093	65.5	16.00	8.00
60	27	1.5240	1.11	0.004100	89.0	21.71	10.85
70	36	1.7780	1.29	0.004108	118.7	28.89	14.44
80	42	2.0320	1.48	0.004116	138.5	33.65	16.82
90	47	2.2860	1.66	0.004123	155.0	37.58	18.79
100	54	2.5400	1.85	0.004131	178.0	43.09	21.55
110	58	2.7940	2.03	0.004139	191.2	46.20	23.10
120	64	3.0480	2.22	0.004147	211.0	50.89	25.44
130	67	3.3020	2.40	0.004155	220.9	53.17	26.58
140	71	3.5560	2.59	0.004163	234.1	56.24	28.12
150	74	3.8100	2.77	0.004170	244.0	58.50	29.25
160	78	4.0640	2.96	0.004178	257.2	61.54	30.77
170	81	4.3180	3.14	0.004186	267.1	63.79	31.90
180	83	4.5720	3.33	0.004194	273.7	65.24	32.62
190	85	4.8260	3.51	0.004202	280.2	66.68	33.34
200	87	5.0800	3.70	0.004211	286.8	68.12	34.06
210	87	5.3340	3.88	0.004219	286.8	67.99	33.99
220	86	5.5880	4.07	0.004227	283.5	67.08	33.54
230	84	5.8420	4.25	0.004235	276.9	65.40	32.70

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Data (cont'd)

Elapsed Time (s)	Axial Disp. (mm)	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	83	6.0960	4.4366	0.004243	273.7	64.49	32.25
250	81	6.3500	4.62	0.004251	267.1	62.82	31.41
260	80	6.6040	4.81	0.004260	263.8	61.93	30.96
270	79	6.8580	4.99	0.004268	260.4	61.02	30.51
280	78	7.1120	5.18	0.004276	257.2	60.14	30.07
290	77	7.3660	5.36	0.004285	253.9	59.25	29.63
300	76	7.6200	5.55	0.004293	250.6	58.37	29.18

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Shelby Tube Visual

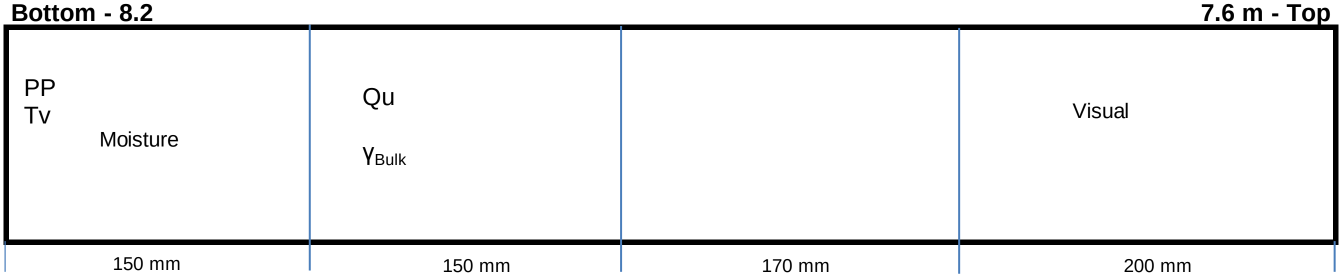
Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave

Test Hole	TH13-14
Sample #	T89
Depth (m)	7.6 - 8.2
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Chiran Peiirs

Tube Extraction

Recovery (mm) 670

Bottom - 8.2 **7.6 m - Top**



Visual Classification

Material	Clay
Composition	silty

trace gravel
trace silt inclusions <10mm diam

Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.26
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	25.5

Pocket Penetrometer

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

Moisture Content

Tare ID	F72
Mass tare (g)	8.5
Mass wet + tare (g)	311.8
Mass dry + tare (g)	207.7
Moisture %	52.3%

Unit Weight

Bulk Weight (g)		1043.40
Length (mm)	**1**	144.34
	2	144.36
	3	144.65
	4	144.59
Average Length (m)		0.144
Diam. (mm)	**1**	72.34
	2	72.52
	3	72.55
	4	72.47
Average Diameter (m)		0.072
Volume (m^3)		5.96E-04
Bulk Unit Weight (kN/m^3)		17.2
Bulk Unit Weight (pcf)		109.3
Dry Unit Weight (kN/m^3)		11.3
Dry Unit Weight (pcf)		71.8

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Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave

Test Hole TH13-14
Sample # T89
Depth (m) 7.6 - 8.2
Sample Date 23-Oct-13
Test Date 7-Nov-13
Technician Chiran Peiris

Unconfined Strength

	kPa	ksf
Max q_u	65.7	1.4
Max S_u	32.8	0.7

Specimen Data

Description Clay - silty, trace gravel, trace silt inclusions <10mm diam, dark grey, moist, firm, high plasticity

Length	144.5	(mm)	**Moisture %**	52%	
Diameter	72.5	(mm)	**Bulk Unit Wt.**	17.2	(kN/m^3)
L/D Ratio	2.0		**Dry Unit Wt.**	11.3	(kN/m^3)
Initial Area	0.00412	(m^2)	**Liquid Limit**	-	
Load Rate	1.00	(%/min)	**Plastic Limit**	-	
			Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

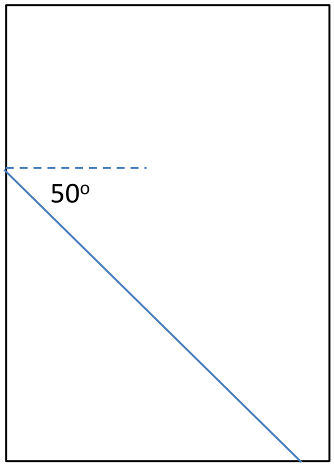
Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.26	25.5	0.53
Vane Size		
m		

Pocket Penetrometer

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

Failure Geometry

Sketch:



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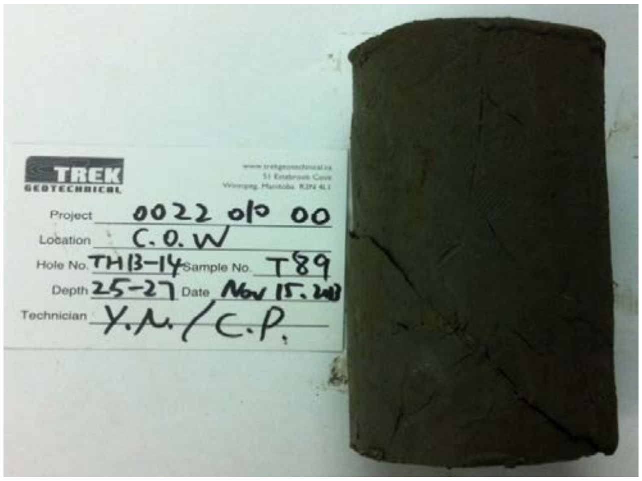


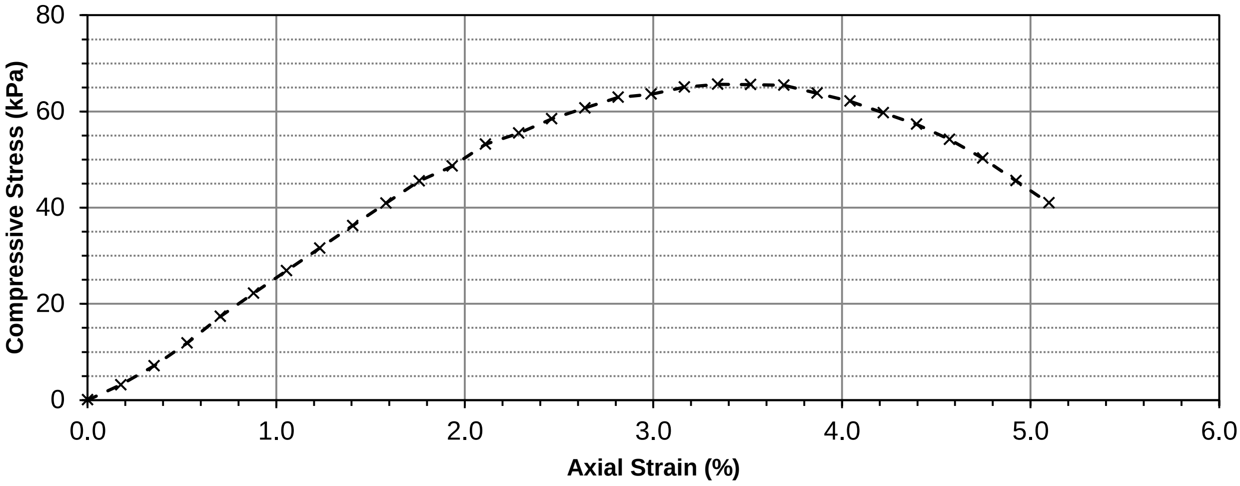
Project 0022 010 00
Location C.O.W
Hole No. TH13-14 Sample No. T89
Depth 25-27 Date Nov 15. 2013
Technician Y.M. / C.P.

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004125	0.0	0.00	0.00
10	4	0.2540	0.18	0.004132	13.1	3.16	1.58
20	9	0.5080	0.35	0.004139	29.4	7.11	3.56
30	15	0.7620	0.53	0.004147	49.1	11.84	5.92
40	22	1.0160	0.70	0.004154	72.1	17.35	8.67
50	28	1.2700	0.88	0.004161	92.3	22.18	11.09
60	34	1.5240	1.05	0.004169	112.1	26.89	13.44
70	40	1.7780	1.23	0.004176	131.9	31.58	15.79
80	46	2.0320	1.41	0.004184	151.7	36.26	18.13
90	52	2.2860	1.58	0.004191	171.4	40.90	20.45
100	58	2.5400	1.76	0.004199	191.2	45.55	22.77
110	62	2.7940	1.93	0.004206	204.4	48.59	24.30
120	68	3.0480	2.11	0.004214	224.2	53.20	26.60
130	71	3.3020	2.29	0.004221	234.1	55.46	27.73
140	75	3.5560	2.46	0.004229	247.3	58.47	29.24
150	78	3.8100	2.64	0.004237	257.2	60.70	30.35
160	81	4.0640	2.81	0.004244	267.1	62.93	31.46
170	82	4.3180	2.99	0.004252	270.4	63.59	31.79
180	84	4.5720	3.16	0.004260	276.9	65.02	32.51
190	85	4.8260	3.34	0.004267	280.2	65.67	32.83
200	85	5.0800	3.52	0.004275	280.2	65.55	32.78
210	85	5.3340	3.69	0.004283	280.2	65.43	32.72
220	83	5.5880	3.87	0.004291	273.7	63.78	31.89
230	81	5.8420	4.04	0.004299	267.1	62.13	31.06

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave

Unconfined Compression Test Data (cont'd)

Elapsed Time (s)	Axial Disp. (mm)	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	78	6.0960	4.2191	0.004307	257.2	59.71	29.86
250	75	6.3500	4.39	0.004314	247.3	57.31	28.66
260	71	6.6040	4.57	0.004322	234.1	54.16	27.08
270	66	6.8580	4.75	0.004330	217.6	50.25	25.13
280	60	7.1120	4.92	0.004338	197.8	45.60	22.80
290	54	7.3660	5.10	0.004346	178.0	40.96	20.48

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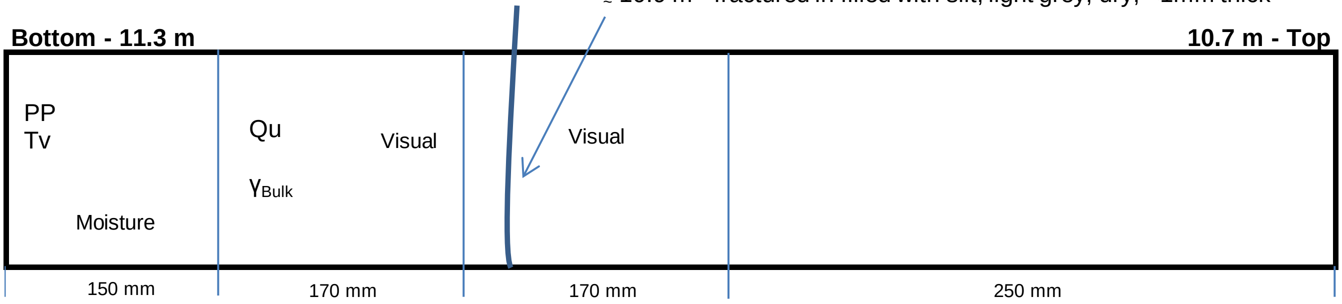
Shelby Tube Visual

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-14
Sample #	T91
Depth (m)	10.7-11.3
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Yongnan Sun/Chiran Peiris

Tube Extraction

Recovery (mm) 740



Visual Classification

Material	Clay
Composition	silty

trace silt inclusions <10mm diam.

trace oxidation

Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	stratified
Gradation	-

Torvane

Reading	0.24
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	23.5

Pocket Penetrometer

Reading	**1**	0.60
	2	0.60
	3	0.60
	Average	0.60
Undrained Shear Strength (kPa)		29.4

Moisture Content

Tare ID	K33
Mass tare (g)	8.5
Mass wet + tare (g)	359.6
Mass dry + tare (g)	240.2
Moisture %	51.5%

Unit Weight

Bulk Weight (g)		1050.20
Length (mm)	**1**	154.34
	2	154.37
	3	154.38
	4	154.34
Average Length (m)		0.154
Diam. (mm)	**1**	71.32
	2	71.68
	3	71.40
	4	71.53
Average Diameter (m)		0.071
Volume (m^3)		6.19E-04
Bulk Unit Weight (kN/m^3)		16.6
Bulk Unit Weight (pcf)		105.8
Dry Unit Weight (kN/m^3)		11.0
Dry Unit Weight (pcf)		69.8

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 010 00
Client	Dillon Consulting Ltd.
Project	City of Winnipeg Resource Recovery Centres - Pacific Ave.

Test Hole	TH13-14
Sample #	T91
Depth (m)	10.7-11.3
Sample Date	23-Oct-13
Test Date	15-Nov-13
Technician	Yongnan Sun/Chiran Peiris

Unconfined Strength

	kPa	ksf
Max q_u	42.6	0.9
Max S_u	21.3	0.4

Specimen Data

Description Clay - silty, trace silt inclusions <10mm diam., trace oxidation, dark grey, moist, firm, high plasticity, stratified

Length	154.4	(mm)	**Moisture %**	52%	
Diameter	71.5	(mm)	**Bulk Unit Wt.**	16.6	(kN/m^3)
L/D Ratio	2.2		**Dry Unit Wt.**	11.0	(kN/m^3)
Initial Area	0.00401	(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**
0.24	23.5	0.49
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.60	29.4	0.61
0.60	29.4	0.61
0.60	29.4	0.61
0.60	**29.4**	**0.61**

Failure Geometry

Sketch:

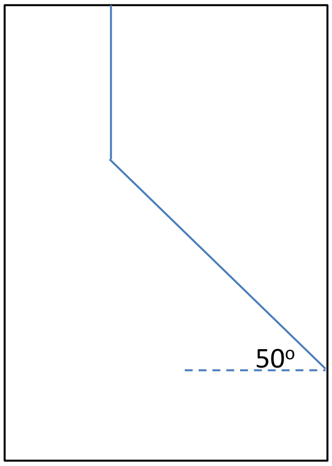

Photo:

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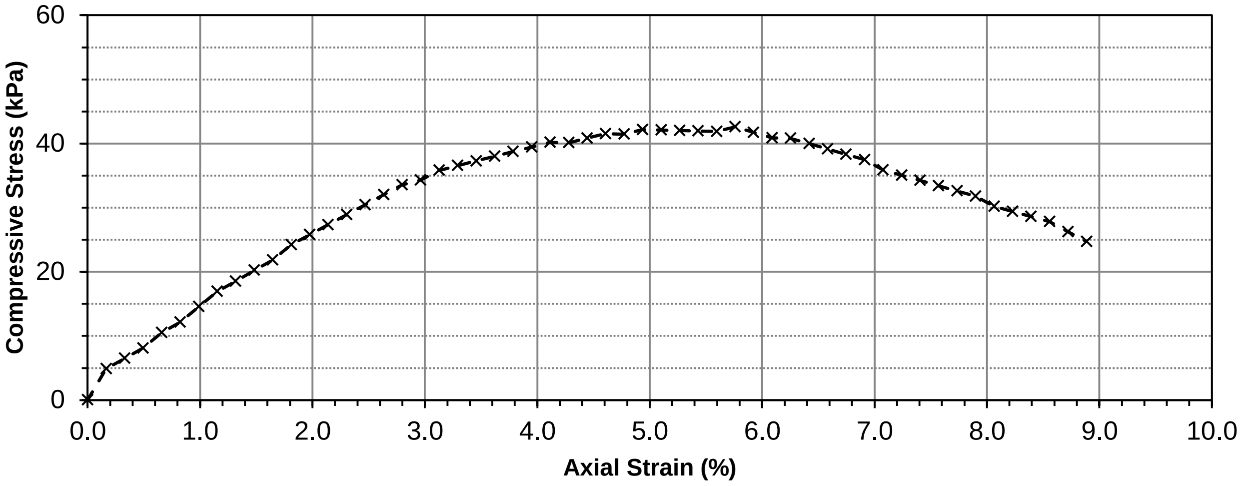
Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	0	0.0000	0.00	0.004013	0.0	0.00	0.00
10	6	0.2540	0.16	0.004020	19.6	4.88	2.44
20	8	0.5080	0.33	0.004026	26.2	6.50	3.25
30	10	0.7620	0.49	0.004033	32.7	8.11	4.05
40	13	1.0160	0.66	0.004040	42.5	10.53	5.26
50	15	1.2700	0.82	0.004046	49.1	12.13	6.07
60	18	1.5240	0.99	0.004053	58.9	14.54	7.27
70	21	1.7780	1.15	0.004060	68.8	16.94	8.47
80	23	2.0320	1.32	0.004067	75.3	18.53	9.26
90	25	2.2860	1.48	0.004074	82.4	20.23	10.12
100	27	2.5400	1.65	0.004080	89.0	21.81	10.91
110	30	2.7940	1.81	0.004087	98.9	24.20	12.10
120	32	3.0480	1.97	0.004094	105.5	25.77	12.89
130	34	3.3020	2.14	0.004101	112.1	27.33	13.67
140	36	3.5560	2.30	0.004108	118.7	28.89	14.45
150	38	3.8100	2.47	0.004115	125.3	30.45	15.23
160	40	4.0640	2.63	0.004122	131.9	32.00	16.00
170	42	4.3180	2.80	0.004129	138.5	33.54	16.77
180	43	4.5720	2.96	0.004136	141.8	34.28	17.14
190	45	4.8260	3.13	0.004143	148.3	35.81	17.90
200	46	5.0800	3.29	0.004150	151.7	36.55	18.28
210	47	5.3340	3.46	0.004157	155.0	37.28	18.64
220	48	5.5880	3.62	0.004164	158.3	38.01	19.00
230	49	5.8420	3.78	0.004171	161.6	38.73	19.37

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 010 00
Client Dillon Consulting Ltd.
Project City of Winnipeg Resource Recovery Centres - Pacific Ave.

Unconfined Compression Test Data (cont'd)

Elapsed Time (s)	Axial Disp. (mm)	Deflection (mm)	Axial Strain (%)	Corrected Area (m^2)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	50	6.0960	3.9493	0.004178	164.9	39.46	19.73
250	51	6.3500	4.11	0.004185	168.1	40.17	20.09
260	51	6.6040	4.28	0.004193	168.1	40.11	20.05
270	52	6.8580	4.44	0.004200	171.4	40.82	20.41
280	53	7.1120	4.61	0.004207	174.7	41.53	20.77
290	53	7.3660	4.77	0.004214	174.7	41.46	20.73
300	54	7.6200	4.94	0.004222	178.0	42.17	21.08
310	54	7.8740	5.10	0.004229	178.0	42.10	21.05
320	54	8.1280	5.27	0.004236	178.0	42.02	21.01
330	54	8.3820	5.43	0.004244	178.0	41.95	20.97
340	54	8.6360	5.59	0.004251	178.0	41.88	20.94
350	55	8.8900	5.76	0.004258	181.4	42.59	21.29
360	54	9.1440	5.92	0.004266	178.0	41.73	20.87
370	53	9.3980	6.09	0.004273	174.7	40.89	20.44
380	53	9.6520	6.25	0.004281	174.7	40.82	20.41
390	52	9.9060	6.42	0.004288	171.4	39.98	19.99
400	51	10.1600	6.58	0.004296	168.1	39.14	19.57
410	50	10.4140	6.75	0.004304	164.9	38.31	19.15
420	49	10.6680	6.91	0.004311	161.6	37.47	18.74
430	47	10.9220	7.08	0.004319	155.0	35.88	17.94
440	46	11.1760	7.24	0.004326	151.7	35.06	17.53
450	45	11.4300	7.40	0.004334	148.3	34.23	17.11
460	44	11.6840	7.57	0.004342	145.1	33.41	16.70
470	43	11.9380	7.73	0.004350	141.8	32.59	16.30
480	42	12.1920	7.90	0.004357	138.5	31.78	15.89
490	40	12.4460	8.06	0.004365	131.9	30.21	15.11
500	39	12.7000	8.23	0.004373	128.6	29.41	14.70
510	38	12.9540	8.39	0.004381	125.3	28.60	14.30
520	37	13.2080	8.56	0.004389	122.0	27.79	13.90
530	35	13.4620	8.72	0.004397	115.4	26.24	13.12
540	33	13.7160	8.89	0.004405	108.8	24.70	12.35