APPENDIX ‘A’

GEOTECHNICAL INVESTIGATION

Dillon Consulting Ltd.

Dugald Drain Crossing Replacement at Happyland Park

Prepared for:

Dillon Consulting Ltd.
1558 Willson Place
Winnipeg, MB R3T 0Y4

Project Number: 0022 011 00

Date: December 9, 2013

TREK
GEOTECHNICAL
Quality Engineering | Valued Relationships

December 9, 2013

Our File No. 0022 011 00

Mr. Mark Doucet
Dillon Consulting Ltd.
1558 Willson Place
Winnipeg, MB R4T 0Y4

RE: Geotechnical Investigation for Dugald Drain Crossing Replacement at Happyland Park

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

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Ken Skaftfeld, P.Eng.
Geotechnical Engineer
Tel: 204.975.9433 ext. 106

JB: jh
Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	KMS	Dec. 9, 2013	Final Report

Authorization Signatures

Prepared By:

Ken Skaftfeld, P.Eng.
Senior Geotechnical Engineer

PROVINCE OF MANITOBA
KMS.
K. M.
SKAFTFELD
Dec 9/13.
REGISTERED ENGINEER

Reviewed By:

James Blatz, Ph.D., P.Eng.
Geotechnical Engineer

APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Dec 9/13.

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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 Dugald Drain crossing replacement at the Happyland Park in Winnipeg, Manitoba. A site plan showing the crossing location is presented on Drawing 01. The terms of reference for the investigation are included in our proposal to Dillon Consulting Limited (Dillon) dated October 9, 2013. The scope of work includes a sub-surface investigation, laboratory testing, and the provision of recommendations for foundations, general site works and slope geometry.

2.0 Background and Existing Information

The existing crossing consists of a single 2.7 m diameter by 7.9 m long concrete pipe with concrete headwalls and wingwalls (Figure 01). While the concrete pipe appears to be in relatively good condition (Figure 02), the wingwalls are badly cracked and are leaning towards the channel (Figure 03). The crossing has reached the end of its service life and is scheduled for replacement. It is our understanding that replacement options include a single precast concrete pipe without headwalls, a reinforced concrete box culvert with headwalls and wingwalls, and a clear span bridge[1].

Figure 01 View SE at Crossing

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[1] Report by Bruce Harding Consultants dated November 2013

Figure 02
View E (upstream) at Concrete Pipe

Figure 03
View SE at Wingwall

3.0 Field Program

3.1 Site Reconnaissance

A site reconnaissance was carried out on November 4th by Mr. Ken Skaftfeld, P.Eng. and Mr. Brent Hay, P.Eng. of TREK. A set of photographs taken at the time of the reconnaissance are included in Appendix A with select photos presented in this report. Photo locations are based on a hand held GPS and are shown on the attached figure in Appendix A. The reconnaissance was carried out to assess the general condition of the channel in the vicinity of the crossing, in particular, any visual evidence of bank instabilities. Test hole locations were also determined at the time of the site reconnaissance.

3.2 Sub-Surface Investigation

A sub-surface investigation was undertaken on November 15, 2013 under the supervision of TREK personnel to determine the soil stratigraphy and groundwater conditions at the site. Test holes TH13-01 and TH13-02 were drilled to power auger refusal (PAR) at the locations shown on Drawing 01. Test holes were drilled using a Soilmec STM-20 truck mounted piling rig equipped with a 508 mm diameter auger. Sub-surface soils observed during the drilling were visually classified based on the Unified Soil Classification System (USCS). Samples retrieved during drilling included disturbed grab samples and relatively undisturbed Shelby tube samples which were transported to TREK's laboratory in Winnipeg, Manitoba for further classification and testing.

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 were referenced to existing features at the site and elevations were surveyed by TREK personnel using a local benchmark established by Dillon.

3.3 Laboratory Testing

Laboratory testing consisted of moisture content determination on all samples. Undrained shear strength testing (pocket penetrometer, torvane and unconfined compression) and unit weight determination was also completed on select undisturbed samples. The laboratory test results are shown on the test hole logs in Appendix A or separately in Appendix B.

4.0 Site Conditions

The Dugald drain consists of a narrow meandering main channel which is incised into a wider overflow channel (Figure 04). The incised channel is about 1 m wide and up to 1 m deep while the overflow channel is terraced and up to about 75 m wide in some locations (below prairie level). The incised channel bed is mud with woody debris. Concrete debris and cobble size rocks can be seen within the channel at the inlet and outlet to the concrete pipe (Figure 05). The overflow channel is grassed with mature trees.

There are a number of localized slump blocks along the toe of the main channel upstream and downstream of the crossing; these slumps are typically at the outside bend of the incised channel and likely as a consequence of continued bank erosion and over-steepening at these locations (Figures 06 and 07). The terraces visible along the overflow channel are believed to be remnants of historical slope movements although none of these appear active. The channel banks in the immediate vicinity of the crossing appear stable, although they are generally over-steepened (Figures 08 and 09).

Figure 04
View W Downstream of Crossing

Figure 05
Channel at Outlet of Concrete Pipe

Figure 06
Toe Slump Downstream of Crossing

Figure 07
Toe Slump Upstream of Crossing

Figure 08
Bank at SW Wingwall

Figure 09
Bank at NW Wingwall

5.0 Sub-Surface Conditions

5.1 Soil Stratigraphy

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

Clay (Fill)

Silty clay (fill) was encountered at surface in both test holes drilled behind the wingwalls and extends to 2.1 m to 2.4 m below ground surface (bgs). The clay is dark brown to black with trace organics and contains varying amounts of gravel (estimated to range from 20% to 35% by weight). Moisture contents range from 13% to 20%, with an average of 16%. The bulk unit weight (based on one sample) is 18.4 kN/m^3. The clay portion of the fill is considered to be highly plastic with a stiff consistency.

Silty Clay

High plastic silty clay was observed below the clay fill in both test holes to 13.7 m (TH13-02) and 14.6 m (TH 13-01) bgs. The clay is mottled brown and grey, becoming grey at a depth of approximately 7.6 m bgs. Moisture contents generally increase with depth, ranging from 36% to 61%, with an average of 46%. Bulk unit weights range from 17.0 to 17.4 kN/m^3 with an average of 17.2 kN/m^3 based on seven tests. Undrained shear strengths generally decrease with increasing depth, ranging from about 75 kPa immediately below the clay fill (stiff) to 25 kPa below a depth of approximately 9 m (soft).

It should be noted that cobbles were encountered in the clay below a depth of 7 m in TH 13-02 and 12.5 m in TH 12-01. Based on observations of auger resistance by the operator, it is also possible a boulder was encountered at a depth of 8.3 m in TH 13-02.

Silt (Till)

Silt (till) underlies the silty clay at a contact elevation of 214.0 m (TH 13-01) and 215.2 m (TH 13-02). The till contains trace clay, trace gravel, is moist, light grey and has low plasticity. Moisture contents decrease with depth, ranging from 15% below the clay contact to 3% near the termination of the test holes. Corresponding to the moisture contents, the consistency of the till ranges from loose below the clay contact to very dense near the termination of the test holes (at power auger refusal).

5.2 Power Auger Refusal

Power auger refusal was reached at depths between 16.6 to 17.4 m in silt till. It is unclear if refusal occurred on boulders within the till but the very low moisture contents from samples collected near the termination depth are reflective of a very dense till matrix often associated with power auger refusal in Winnipeg.

5.3 Seepage and Sloughing Conditions

Minor seepage occurred at undetermined depths within the silt (till). The depth to water in the open holes was 14.2 m bgs in TH 13-02 and 14.9 m bgs in TH 13-01 about 10 to 15 minutes after drilling. Sloughing of the till was observed below the measured water levels in both test holes.

These observations are short term and should not be considered reflective of (static) groundwater levels in the silt (till), 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.

6.0 Foundation Recommendations

6.1 Limit States Design

A shallow foundation (*e.g.* wide footing or mat foundation) would be a feasible alternative for the concrete pipe or box culvert options. The single span bridge option would require a deep (piled) foundation. Recommendations for foundation design provided in this report are based on limit states design following the 2006 Canadian Highway Bridge Design Code (CHBDC). Table 6-1 summarizes the resistance factors provided in Chapter 6 of the CHBDC and which have been used where applicable to determine the factored geotechnical resistance or capacities at the Ultimate Limit State (ULS) for shallow and deep foundations. Unless otherwise noted, a resistance factor based on the results of static analysis (*e.g.* 0.4 for compression) has been used to determine the factored geotechnical resistances for piled foundations; higher resistance factors can only be used if static or dynamic load tests are carried out which are not likely economical for a project of this size.

Table 6-1 ULS Resistance Factors for Foundations (CHBDC, 2006)

APPLICATION		RESISTANCE FACTOR
Shallow Foundations	Bearing resistance	**0.5**
	Passive resistance	**0.5**
	Horizontal resistance (sliding)	**0.8**
Ground anchors (soil or rock)	Static analysis - Tension	**0.4**
	Static analysis - Tension	**0.6**
Deep Foundations - Piles	Static analysis – Compression	**0.4**
	Static Analysis – Tension	**0.3**
	Static Test – Compression	**0.6**
	Static test – Tension	**0.4**
	Dynamic analysis – Compression	**0.4**
	Dynamic test – Compression (field measurements and analysis)	**0.5**
	Horizontal passive resistance	**0.5**

6.2 Shallow Foundations

Provided seasonal movements relating to moisture changes and freeze/thaw are tolerable for lightly loaded structures such as the concrete pipe or box culvert options, a shallow (mat) foundation would be an appropriate foundation system, with the option of thickened edges around the perimeter of the mat. A circular concrete pipe may have a concrete cradle.

Shallow foundations bearing on undisturbed clay can be designed using a Service Limit State (SLS) bearing capacity of 100 kPa and a factored geotechnical resistance at the Ultimate Limit State (ULS) of 150 kPa (based on a resistance factor of 0.5). Sliding is not expected to be a concern for design; however Limit States design values can be provided if necessary once the bedding material type is known.

Additional considerations for the design and construction of the circular pipe and box culvert options are provided below:

1. Based on an anticipated elevation of about 224.0 m for the foundation base, it is expected that the bearing surface for either the circular pipe or box culvert will be undisturbed clay. However, if encountered, any organics, silts, fill soils, and any other deleterious material should be stripped such that the subgrade consists of native, undisturbed high plastic clay. No such deleterious materials were encountered at this depth in the test holes, although conditions may vary between test holes.
2. Subgrade 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.
3. After excavation, the subgrade should be inspected by a qualified geotechnical engineer and compacted to 95% of Standard Proctor Maximum Dry Density (SPMDD). The exposed subgrade surface should be protected from freezing, drying, inundation and disturbance. If any of these conditions occur the subgrade should be scarified, moisture conditioned as appropriate, and re-compacted to a minimum of 95% of SPMDD.
4. Where soft or weak subgrade materials are identified by the geotechnical engineer, these areas should be repaired as directed by the geotechnical engineer. This may require excavation and placement/compaction of in-situ material or granular material.
5. A concrete cradle for a circular pipe can be poured neat with the subgrade if in agreement and in accordance with the supplier's recommendations.
6. The granular bedding for a circular pipe should consist of granular fill and thickness in accordance with the supplier's recommendation. The granular bedding for a concrete box culvert should consist of a minimum of 150 mm of crushed limestone base material (19 mm down) with a 75 mm thick levelling course of finer granular fill material if necessary. The granular base for box culverts should be placed in lifts not exceeding 150 mm thickness and compacted to 98% of SPMDD.
7. Where the underside of the granular fill is less than 2.4 m below grade, ground freezing can be expected in the long-term and frost heave may occur. If potential ground movements associated with ground freezing cannot be tolerated, TREK can provide additional recommendations for remedial measures, which may consist of a thickened sub-base or a rigid polystyrene skirt for insulation.
8. A concrete toe (curtain) wall is recommended at the upstream and downstream ends of the box culvert structure. The wall should extend 300 mm minimum below the underside of the structure and in any case, should penetrate through the full thickness of the granular base material and into

the underlying clay. Any exposed granular base and granular base materials beyond the perimeter of the structural slab should be capped with impermeable clay compacted to 95% SPMDD to prevent saturation of the bearing soil and scouring during high flow events.

6.3 Deep Foundations

6.3.1 *Cast-in-Place Concrete Friction Piles*

The factored geotechnical resistance values (adhesion and end bearing) at the ULS for determining the capacity of cast-in-place concrete friction piles are provided in Table 6-2 (based on a resistance factor of 0.4). The pile capacity can be calculated based on the adhesion values provided in Table 6-2 for evaluation of the Service Limit State. The contribution from end bearing should be ignored for the calculation of the SLS pile capacity. 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.

Table 6-2 Adhesion Values for Cast-in-Place Concrete Friction Piles (Compression)

Soil	Depth (m)		ULS[1]		SLS Skin Friction Value
	From	To	Factored Adhesion	Factored End-Bearing	
Silty Clay / Frost Zone	0.0	2.4	0	0	0
Silty Clay	2.4	Elev. 220 m	18 kPa	160 kPa	15 kPa
Silty Clay	Elev. 220 m	Elev. 214m	12 kPa	100 kPa	10 kPa

[1] ULS - A Resistance Factor of 0.4 has been 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. Cobbles (and possibly boulders) may be encountered during drilling for cast-in-place piles. While it is not anticipated that the frequency or size of cobbles is sufficient to prevent auger advancement, piling contractors should be made aware of the possibility of boulders and be prepared to extract any such obstructions if necessary.
3. Adhesion within the upper 2.4 m of the pile 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.
4. A minimum pile length of 9 m below ground surface is generally recommended for exterior or unheated straight shaft piles to protect against frost jacking. In this regard, uplift forces due to ad-freezing in the upper 2.4 m below ground should be based on an uplift adhesion of 65 kPa. Frost

jacking can be resisted by structural dead loads as well as uplift resistance afforded by the pile length below the depth of frost.

5. A factored geotechnical resistance value for uplift against frost jacking forces or due to live loads on the piles of 13 kPa to a depth of 8 m (approx. Elev. 220m) and 8 kPa below Elev. 220.0 m should be used (a resistance factor of 0.3 has been applied).
6. 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.
7. All piles should be reinforced for their full length.
8. Based on observed conditions, sleeving of drilled shafts will likely be unnecessary. Seepage conditions at the time of construction may differ from that observed at the time of drilling, in particular from near surface layers. If seepage and sloughing conditions are observed during drilling, sleeves should be used.
9. Piles should not extend any deeper than elevation 214.0 m to avoid drilled shafts from penetrating the till unit, which may result in seepage or sloughing during construction.
10. Drilling and concrete placement for the piles should be inspected by a geotechnical engineer 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.
13. Grade beams and pile caps should be constructed with a minimum 150 mm void space between soils and the underside of the concrete to minimize the effects of soil heave due to swelling or frost action.
14. All cast-in-place piles require reinforcement design by a qualified structural engineer for the required axial, lateral and bending loads from the structure.

6.3.2 Driven Precast Concrete Piles

Both SLS and ULS pile capacities are provided in Table 3 for precast-prestressed hexagonal concrete (PPHC) 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 6-3.

The SLS capacity provided in Table 6-3 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 specified.

Table 6-3 Recommended ULS and SLS Pile Capacity 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	570	445	5
	360 mm	800	625	8
	405 mm	1000	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. Cobbles (and possibly boulders) may be encountered in the clay unit overlying till. If a pile cannot be advanced into the till, a replacement pile may be necessary.
4. 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.
5. The piles should be specified to have cured for at least 7 days prior to driving.
6. 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 must be filled with lean mix concrete for compliance with the surrounding soil.
7. Piles should be driven continuously once driving is initiated to the required refusal criteria.
8. All piles driven within 5 pile diameters of a previously installed (driven) pile should be monitored for pile heave. If pile heave is observed, all piles should be checked. Piles that have heaved should be re-driven to the refusal criteria.
9. 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.
10. Inspection of driven pile installation should be undertaken by qualified and experienced geotechnical engineer familiar with this type of pile installation.
11. Any piles damaged, misaligned an excessive amount or reaching premature refusal may need to

be abandoned and replaced. The structural designer should assess non-conforming piles to determine if they are acceptable.

12. 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. Any existing foundations should be excavated and removed to a depth of at least 0.5 m from the underside of grade beams and pile caps.

6.3.3 *Steel Piles*

Steel H piles or pipe piles driven to practical refusal could be considered, in particular if it is necessary to achieve individual pile capacities higher than for PPHC piles. Capacities for steel piles and recommendations for their installation can be provided upon request.

6.4 Lateral Loads

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 (Ks). For clays, the lateral subgrade reaction modulus is typically independent of depth or vertical overburden stress. Table 6-4 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. The lateral subgrade reaction within 2.4 m of ground surface should be neglected due to fill materials, silts, seasonal moisture changes and freeze/thaw effects. 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 6-4 Recommended Values for Lateral Subgrade Reaction Modulus (K_s)

Soil	Depth (m)		K_s (kN/m^3)
	From	To	
Fill Soils, Silts, and Frost Zone	0.0	2.4	0
Clays	2.4	6.0	2000 / 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 rigorous elastic-plastic model of the soil response to loading can be carried out by TREK out to confirm the lateral load capacity of the piles and pile groups, if required.

6.5 Ad-freezing

Buried concrete structures can be subject to ad-freezing forces acting along their vertical surfaces. For foundation components located within the depth of frost penetration, an ultimate (unfactored) load of 65 kPa is recommended for design. These forces will be resisted by structural dead loads and uplift resistance afforded by the length of the foundation system below the depth of frost penetration. For design, a frost penetration depth of 2.4 m should also be assumed. A factored geotechnical resistance value for uplift against frost jacking forces or due to live loads on the piles of 13 kPa to a depth of 8 m (approx. Elev. 220m) and 8 kPa below Elev. 220.0 m should be used (a resistance factor of 0.3 has been applied). Alternatively, measures such as rigid Styrofoam insulation could be considered to reduce frost penetration depths and thereby uplift forces.

7.0 Foundation Concrete

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 Excavations and Shoring

All excavations must be carried out in compliance with the appropriate regulation(s) under the Manitoba Workplace Safety and Health Act. Cantilevered (unbraced) walls should be designed using the earth pressure coefficients outlined in Table 8-1 for the appropriate earth pressure condition. Braced excavations in silt or clay/clay fill should be designed using the earth pressure distributions shown on Figure 10 (attached). The effect of any surcharge loads must be added to the force on the wall in addition to the calculated earth pressures, as noted in the figures.

Table 8-1 Recommended Design Parameters for Cantilevered Walls (Shoring)

Earth Pressure Condition	Earth Pressure Coefficient
	Clay / Clay Fill
Active (K_a)	0.5
At-rest (K_o)	0.65
Passive (K_p)	2.0

A certain amount of ground movement behind the shoring will occur, and is largely unavoidable. The amount of movement that will occur cannot be accurately predicted, mainly because the movement is as much a function of excavation procedures and workmanship as it is a function of theoretical considerations. It is anticipated that the design of temporary shoring will be the responsibility of the Contractor. The proposed shoring design should be reviewed prior to construction and the performance of the excavation system monitored during and subsequent to construction. Basal instability in the clay/clay fill is not expected to be of concern for the anticipated depth of any necessary excavations (less than 4m).

9.0 Lateral Earth Pressure

The magnitude of lateral earth pressures from retained soil against permanent culvert wing-walls will depend on the backfill material type, method of placement, backfill compaction 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 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.

An active pressure coefficient (Ka) of 0.3 should be used to calculate lateral loads from free draining granular soils against retaining structures which are free to translate horizontally by at least 0.2 percent of the wall height. For retaining structures which are not free to translate, an at-rest earth pressure coefficient (Ko) of 0.4 should be used. An appropriate surface surcharge should also be included in the earth pressure distribution to account for potential surface loads. The active pressure coefficient (Ka) can be used to calculate the component of lateral load on wall structures due to surcharge loads.

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 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. Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

10.0 Slope Stability

Slope stability analysis was carried out to estimate the factor of safety (FS) of the existing slopes at the crossing and determine appropriate slope geometry for long term stability. In this regard, the existing FS for the existing slopes on either side of the drain are estimated to range from 1.1 to 1.3 with the lower values associated with steeper slopes (*e.g.* the southwest side). With respect to a design objective for long term stability, we recommend a minimum FS of 1.3 be used under channel drawdown conditions associated with the recession of a spring flood. To satisfy this design objective, the recommended slope geometry for varying bank heights up to 6 m are summarized in Table 10-1. The slope angles in Table 10-1 should extend laterally to a distance equal to or greater than the length of the slope measured from the structure.

Table 10-1 Recommended Channel Side Slopes

Slope Height (from base of channel)	Recommended Channel Side Slope (horizontal to vertical)
Less than 4 m	3H:1V
Between 4 and 5 m	4H:1V
Between 5 and 6 m	4.5H:1V
Greater than 6 m	Additional analysis required if slope height exceed 6 m

11.0 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 sub-surface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

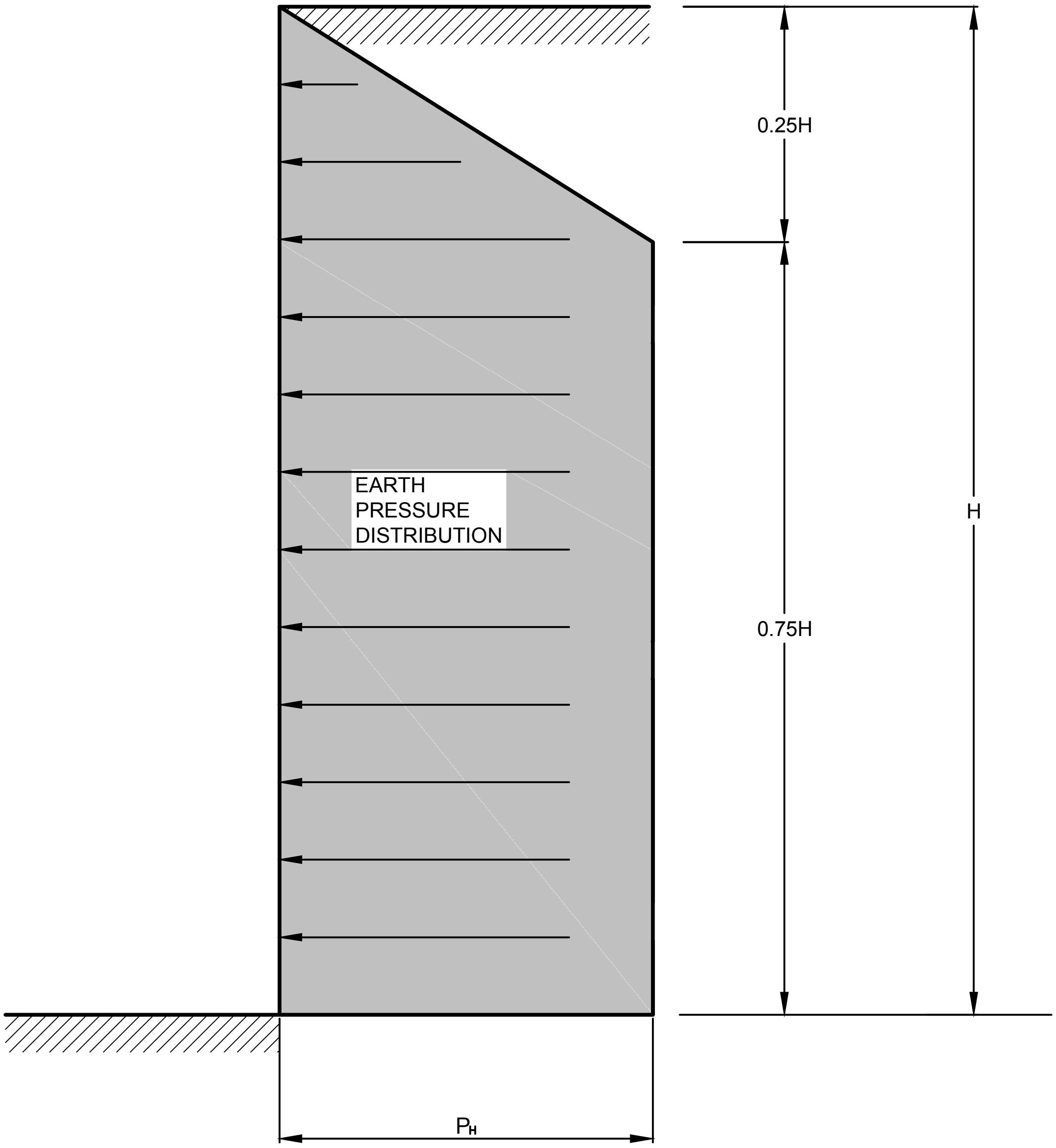
All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already 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 Dillon Consulting Limited (the Client) and their agents for the proposed crossing replacement. 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.

Figures

0022 011 00
Dillon Consulting Ltd.
Happyland Park Culvert Replacement

8 1/2" x 11"
PLOT: 09/12/2013 2:35:00 PM
FILE NAME: 0022 001 00_F_RB.dwg



$P_H = K_a \gamma H$

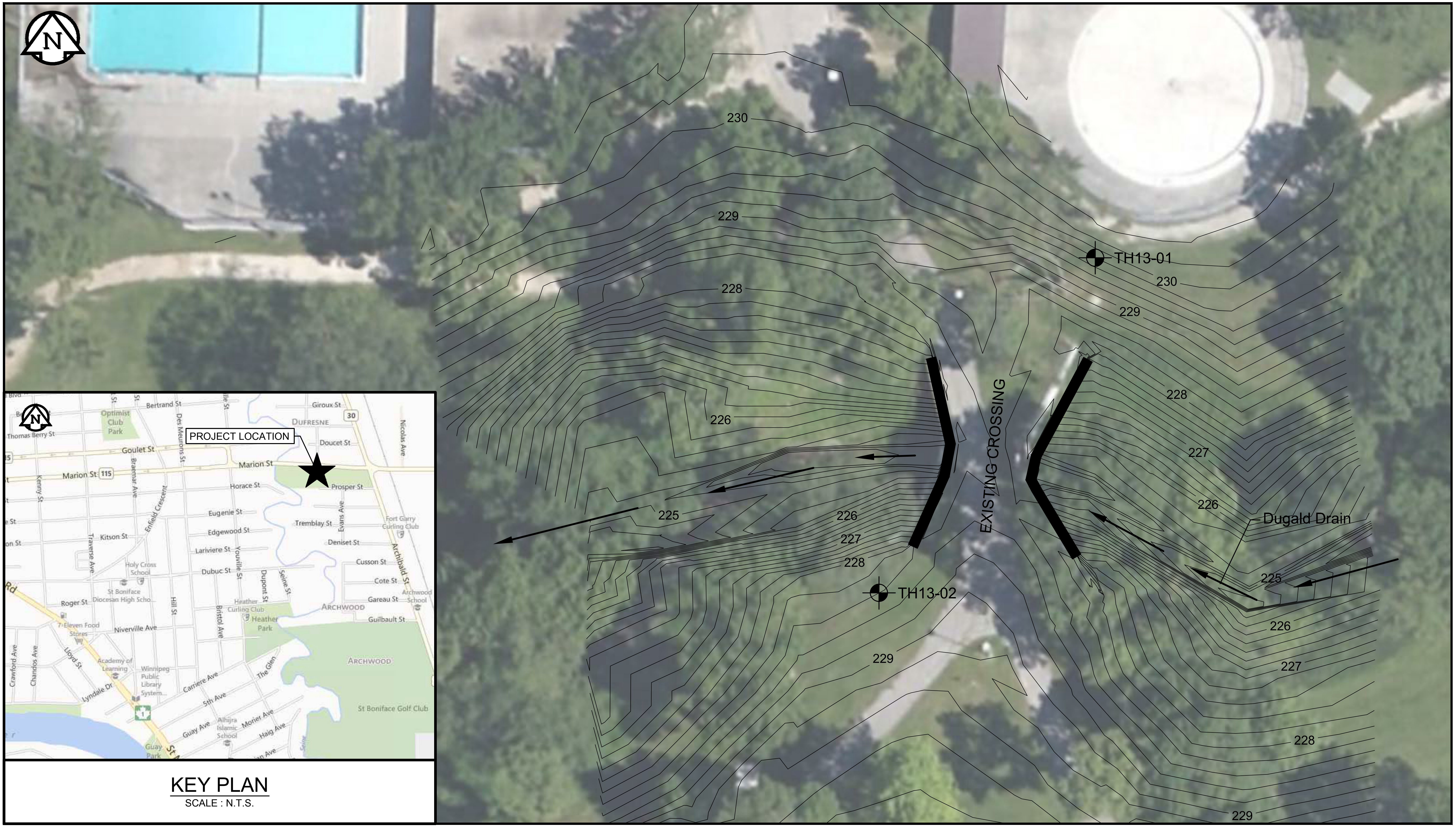
WHERE:
H = DEPTH OF EXCAVATION (m)
P_H = LATERAL EARTH PRESSURE (kPa)
γ = SOIL UNIT WEIGHT (kN/m³) = 17.2 (clay), 18.4 (clay fill).
K_a = ACTIVE EARTH PRESSURE COEFFICIENT = 0.5

NOTE:
ADD SURFACE LOAD SURCHARGE WHERE APPLICABLE

Figure 10
Apparent Earth Pressure Distributions
Temporary Braced Shoring

Drawings

0022 011 00
Dillon Consulting Ltd.
Happyland Park Culvert Replacement



KEY PLAN
SCALE : N.T.S.

Tabloid (279mm x 432mm)

PLOT: 09/12/2013 2:48:38 PM

FILE NAME: 0022 011 00_RE.dwg

0 2.5 5 7.5 10m
SCALE : 1:300 (279mm x 432mm)

LEGEND :

⌖ TEST HOLE (TREK, NOV.15, 2013)

NOTES :

1. AERIAL IMAGE FROM GOOGLE EARTH ON JUNE 22, 2013
2. CONTOUR INTERVAL = 0.2m.

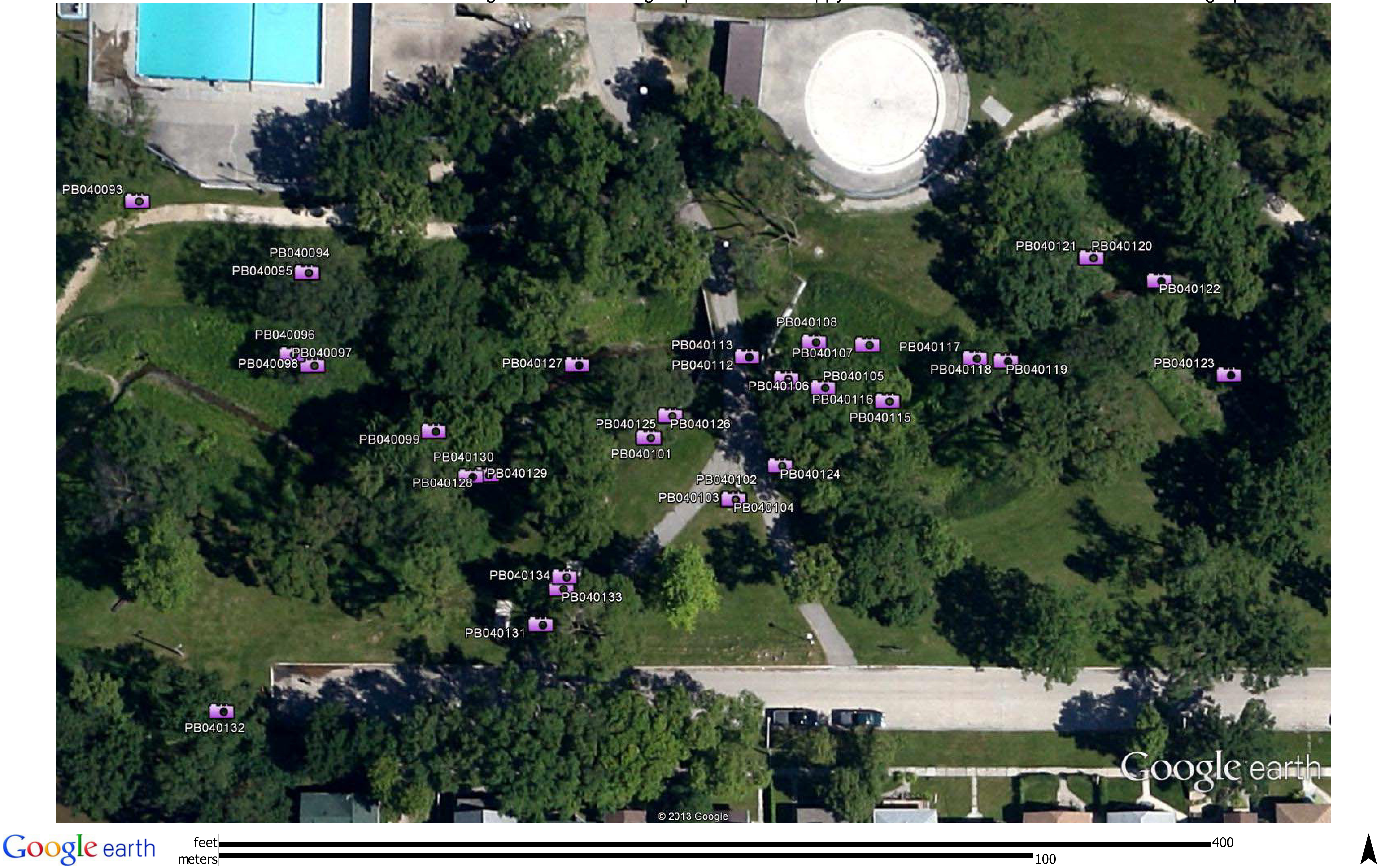
Drawing 01
Site and Test Hole Location Plan

Appendix A

Photographs From site Reconnaissance

Dugald Drain Crossing Replacement - Happyland Park

Photograph Locations



PB040085

PB040086

PB040087

PB040088

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040089

PB040090

PB040091

PB040092

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040093

PB040094

PB040095

PB040096

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040097

PB040098

PB040099

PB040100

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040101

PB040102

PB040103

PB040104

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040105

PB040106

PB040107

PB040108

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040109

PB040110

PB040111

PB040112

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040113

PB040114

PB040115

PB040116

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040117

PB040118

PB040119

PB040120

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040121

PB040122

PB040123

PB040124

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040125

PB040126

PB040127

PB040128

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040129

PB040130

PB040131

PB040132

Dugald Drain Crossing at Happyland Park - November 4, 2013

PB040133

PB040134

Dugald Drain Crossing at Happyland Park - November 4, 2013

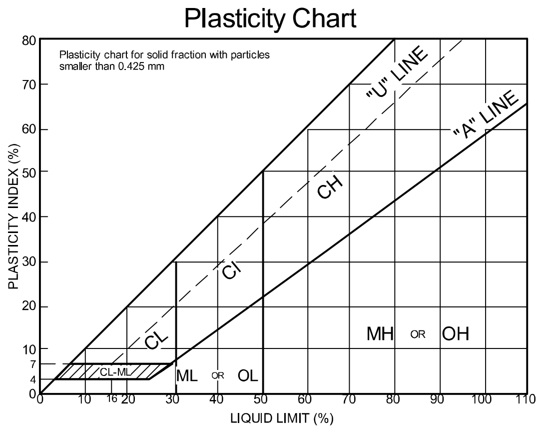
Appendix B

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 Classification	Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW	●	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent....... GW, GP, SW, SP More than 12 percent....... GM, GC, SM, SC 6 to 12 percent...... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	○	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM	▥	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are 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 TH13-01

1 of 2

Client: Dillon Consulting Limited

Project Name: Dugald Drain Crossing Replacement, Happyland Park

Contractor: Subterranean Ltd.

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

Project Number: 0022 011 00

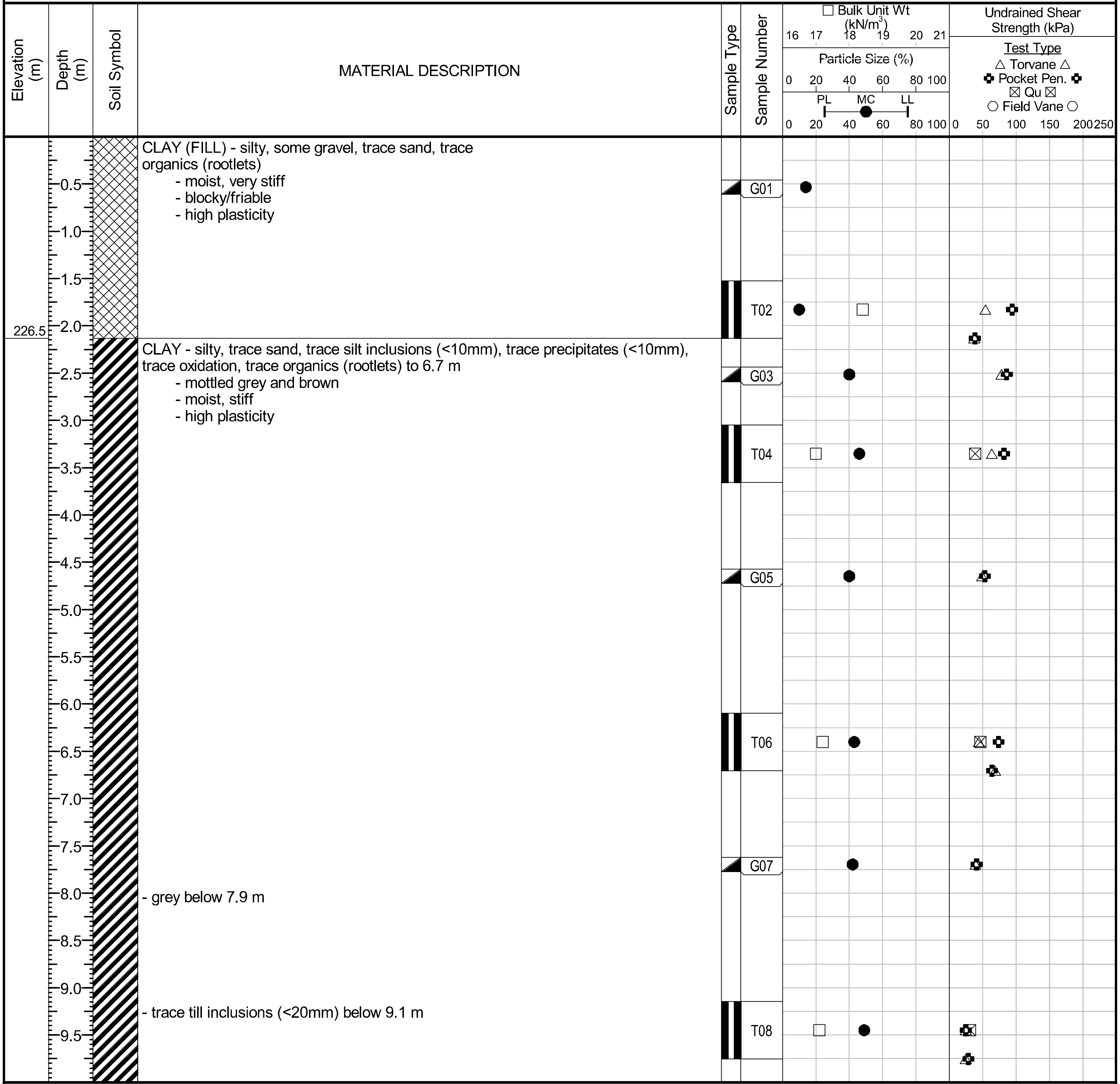
Location: 8m N from center of culvert, 6m E of centerline of pathway.

Ground Elevation: 228.67 m

Date Drilled: 15 November 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, some gravel, trace sand, trace organics (rootlets)
- moist, very stiff
- blocky/friable
- high plasticity

CLAY - silty, trace sand, trace silt inclusions (<10mm), trace precipitates (<10mm), trace oxidation, trace organics (rootlets) to 6.7 m
- mottled grey and brown
- moist, stiff
- high plasticity

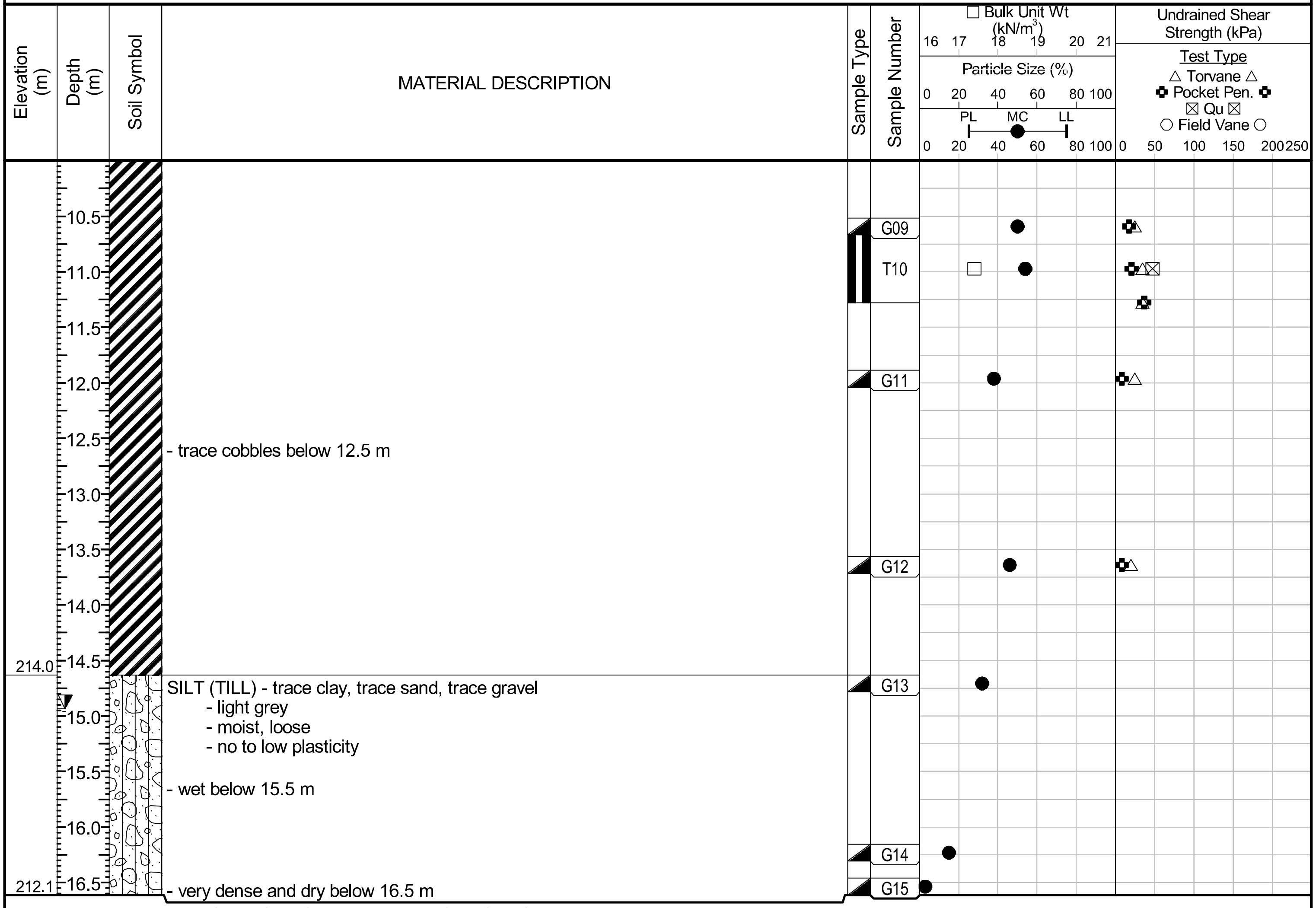
- grey below 7.9 m

- trace till inclusions (<20mm) below 9.1 m

Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

Sub-Surface Log

Test Hole TH13-01



- trace cobbles below 12.5 m

SILT (TILL) - trace clay, trace sand, trace gravel
- light grey
- moist, loose
- no to low plasticity

- wet below 15.5 m

- very dense and dry below 16.5 m

END OF TEST HOLE At 16.6 m in SILT (TILL)
Notes:
1) Power auger refusal at 16.6 m depth.
2) Sourced seepage not observed.
3) Water level at 14.9 m depth immediately after drilling.
4) Test hole sloughed below 15.8 m depth upon completion.
5) Test hole backfilled with cuttings to 0.6 m below surface and topped with benotinte chips.

Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH13-02

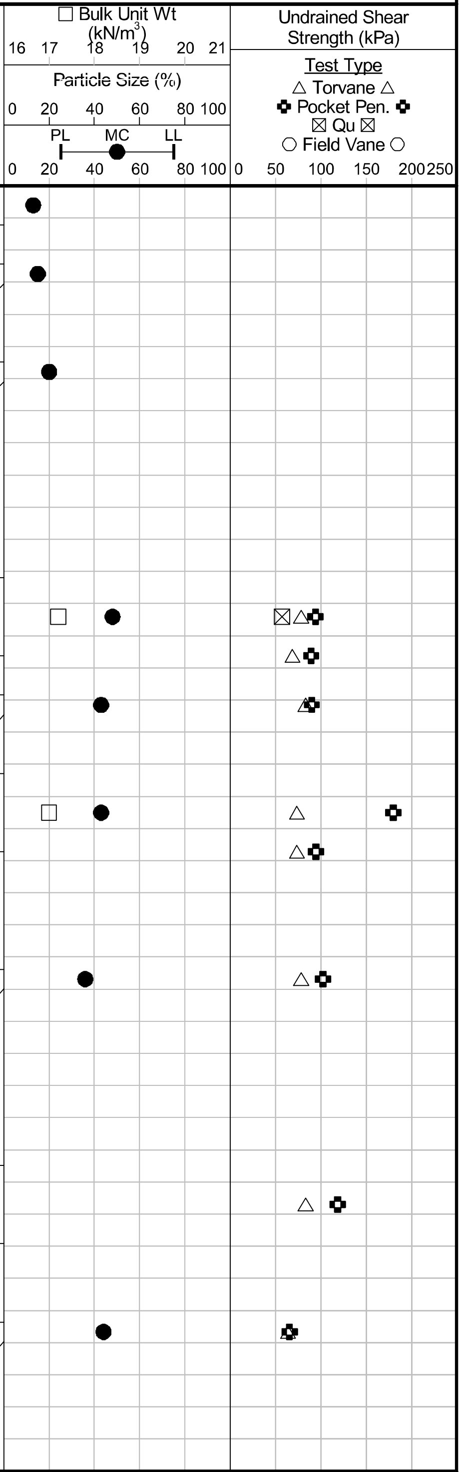
Client:	Dillon Consulting Limited	**Project Number:**	0022 011 00
Project Name:	Dugald Drain Crossing Replacement, Happyland Park	**Location:**	7m S from center of culvert, 3.7 m W of centerline of pathway
Contractor:	Subterranean Ltd.	**Ground Elevation:**	228.87 m
Method:	508 mm Auger, Soilmec STM-20 Truck Mount	**Date Drilled:**	15 November 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
	0.0 – 2.45	▩	CLAY (FILL) - silty, some gravel to gravelly, trace sand, trace silt inclusions (<10mm), trace oxidation, trace organics (rootlets) - black - moist, very stiff - blocky/friable - high plasticity - trace to some gravel below 1.5m
226.4	2.45 –	▨	CLAY - silty laminations (<2mm), trace sand, trace silt inclusions (<20mm), trace precipitates (<5mm), trace oxidation, trace organics (rootlets) - mottled grey and brown - moist, very stiff - high plasticity - trace cobbles at 7.0 m - grey and stiff below 7.3 m

Sample Type	Sample Number
◩	G16
◩	G17
◩	G18
▮	T19
◩	G20
▮	T21
◩	G22
▮	T23
◩	G24

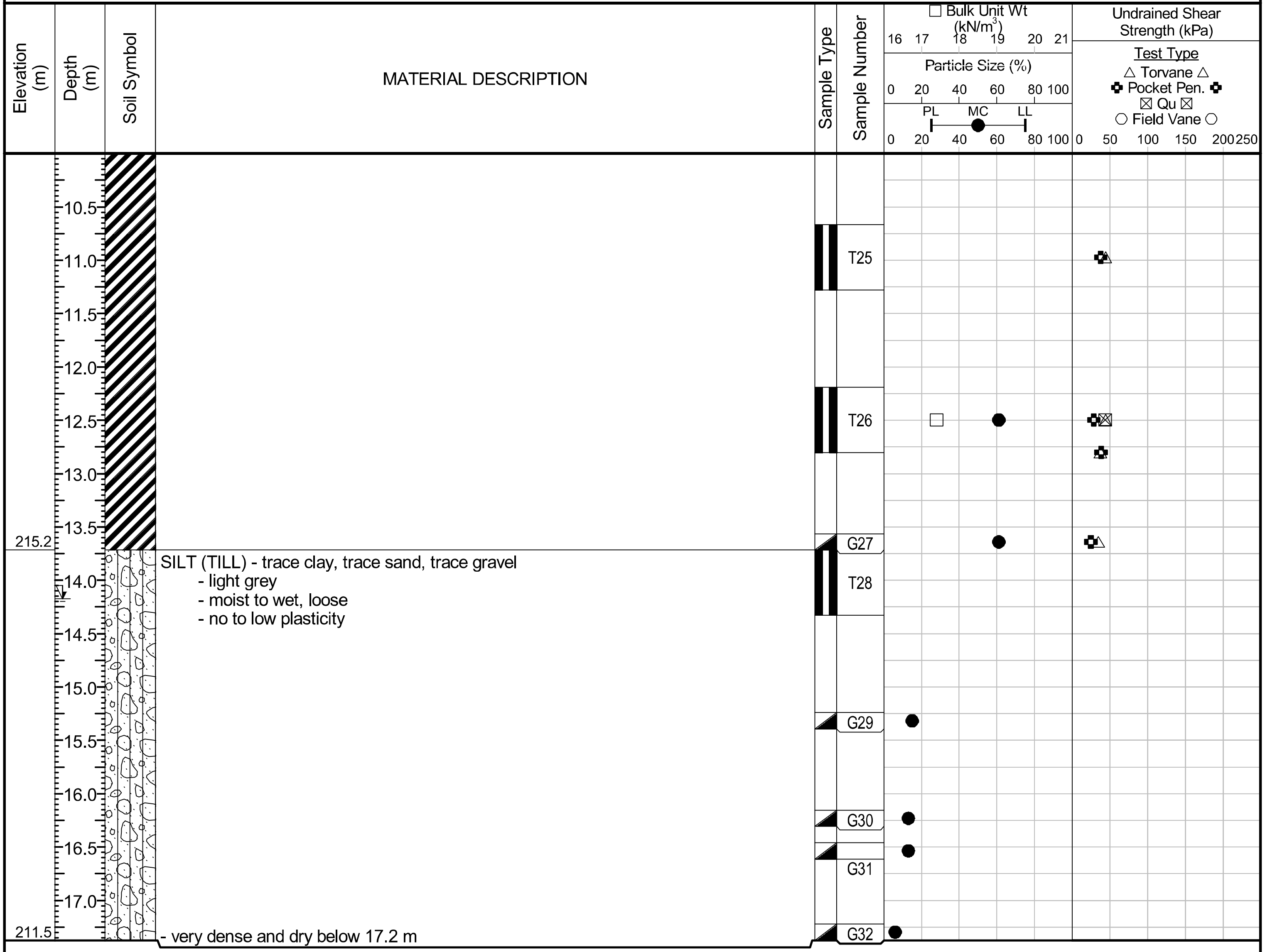


Logged By: Martial Lemoine	**Reviewed By:** Ken Skaftfeld	**Project Engineer:** Ken Skaftfeld

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH13-02



SILT (TILL) - trace clay, trace sand, trace gravel
- light grey
- moist to wet, loose
- no to low plasticity

- very dense and dry below 17.2 m

END OF TEST HOLE At 17.4 m in SILT (TILL)

Notes:

1) Power auger refusal at 17.4 m depth.
2) Sourced seepage not observed.
3) Water level at 14.2 m depth immediately after drilling
4) Test hole sloughed below 15.8 m depth upon completion.
5) Possible boulder or cobble at 8.3 m
6) Test hole backfilled with cuttings to 0.6 m below surface and topped with benotinte chips.

Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

Appendix C

Laboratory Testing Results

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Winnipeg, MB R3H 0L3
Tel: 204.975.9433 Fax: 204.975.9435

Moisture Content Report
ASTM D2216-98

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

Sample Date 15-Nov-13
Test Date 18-Nov-13
Technician Chiran Peiris

Test Hole	TH13-01	TH13-01	TH13-01	TH13-01	TH13-01	TH13-01
Depth (m)	0.5 - 0.6	2.4 - 2.6	4.6 - 4.7	7.6 - 7.8	10.5 - 10.7	11.9 - 12.0
Sample #	G01	G03	G05	G07	G09	G11
Tare ID	A23	K13	F122	N56	F75	N06
Mass of tare	8.5	8.4	8.3	8.3	8.5	8.4
Mass wet + tare	355.2	408.9	302.6	289.3	272.3	364.5
Mass dry + tare	312.8	294.1	218.7	206.0	184.3	266.0
Mass water	42.4	114.8	83.9	83.3	88.0	98.5
Mass dry soil	304.3	285.7	210.4	197.7	175.8	257.6
Moisture %	13.9%	40.2%	39.9%	42.1%	50.1%	38.2%

Test Hole	TH13-01	TH13-01	TH13-01	TH13-01	TH13-02	TH13-02
Depth (m)	13.6 - 13.7	14.6 - 14.8	16.2 - 16.3	16.5 - 16.6	0.0 - 0.3	0.6 - 0.8
Sample #	G12	G13	G14	G15	G16	G17
Tare ID	E117	Z43	Z137	Z20	N57	W30
Mass of tare	8.3	8.4	8.2	8.3	8.4	8.5
Mass wet + tare	314.5	379.9	524.9	525.2	300.5	302.1
Mass dry + tare	218.3	290.4	458.9	510.7	267.6	263.4
Mass water	96.2	89.5	66.0	14.5	32.9	38.7
Mass dry soil	210.0	282.0	450.7	502.4	259.2	254.9
Moisture %	45.8%	31.7%	14.6%	2.9%	12.7%	15.2%

Test Hole	TH13-02	TH13-02	TH13-02	TH13-02	TH13-02	TH13-02
Depth (m)	1.4 - 1.8	4.0 - 4.1	6.1 - 6.2	8.8 - 9.0	13.6 - 13.7	15.2 - 15.4
Sample #	G18	G20	G22	G24	G27	G29
Tare ID	F68	Z84	H18	K31	W87	N09
Mass of tare	8.6	8.3	8.5	8.3	8.3	8.5
Mass wet + tare	472.2	312.5	308.6	425.1	301.9	463.2
Mass dry + tare	395.9	221.7	229.4	298.2	190.4	404.5
Mass water	76.3	90.8	79.2	126.9	111.5	58.7
Mass dry soil	387.3	213.4	220.9	289.9	182.1	396.0
Moisture %	19.7%	42.5%	35.9%	43.8%	61.2%	14.8%

Moisture Content Report
ASTM D2216-98

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

Sample Date 15-Nov-13
Test Date 18-Nov-13
Technician Chiran Peiris

Test Hole	TH13-02	TH13-02	TH13-02			
Depth (m)	16.2 - 16.3	16.3 - 16.5	17.2 - 17.4			
Sample #	G30	G31	G32			
Tare ID	N44	W59	N60			
Mass of tare	8.2	8.3	8.2			
Mass wet + tare	450.8	438.3	330.1			
Mass dry + tare	400.3	389.7	307.4			
Mass water	50.5	48.6	22.7			
Mass dry soil	392.1	381.4	299.2			
Moisture %	12.9%	12.7%	7.6%			

Test Hole						
Depth (m)						
Sample #						
Tare ID						
Mass of tare						
Mass wet + tare						
Mass dry + tare						
Mass water						
Mass dry soil						
Moisture %						

Test Hole						
Depth (m)						
Sample #						
Tare ID						
Mass of tare						
Mass wet + tare						
Mass dry + tare						
Mass water						
Mass dry soil						
Moisture %						

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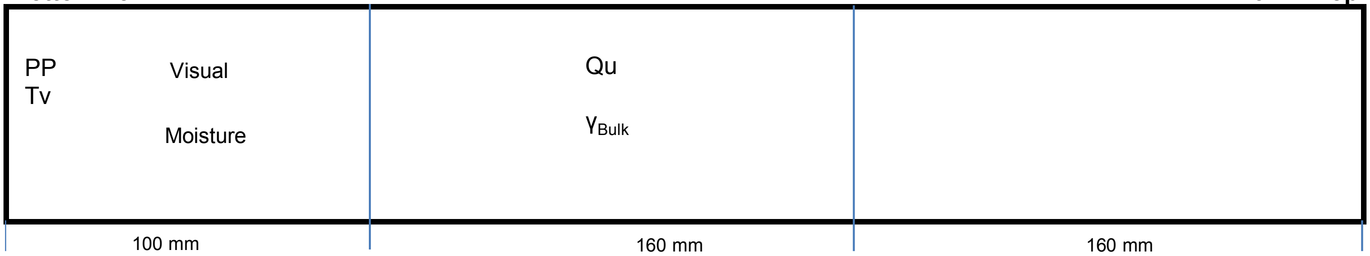
Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement
Test Hole	TH13-01
Sample #	T04
Depth (m)	3.0 - 3.7
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 410

Bottom - 5.1 m **4.6 m - Top**



Visual Classification

Material	CLAY
Composition	silty
trace precipitation (sulphates <5%)	
trace oxidation	
trace organics	
Color	brown
Moisture	moist
Consistency	stiff
Plasticity	high plastic
Structure	lamination <1mm silt infilled
Gradation	-

Torvane

Reading	0.35
Vane Size (s,m,l)	s
Undrained Shear Strength (kPa)	85.8

Pocket Penetrometer

Reading	**1**	1.50
	2	1.50
	3	1.50
	Average	1.50
Undrained Shear Strength (kPa)		73.6

Moisture Content

Tare ID	N10
Mass tare (g)	8.4
Mass wet + tare (g)	348.6
Mass dry + tare (g)	240.7
Moisture %	46.4%

Unit Weight

Bulk Weight (g)		927.50
Length (mm)	**1**	131.44
	2	131.23
	3	131.22
	4	131.36
Average Length (m)		0.131
Diam. (mm)	**1**	71.52
	2	72.47
	3	71.82
	4	71.91
Average Diameter (m)		0.072
Volume (m^3)		5.34E-04
Bulk Unit Weight (kN/m^3)		17.0
Bulk Unit Weight (pcf)		108.5
Dry Unit Weight (kN/m^3)		11.6
Dry Unit Weight (pcf)		74.1

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Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

Test Hole TH13-01
Sample # T04
Depth (m) 3.0 - 3.7
Sample Date 15-Nov-13
Test Date 16-Nov-13
Technician Hachem Ahmed

Unconfined Strength

	kPa	ksf
Max q_u	77.2	1.6
Max S_u	38.6	0.8

Specimen Data

Description CLAY - silty, trace of precipitation (sulphates 5%), trace of oxidation, trace of organic, brown, moist, stiff, high plastic, lamination <1mm silt infilled

Length	131.3	(mm)	**Moisture %**	46%	
Diameter	71.9	(mm)	**Bulk Unit Wt.**	17.0	(kN/m^3)
L/D Ratio	1.8		**Dry Unit Wt.**	11.6	(kN/m^3)
Initial Area	0.00406	(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**
0.35	85.8	1.79
Vane Size		
s		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
1.50	73.6	1.54
1.50	73.6	1.54
1.50	73.6	1.54
1.50	**73.6**	**1.54**

Failure Geometry

Sketch:

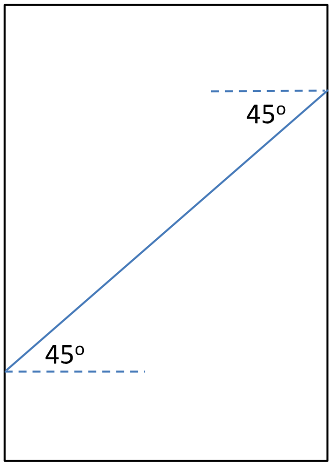


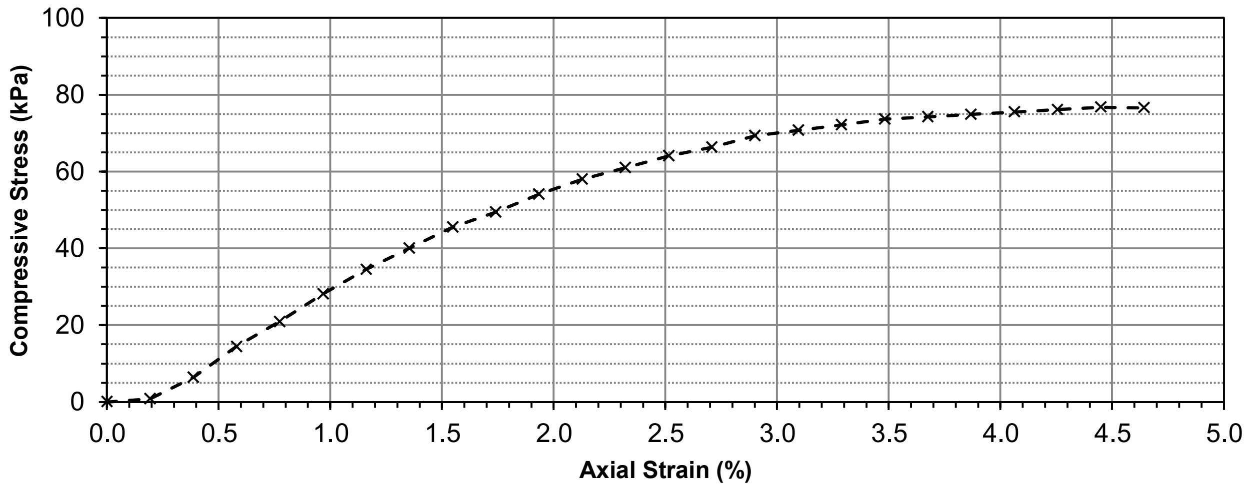
Photo:

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	0.004064	0.0	0.00	0.00
10	1	0.2540	0.19	0.004071	3.3	0.80	0.40
20	8	0.5080	0.39	0.004079	26.2	6.41	3.21
30	18	0.7620	0.58	0.004087	58.9	14.42	7.21
40	26	1.0160	0.77	0.004095	85.7	20.93	10.47
50	35	1.2700	0.97	0.004103	115.4	28.12	14.06
60	43	1.5240	1.16	0.004111	141.8	34.48	17.24
70	50	1.7780	1.35	0.004119	164.9	40.02	20.01
80	57	2.0320	1.55	0.004127	187.9	45.53	22.77
90	62	2.2860	1.74	0.004136	204.4	49.42	24.71
100	68	2.5400	1.93	0.004144	224.2	54.10	27.05
110	73	2.7940	2.13	0.004152	240.7	57.97	28.99
120	77	3.0480	2.32	0.004160	253.9	61.02	30.51
130	81	3.3020	2.51	0.004168	267.1	64.07	32.04
140	84	3.5560	2.71	0.004177	276.9	66.31	33.15
150	88	3.8100	2.90	0.004185	290.2	69.33	34.67
160	90	4.0640	3.09	0.004193	296.7	70.76	35.38
170	92	4.3180	3.29	0.004202	303.3	72.19	36.09
180	94	4.5720	3.48	0.004210	309.9	73.61	36.80
190	95	4.8260	3.68	0.004219	313.2	74.24	37.12
200	96	5.0800	3.87	0.004227	316.5	74.88	37.44
210	97	5.3340	4.06	0.004236	319.8	75.51	37.75
220	98	5.5880	4.26	0.004244	323.1	76.13	38.07
230	99	5.8420	4.45	0.004253	326.4	76.75	38.38

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	99	6.0960	4.6424	0.004261	326.4	76.60	38.30
250	100	6.3500	4.84	0.004270	329.7	77.21	38.61
260	100	6.6040	5.03	0.004279	329.7	77.06	38.53
270	100	6.8580	5.22	0.004288	329.7	76.90	38.45
280	100	7.1120	5.42	0.004296	329.7	76.74	38.37
290	99	7.3660	5.61	0.004305	326.4	75.82	37.91
300	98	7.6200	5.80	0.004314	323.1	74.90	37.45
310	97	7.8740	6.00	0.004323	319.8	73.99	36.99
320	96	8.1280	6.19	0.004332	316.5	73.07	36.54
330	94	8.3820	6.38	0.004341	309.9	71.40	35.70
340	92	8.6360	6.58	0.004350	303.3	69.74	34.87
350	89	8.8900	6.77	0.004359	293.4	67.33	33.66
360	87	9.1440	6.96	0.004368	286.8	65.67	32.83
370	84	9.3980	7.16	0.004377	276.9	63.28	31.64
380	81	9.6520	7.35	0.004386	267.1	60.89	30.45
390	77	9.9060	7.54	0.004395	253.9	57.76	28.88
400	73	10.1600	7.74	0.004404	240.7	54.65	27.32

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Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement
Test Hole	TH13-01
Sample #	T06
Depth (m)	6 - 6.7
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 600

Bottom - 5.1 m **4.6 m - Top**

PP Tv Visual Moisture	Qu γ_{Bulk}	Visual	
100 mm	160 mm	120 mm	220 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<10mm diameter)	
trace gravel (<10mm diameter)	
trace organics	
Color	brown
Moisture	moist
Consistency	stiff
Plasticity	high plastic
Structure	somewhat slicken sided
Gradation	-

Torvane

Reading	0.70
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	68.7

Pocket Penetrometer

Reading	**1**	1.40
	2	1.20
	3	1.30
	Average	1.30
Undrained Shear Strength (kPa)		63.7

Moisture Content

Tare ID	W108
Mass tare (g)	8.7
Mass wet + tare (g)	383.2
Mass dry + tare (g)	270.7
Moisture %	42.9%

Unit Weight

Bulk Weight (g)		1089.00
Length (mm)	**1**	150.44
	2	150.30
	3	150.41
	4	150.12
Average Length (m)		0.150
Diam. (mm)	**1**	72.38
	2	72.93
	3	72.75
	4	72.03
Average Diameter (m)		0.073
Volume (m^3)		6.21E-04
Bulk Unit Weight (kN/m^3)		17.2
Bulk Unit Weight (pcf)		109.5
Dry Unit Weight (kN/m^3)		12.0
Dry Unit Weight (pcf)		76.6

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Unconfined Compressive Strength
ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

Test Hole TH13-01
Sample # T06
Depth (m) 6 - 6.7
Sample Date 15-Nov-13
Test Date 16-Nov-13
Technician Hachem Ahmed

Unconfined Strength

	kPa	ksf
Max q_u	92.5	1.9
Max S_u	46.3	1.0

Specimen Data

Description CLAY - silty, trace silt inclusions (<10mm thick), trace of gravel (<10mm), trace of organic, brown, moist, stiff, high plastic, somewhat slicken sided

Length	150.3	(mm)	**Moisture %**	43%	
Diameter	72.5	(mm)	**Bulk Unit Wt.**	17.2	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	12.0	(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

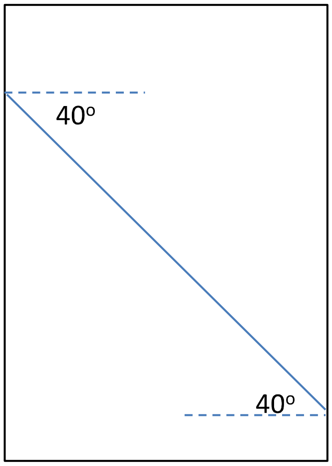
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.70	68.7	1.43
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
1.40	68.7	1.43
1.20	58.9	1.23
1.30	63.8	1.33
1.30	**63.8**	**1.33**

Failure Geometry

Sketch:



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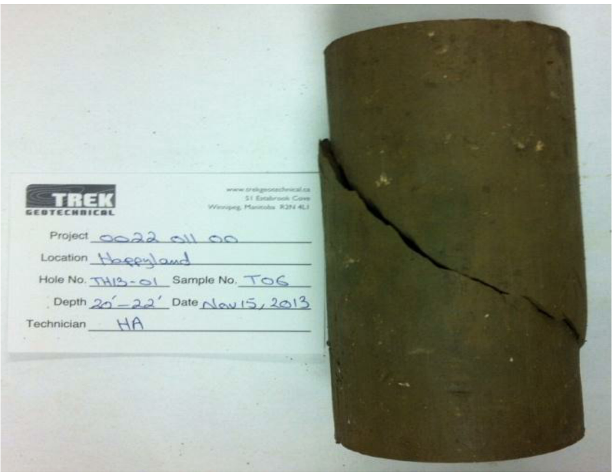
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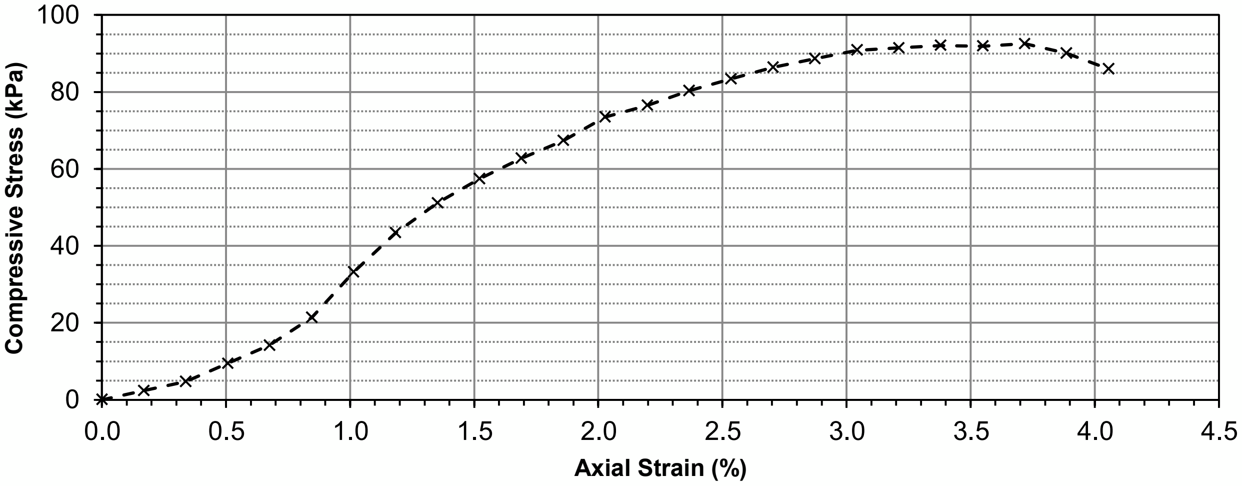
Project 0022 011 00
Location Happyland
Hole No. TH13-01 Sample No. T06
Depth 20'-22' Date Nov 15, 2013
Technician HA

Unconfined Compressive Strength
ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	0.004131	0.0	0.00	0.00
10	3	0.2540	0.17	0.004138	9.8	2.37	1.18
20	6	0.5080	0.34	0.004145	19.6	4.73	2.37
30	12	0.7620	0.51	0.004152	39.3	9.46	4.73
40	18	1.0160	0.68	0.004159	58.9	14.17	7.08
50	27	1.2700	0.84	0.004166	89.0	21.37	10.68
60	42	1.5240	1.01	0.004173	138.5	33.18	16.59
70	55	1.7780	1.18	0.004180	181.4	43.38	21.69
80	65	2.0320	1.35	0.004187	214.3	51.18	25.59
90	73	2.2860	1.52	0.004195	240.7	57.38	28.69
100	80	2.5400	1.69	0.004202	263.8	62.78	31.39
110	86	2.7940	1.86	0.004209	283.5	67.36	33.68
120	94	3.0480	2.03	0.004216	309.9	73.50	36.75
130	98	3.3020	2.20	0.004224	323.1	76.50	38.25
140	103	3.5560	2.37	0.004231	339.8	80.31	40.16
150	107	3.8100	2.53	0.004238	353.3	83.35	41.68
160	111	4.0640	2.70	0.004246	366.8	86.39	43.19
170	114	4.3180	2.87	0.004253	376.9	88.61	44.30
180	117	4.5720	3.04	0.004260	387.0	90.83	45.41
190	118	4.8260	3.21	0.004268	390.3	91.46	45.73
200	119	5.0800	3.38	0.004275	393.7	92.08	46.04
210	119	5.3340	3.55	0.004283	393.7	91.92	45.96
220	120	5.5880	3.72	0.004290	397.0	92.55	46.27
230	117	5.8420	3.89	0.004298	387.0	90.03	45.02

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	112	6.0960	4.0554	0.004305	370.1	85.97	42.99
250	106	6.3500	4.22	0.004313	349.9	81.13	40.56
260	98	6.6040	4.39	0.004321	323.1	74.79	37.39
270	78	6.8580	4.56	0.004328	257.2	59.41	29.71
280	59	7.1120	4.73	0.004336	194.5	44.86	22.43
290	52	7.3660	4.90	0.004344	171.4	39.47	19.73
300	50	7.6200	5.07	0.004351	164.9	37.88	18.94

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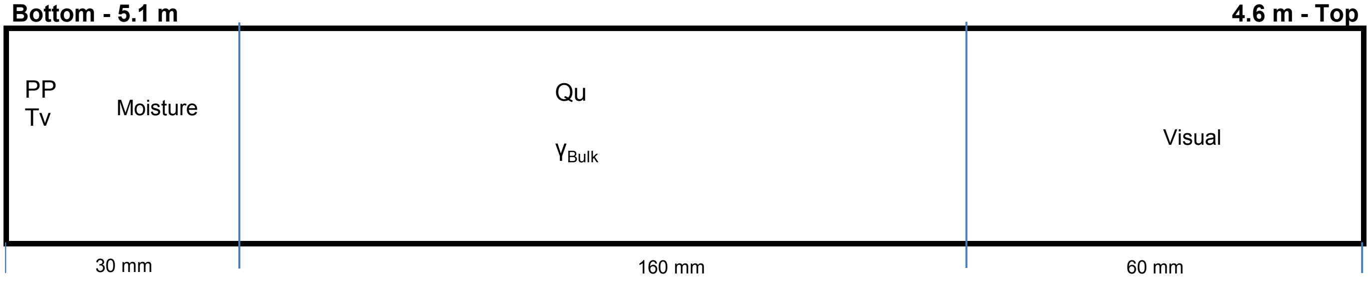
Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement
Test Hole	TH13-01
Sample #	T08
Depth (m)	9.1 - 9.8
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 250



Visual Classification

Material	CLAY
Composition	silty
trace silt inclusion < 10mm diameter	
trace gravel	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plastic
Structure	lamination (<1mm) silt infilled
Gradation	-

Torvane

Reading	0.25
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	24.5

Pocket Penetrometer

Reading	**1**	0.50
	2	0.60
	3	0.65
	Average	0.58
Undrained Shear Strength (kPa)		28.6

Moisture Content

Tare ID	Z59
Mass tare (g)	8.3
Mass wet + tare (g)	323.8
Mass dry + tare (g)	220.2
Moisture %	48.9%

Unit Weight

Bulk Weight (g)		1040.70
Length (mm)	**1**	146.35
	2	146.06
	3	146.38
	4	146.03
Average Length (m)		0.146
Diam. (mm)	**1**	70.97
	2	73.40
	3	73.15
	4	70.79
Average Diameter (m)		0.072
Volume (m^3)		5.97E-04
Bulk Unit Weight (kN/m^3)		17.1
Bulk Unit Weight (pcf)		108.9
Dry Unit Weight (kN/m^3)		11.5
Dry Unit Weight (pcf)		73.1

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement

Test Hole	TH13-01
Sample #	T08
Depth (m)	9.1 - 9.8
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Unconfined Strength

	kPa	ksf
Max q_u	62.2	1.3
Max S_u	31.1	0.6

Specimen Data

Description CLAY - silty, trace of silt inclusion < 10mm, trace of gravel, dark grey, moist, firm, high plastic, lamination (<1mm) silt infilled

Length	146.2	(mm)
Diameter	72.1	(mm)
L/D Ratio	2.0	
Initial Area	0.00408	(m^2)
Load Rate	1.00	(%/min)

Moisture %	49%	
Bulk Unit Wt.	17.1	(kN/m^3)
Dry Unit Wt.	11.5	(kN/m^3)
Liquid Limit	-	
Plastic Limit	-	
Plasticity Index	-	

Undrained Shear Strength Tests

Torvane

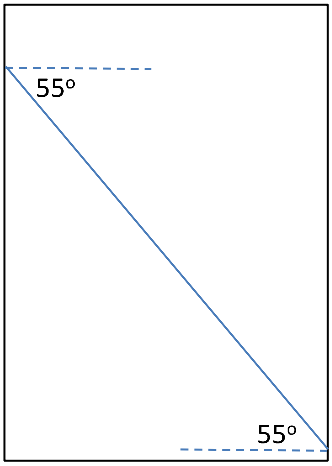
Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.25	24.5	0.51
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.50	24.5	0.51
0.60	29.4	0.61
0.65	31.9	0.67
0.58	**28.6**	**0.60**

Failure Geometry

Sketch:



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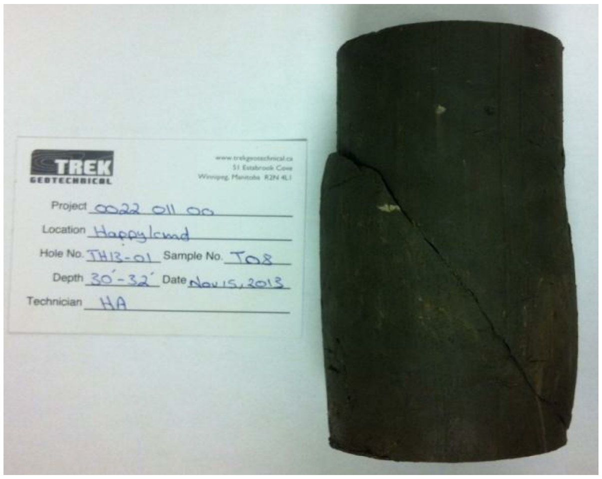


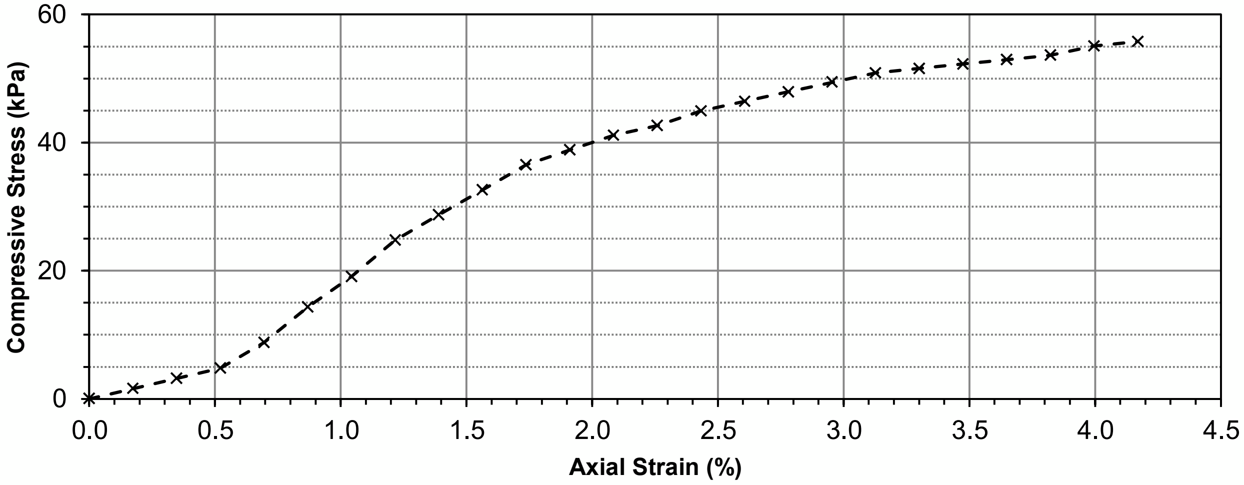
Project 0022 011 00
Location Happyland
Hole No. TH13-01 Sample No. T08
Depth 30′-32′ Date Nov 15, 2013
Technician HA

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	0.004080	0.0	0.00	0.00
10	2	0.2540	0.17	0.004087	6.5	1.60	0.80
20	4	0.5080	0.35	0.004095	13.1	3.19	1.60
30	6	0.7620	0.52	0.004102	19.6	4.78	2.39
40	11	1.0160	0.69	0.004109	36.0	8.76	4.38
50	18	1.2700	0.87	0.004116	58.9	14.32	7.16
60	24	1.5240	1.04	0.004123	78.6	19.07	9.54
70	31	1.7780	1.22	0.004131	102.2	24.75	12.37
80	36	2.0320	1.39	0.004138	118.7	28.68	14.34
90	41	2.2860	1.56	0.004145	135.2	32.61	16.31
100	46	2.5400	1.74	0.004152	151.7	36.53	18.26
110	49	2.7940	1.91	0.004160	161.6	38.84	19.42
120	52	3.0480	2.08	0.004167	171.4	41.14	20.57
130	54	3.3020	2.26	0.004175	178.0	42.64	21.32
140	57	3.5560	2.43	0.004182	187.9	44.94	22.47
150	59	3.8100	2.61	0.004189	194.5	46.43	23.22
160	61	4.0640	2.78	0.004197	201.1	47.92	23.96
170	63	4.3180	2.95	0.004204	207.7	49.41	24.70
180	65	4.5720	3.13	0.004212	214.3	50.88	25.44
190	66	4.8260	3.30	0.004220	217.6	51.57	25.79
200	67	5.0800	3.47	0.004227	220.9	52.26	26.13
210	68	5.3340	3.65	0.004235	224.2	52.94	26.47
220	69	5.5880	3.82	0.004242	227.5	53.62	26.81
230	71	5.8420	4.00	0.004250	234.1	55.08	27.54

Unconfined Compressive Strength
ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	72	6.0960	4.1695	0.004258	237.4	55.76	27.88
250	73	6.3500	4.34	0.004266	240.7	56.43	28.21
260	74	6.6040	4.52	0.004273	244.0	57.10	28.55
270	74	6.8580	4.69	0.004281	244.0	56.99	28.50
280	75	7.1120	4.86	0.004289	247.3	57.65	28.83
290	76	7.3660	5.04	0.004297	250.6	58.32	29.16
300	77	7.6200	5.21	0.004305	253.9	58.97	29.49
310	78	7.8740	5.39	0.004313	257.2	59.63	29.81
320	78	8.1280	5.56	0.004320	257.2	59.52	29.76
330	79	8.3820	5.73	0.004328	260.4	60.17	30.09
340	79	8.6360	5.91	0.004336	260.4	60.06	30.03
350	80	8.8900	6.08	0.004344	263.8	60.72	30.36
360	81	9.1440	6.25	0.004352	267.1	61.36	30.68
370	81	9.3980	6.43	0.004361	267.1	61.25	30.62
380	81	9.6520	6.60	0.004369	267.1	61.13	30.57
390	82	9.9060	6.78	0.004377	270.4	61.77	30.89
400	82	10.1600	6.95	0.004385	270.4	61.66	30.83
410	82	10.4140	7.12	0.004393	270.4	61.54	30.77
420	83	10.6680	7.30	0.004401	273.7	62.17	31.09
430	83	10.9220	7.47	0.004410	273.7	62.06	31.03
440	83	11.1760	7.64	0.004418	273.7	61.94	30.97
450	83	11.4300	7.82	0.004426	273.7	61.82	30.91
460	83	11.6840	7.99	0.004435	273.7	61.71	30.85
470	83	11.9380	8.17	0.004443	273.7	61.59	30.80
480	83	12.1920	8.34	0.004451	273.7	61.47	30.74
490	83	12.4460	8.51	0.004460	273.7	61.36	30.68
500	83	12.7000	8.69	0.004468	273.7	61.24	30.62
510	83	12.9540	8.86	0.004477	273.7	61.13	30.56
520	82	13.2080	9.03	0.004485	270.4	60.28	30.14
530	82	13.4620	9.21	0.004494	270.4	60.16	30.08
540	82	13.7160	9.38	0.004503	270.4	60.04	30.02
550	81	13.9700	9.56	0.004511	267.1	59.20	29.60
560	81	14.2240	9.73	0.004520	267.1	59.09	29.54
570	80	14.4780	9.90	0.004529	263.8	58.25	29.12
580	80	14.7320	10.08	0.004537	263.8	58.13	29.07
590	79	14.9860	10.25	0.004546	260.4	57.29	28.64
600	79	15.2400	10.42	0.004555	260.4	57.18	28.59
620	77	15.7480	10.77	0.004573	253.9	55.51	27.76
640	75	16.2560	11.12	0.004591	247.3	53.86	26.93
660	74	16.7640	11.47	0.004609	243.9850	52.94	26.47
680	72	17.2720	11.81	0.004627	237.4016	51.31	25.65
700	70	17.7800	12.16	0.004645	230.7737	49.68	24.84

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Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement
Test Hole	TH13-01
Sample #	T10
Depth (m)	10.7 - 11.3
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 250

Bottom - 5.1 m **4.6 m - Top**

PP Tv Moisture Visual	Qu γ_{Bulk}		Visual
110 mm	200 mm	200 mm	90 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusion < 10mm diameter	
trace gravel	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plastic
Structure	lamination (<1mm) silt infilled
Gradation	-

Moisture Content

Tare ID	Z56
Mass tare (g)	8.4
Mass wet + tare (g)	364.8
Mass dry + tare (g)	239.2
Moisture %	54.4%

Unit Weight

Bulk Weight (g)		1110.60
Length (mm)	**1**	152.43
	2	152.51
	3	152.56
	4	152.57
Average Length (m)		0.153
Diam. (mm)	**1**	71.95
	2	72.20
	3	72.59
	4	72.43
Average Diameter (m)		0.072
Volume (m^3)		6.26E-04
Bulk Unit Weight (kN/m^3)		17.4
Bulk Unit Weight (pcf)		110.8
Dry Unit Weight (kN/m^3)		11.3
Dry Unit Weight (pcf)		71.7

Torvane

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

Pocket Penetrometer

Reading	**1**	0.70
	2	0.75
	3	0.80
	Average	0.75
Undrained Shear Strength (kPa)		36.8

Unconfined Compressive Strength

ASTM D2166

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement

Test Hole	TH13-01
Sample #	T10
Depth (m)	10.7 - 11.3
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Unconfined Strength

	kPa	**ksf**
Max q_u	94.6	2.0
Max S_u	47.3	1.0

Specimen Data

Description CLAY - silty, trace of silt inclusion < 10mm, trace of gravel, dark grey, moist, firm, high plastic, lamination (<1mm) silt infilled

Length	152.5	(mm)	**Moisture %**	54%	
Diameter	72.3	(mm)	**Bulk Unit Wt.**	17.4	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	11.3	(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**
0.35	34.3	0.72
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.70	34.3	0.72
0.75	36.8	0.77
0.80	39.2	0.82
0.75	**36.8**	**0.77**

Failure Geometry

Sketch:

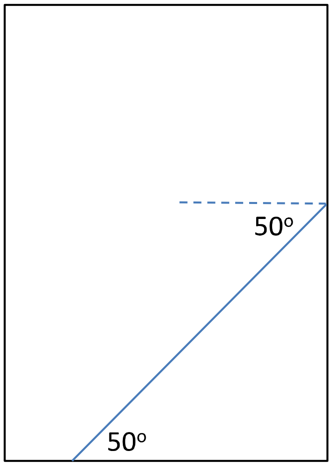


Photo:

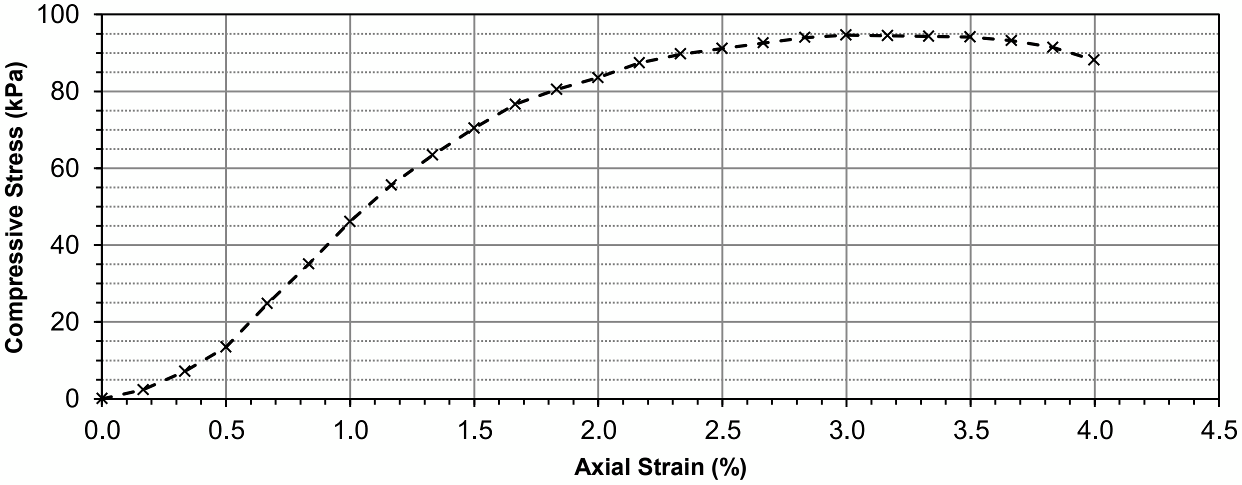
Project 0022 011 00
Location Happyland
Hole No. TH13-01 Sample No. T10
Depth 35'-37' Date Nov 15, 201
Technician HA

Unconfined Compressive Strength

ASTM D2166

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement

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	0.004105	0.0	0.00	0.00
10	3	0.2540	0.17	0.004111	9.8	2.38	1.19
20	9	0.5080	0.33	0.004118	29.4	7.15	3.57
30	17	0.7620	0.50	0.004125	55.7	13.49	6.75
40	31	1.0160	0.67	0.004132	102.2	24.74	12.37
50	44	1.2700	0.83	0.004139	145.1	35.05	17.52
60	58	1.5240	1.00	0.004146	191.2	46.12	23.06
70	70	1.7780	1.17	0.004153	230.8	55.57	27.78
80	80	2.0320	1.33	0.004160	263.8	63.41	31.70
90	89	2.2860	1.50	0.004167	293.4	70.42	35.21
100	97	2.5400	1.67	0.004174	319.8	76.62	38.31
110	102	2.7940	1.83	0.004181	336.4	80.46	40.23
120	106	3.0480	2.00	0.004188	349.9	83.54	41.77
130	111	3.3020	2.16	0.004195	366.8	87.42	43.71
140	114	3.5560	2.33	0.004203	376.9	89.67	44.84
150	116	3.8100	2.50	0.004210	383.6	91.11	45.56
160	118	4.0640	2.66	0.004217	390.3	92.56	46.28
170	120	4.3180	2.83	0.004224	397.0	93.99	47.00
180	121	4.5720	3.00	0.004231	400.4	94.63	47.32
190	121	4.8260	3.16	0.004239	400.4	94.47	47.23
200	121	5.0800	3.33	0.004246	400.4	94.31	47.15
210	121	5.3340	3.50	0.004253	400.4	94.14	47.07
220	120	5.5880	3.66	0.004261	397.0	93.19	46.59
230	118	5.8420	3.83	0.004268	390.3	91.45	45.73

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	114	6.0960	3.9969	0.004276	376.9	88.14	44.07
250	109	6.3500	4.16	0.004283	360.0	84.05	42.03
260	102	6.6040	4.33	0.004290	336.4	78.41	39.21
270	95	6.8580	4.50	0.004298	313.2	72.87	36.44
280	89	7.1120	4.66	0.004305	293.4	68.16	34.08
290	82	7.3660	4.83	0.004313	270.4	62.69	31.34
300	76	7.6200	5.00	0.004321	250.6	58.00	29.00

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Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement
Test Hole	TH13-02
Sample #	T19
Depth (m)	3.0 - 3.7
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 580

Bottom - 5.1 m **4.6 m - Top**

PP Tv Moisture Visual	Qu γ_{Bulk}	
140 mm	160 mm	280 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusion < 5mm diameter	
trace precipitation (sulphates <5%)	
trace oxidation	
Color	brown
Moisture	moist
Consistency	stiff
Plasticity	high plastic
Structure	lamination (<1mm)
Gradation	-

Torvane

Reading	0.70
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	68.7

Pocket Penetrometer

Reading	**1**	1.80
	2	1.85
	3	1.80
	Average	1.82
Undrained Shear Strength (kPa)		89.1

Moisture Content

Tare ID	Z100
Mass tare (g)	8.3
Mass wet + tare (g)	380.5
Mass dry + tare (g)	259.5
Moisture %	48.2%

Unit Weight

Bulk Weight (g)		1085.10
Length (mm)	**1**	151.49
	2	151.58
	3	151.61
	4	151.50
Average Length (m)		0.152
Diam. (mm)	**1**	71.83
	2	71.74
	3	72.17
	4	72.68
Average Diameter (m)		0.072
Volume (m^3)		6.19E-04
Bulk Unit Weight (kN/m^3)		17.2
Bulk Unit Weight (pcf)		109.5
Dry Unit Weight (kN/m^3)		11.6
Dry Unit Weight (pcf)		73.9

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement

Test Hole	TH13-02
Sample #	T19
Depth (m)	3.0 - 3.7
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Unconfined Strength

	kPa	ksf
Max q_u	114.7	2.4
Max S_u	57.3	1.2

Specimen Data

Description CLAY - silty, trace of silt inclusion < 5mm diameter, trace of precipitation (sulphates <5%), trace of oxidation, brown, moist, stiff, high plastic, lamination (<1mm)

Length	151.5	(mm)	**Moisture %**	48%	
Diameter	72.1	(mm)	**Bulk Unit Wt.**	17.2	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	11.6	(kN/m^3)
Initial Area	0.00408	(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**
0.70	68.7	1.43
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
1.80	88.3	1.84
1.85	90.7	1.90
1.80	88.3	1.84
1.82	**89.1**	**1.86**

Failure Geometry

Sketch:

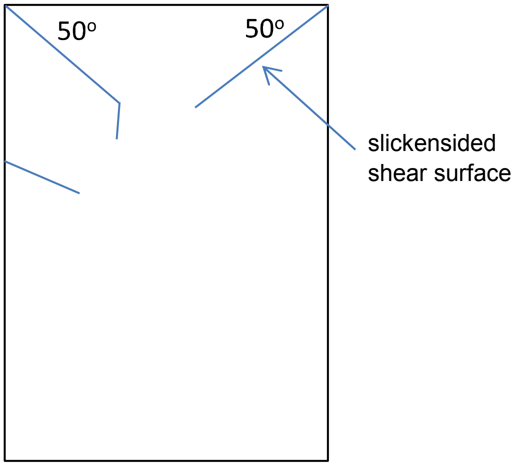


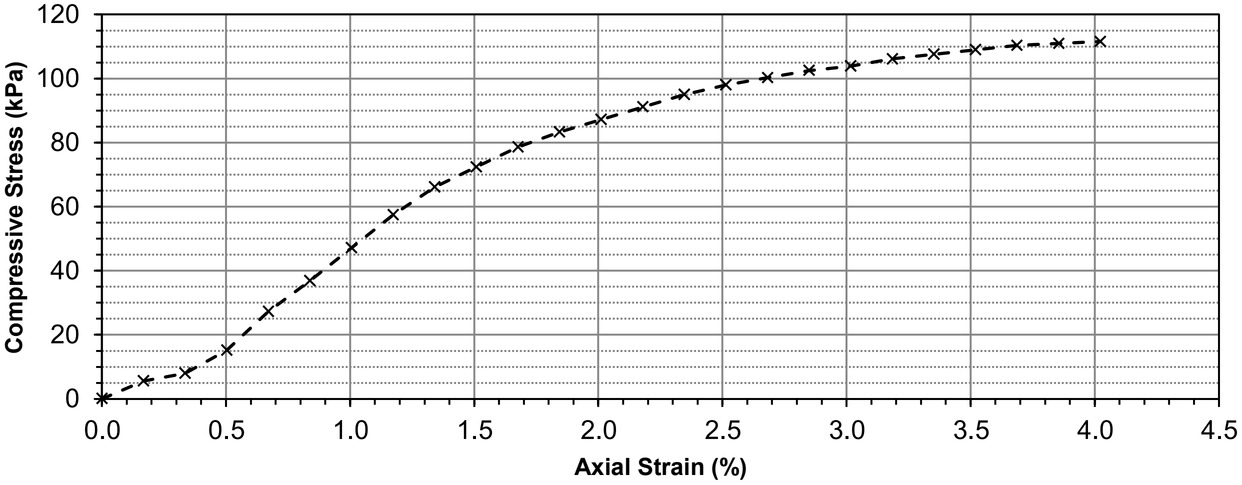
Photo:

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	7	0.2540	0.17	0.004090	22.9	5.60	2.80
20	10	0.5080	0.34	0.004097	32.7	7.98	3.99
30	19	0.7620	0.50	0.004104	62.2	15.16	7.58
40	34	1.0160	0.67	0.004111	112.1	27.27	13.63
50	46	1.2700	0.84	0.004118	151.7	36.84	18.42
60	59	1.5240	1.01	0.004125	194.5	47.16	23.58
70	72	1.7780	1.17	0.004132	237.4	57.46	28.73
80	83	2.0320	1.34	0.004139	273.7	66.12	33.06
90	91	2.2860	1.51	0.004146	300.0	72.37	36.18
100	99	2.5400	1.68	0.004153	326.4	78.60	39.30
110	105	2.7940	1.84	0.004160	346.6	83.31	41.65
120	110	3.0480	2.01	0.004167	363.4	87.20	43.60
130	115	3.3020	2.18	0.004174	380.2	91.09	45.54
140	120	3.5560	2.35	0.004182	397.0	94.95	47.48
150	124	3.8100	2.51	0.004189	410.5	98.01	49.00
160	127	4.0640	2.68	0.004196	420.6	100.25	50.12
170	130	4.3180	2.85	0.004203	430.7	102.48	51.24
180	132	4.5720	3.02	0.004210	437.5	103.90	51.95
190	135	4.8260	3.18	0.004218	447.6	106.12	53.06
200	137	5.0800	3.35	0.004225	454.3	107.53	53.76
210	139	5.3340	3.52	0.004232	461.1	108.94	54.47
220	141	5.5880	3.69	0.004240	467.8	110.33	55.17
230	142	5.8420	3.85	0.004247	471.2	110.94	55.47

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	143	6.0960	4.0226	0.004255	474.5	111.54	55.77
250	144	6.3500	4.19	0.004262	477.9	112.12	56.06
260	145	6.6040	4.36	0.004269	481.3	112.72	56.36
270	146	6.8580	4.53	0.004277	484.6	113.31	56.66
280	147	7.1120	4.69	0.004284	488.0	113.89	56.95
290	148	7.3660	4.86	0.004292	491.4	114.48	57.24
300	148	7.6200	5.03	0.004300	491.4	114.28	57.14
310	148	7.8740	5.20	0.004307	491.4	114.08	57.04
320	149	8.1280	5.36	0.004315	494.7	114.66	57.33
330	149	8.3820	5.53	0.004322	494.7	114.46	57.23
340	149	8.6360	5.70	0.004330	494.7	114.25	57.13
350	149	8.8900	5.87	0.004338	494.7	114.05	57.02
360	149	9.1440	6.03	0.004346	494.7	113.85	56.92
370	149	9.3980	6.20	0.004353	494.7	113.64	56.82
380	150	9.6520	6.37	0.004361	498.1	114.21	57.10
390	150	9.9060	6.54	0.004369	498.1	114.00	57.00
400	150	10.1600	6.70	0.004377	498.1	113.80	56.90
410	151	10.4140	6.87	0.004385	501.4	114.36	57.18
420	151	10.6680	7.04	0.004393	501.4	114.16	57.08
430	151	10.9220	7.21	0.004401	501.4	113.95	56.98
440	151	11.1760	7.37	0.004409	501.4	113.75	56.87
450	151	11.4300	7.54	0.004416	501.4	113.54	56.77
460	151	11.6840	7.71	0.004425	501.4	113.33	56.67
470	151	11.9380	7.88	0.004433	501.4	113.13	56.56
480	151	12.1920	8.05	0.004441	501.4	112.92	56.46
490	150	12.4460	8.21	0.004449	498.1	111.96	55.98
500	150	12.7000	8.38	0.004457	498.1	111.75	55.88
510	149	12.9540	8.55	0.004465	494.7	110.80	55.40
520	148	13.2080	8.72	0.004473	491.4	109.84	54.92
530	147	13.4620	8.88	0.004481	488.0	108.89	54.44
540	145	13.7160	9.05	0.004490	481.3	107.19	53.59
550	144	13.9700	9.22	0.004498	477.9	106.24	53.12
560	142	14.2240	9.39	0.004506	471.2	104.55	52.28
570	140	14.4780	9.55	0.004515	464.4	102.86	51.43
580	138	14.7320	9.72	0.004523	457.7	101.19	50.59
590	136	14.9860	9.89	0.004531	451.0	99.52	49.76
600	133	15.2400	10.06	0.004540	440.8	97.10	48.55
620	127	15.7480	10.39	0.004557	420.6	92.30	46.15
640	123	16.2560	10.73	0.004574	407.1	89.01	44.51
660	118	16.7640	11.06	0.004591	390.3314	85.02	42.51
680	114	17.2720	11.40	0.004609	376.8533	81.77	40.89
700	110	17.7800	11.73	0.004626	363.3752	78.55	39.27

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Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement
Test Hole	TH13-02
Sample #	T21
Depth (m)	4.6 - 5.2
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 400

Bottom - 5.1 m		**4.6 m - Top**
Visual Moisture	PP Tv	γ_{Bulk}
150 mm	150 mm	100 mm

Visual Classification

Material	CLAY
Composition	silty
trace precipitation (sulphates <5%)	
trace gravel	
trace organics	
trace oxidation	
Color	brown
Moisture	moist
Consistency	stiff
Plasticity	high plastic
Structure	lamination <1mm silt infilled
Gradation	-

Torvane

Reading	0.75
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	73.6

Pocket Penetrometer

Reading	**1**	2.10
	2	1.80
	3	1.90
	Average	1.93
Undrained Shear Strength (kPa)		94.8

Moisture Content

Tare ID	P31
Mass tare (g)	8.3
Mass wet + tare (g)	446.5
Mass dry + tare (g)	315.3
Moisture %	42.7%

Unit Weight

Bulk Weight (g)		165.70
Length (mm)	**1**	23.88
	2	23.50
	3	22.84
	4	23.78
Average Length (m)		0.024
Diam. (mm)	**1**	70.92
	2	72.83
	3	73.36
	4	70.91
Average Diameter (m)		0.072
Volume (m^3)		9.57E-05
Bulk Unit Weight (kN/m^3)		17.0
Bulk Unit Weight (pcf)		108.1
Dry Unit Weight (kN/m^3)		11.9
Dry Unit Weight (pcf)		75.7

Shelby Tube Visual

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement

Test Hole	TH13-01
Sample #	T26
Depth (m)	12.2 - 12.8
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Tube Extraction

Recovery (mm) 660

Bottom - 5.1 m **4.6 m - Top**

PP Tv Visual Moisture	Qu γ_{Bulk}	
130 mm	190 mm	340 mm

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions <10mm diameter	
trace gravel	
Color	dark grey
Moisture	moist
Consistency	firm
Plasticity	high plastic
Structure	-
Gradation	-

Torvane

Reading	0.38
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	37.3

Pocket Penetrometer

Reading	**1**	0.75
	2	0.80
	3	0.80
	Average	0.78
Undrained Shear Strength (kPa)		38.4

Moisture Content

Tare ID	N53
Mass tare (g)	8.4
Mass wet + tare (g)	392.5
Mass dry + tare (g)	247
Moisture %	61.0%

Unit Weight

Bulk Weight (g)		1076.90
Length (mm)	**1**	150.49
	2	150.29
	3	149.76
	4	149.69
Average Length (m)		0.150
Diam. (mm)	**1**	72.10
	2	71.82
	3	71.39
	4	71.93
Average Diameter (m)		0.072
Volume (m^3)		6.08E-04
Bulk Unit Weight (kN/m^3)		17.4
Bulk Unit Weight (pcf)		110.6
Dry Unit Weight (kN/m^3)		10.8
Dry Unit Weight (pcf)		68.7

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Unconfined Compressive Strength

ASTM D2166

Project No.	0022 011 00
Client	Dillon Consulting Ltd.
Project	Happyland Park Culvert Replacement

Test Hole	TH13-01
Sample #	T26
Depth (m)	12.2 - 12.8
Sample Date	15-Nov-13
Test Date	16-Nov-13
Technician	Hachem Ahmed

Unconfined Strength

	kPa	**ksf**
Max q_u	87.5	1.8
Max S_u	43.7	0.9

Specimen Data

Description CLAY - silty, trace of silt inclusions <10mm diameter, trace of gravel, dark grey, moist, firm, high plastic

Length	150.1	(mm)	**Moisture %**	61%	
Diameter	71.8	(mm)	**Bulk Unit Wt.**	17.4	(kN/m^3)
L/D Ratio	2.1		**Dry Unit Wt.**	10.8	(kN/m^3)
Initial Area	0.00405	(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	37.3	0.78
Vane Size		
m		

Pocket Penetrometer

Reading	**Undrained Shear Strength**	
tsf	**kPa**	**ksf**
0.75	36.8	0.77
0.80	39.2	0.82
0.80	39.2	0.82
0.78	**38.4**	**0.80**

Failure Geometry

Sketch:

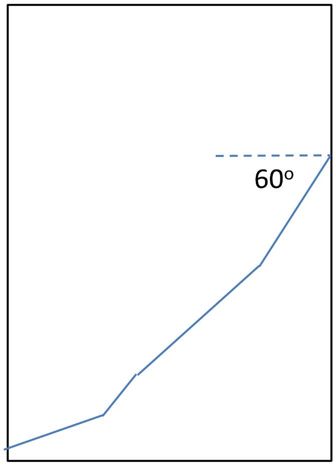
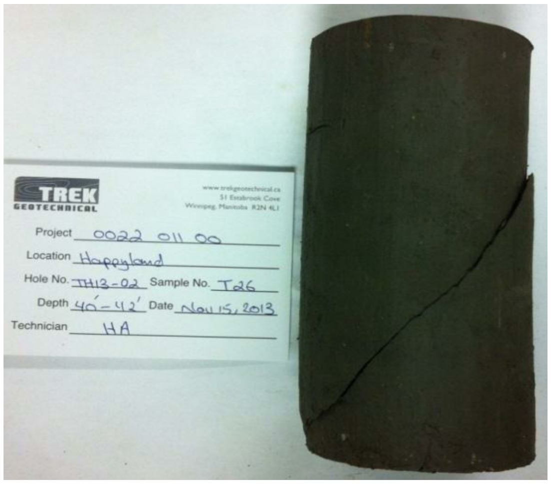


Photo:



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Project 0022 011 00
Location Happyland
Hole No. TH13-02 Sample No. T26
Depth 40′-42′ Date Nov 15, 2013
Technician HA

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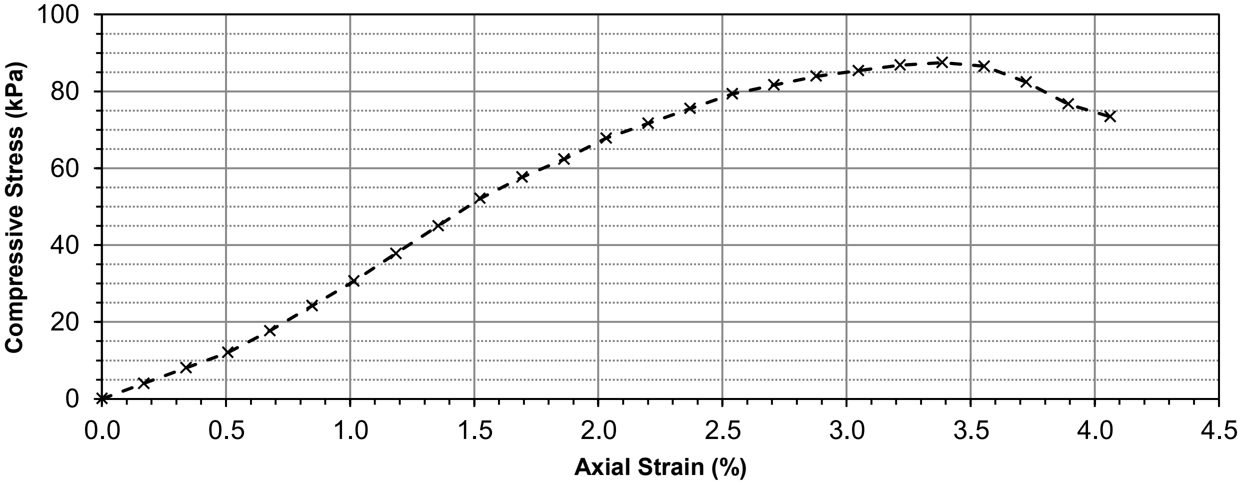
Unconfined Compressive Strength
ASTM D2166

Project No. 0022 011 00
Client Dillon Consulting Ltd.
Project Happyland Park Culvert Replacement

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	0.004050	0.0	0.00	0.00
10	5	0.2540	0.17	0.004057	16.3	4.03	2.01
20	10	0.5080	0.34	0.004064	32.7	8.05	4.02
30	15	0.7620	0.51	0.004071	49.1	12.06	6.03
40	22	1.0160	0.68	0.004078	72.1	17.67	8.84
50	30	1.2700	0.85	0.004085	98.9	24.22	12.11
60	38	1.5240	1.02	0.004092	125.3	30.63	15.31
70	47	1.7780	1.18	0.004099	155.0	37.81	18.91
80	56	2.0320	1.35	0.004106	184.6	44.97	22.49
90	65	2.2860	1.52	0.004113	214.3	52.11	26.06
100	72	2.5400	1.69	0.004120	237.4	57.62	28.81
110	78	2.7940	1.86	0.004127	257.2	62.31	31.16
120	85	3.0480	2.03	0.004134	280.2	67.79	33.89
130	90	3.3020	2.20	0.004141	296.7	71.66	35.83
140	95	3.5560	2.37	0.004148	313.2	75.50	37.75
150	100	3.8100	2.54	0.004156	329.7	79.34	39.67
160	103	4.0640	2.71	0.004163	339.8	81.63	40.81
170	106	4.3180	2.88	0.004170	349.9	83.91	41.95
180	108	4.5720	3.05	0.004177	356.7	85.38	42.69
190	110	4.8260	3.22	0.004185	363.4	86.84	43.42
200	111	5.0800	3.39	0.004192	366.8	87.49	43.75
210	110	5.3340	3.55	0.004199	363.4	86.53	43.27
220	105	5.5880	3.72	0.004207	346.6	82.38	41.19
230	98	5.8420	3.89	0.004214	323.1	76.68	38.34

Unconfined Compressive Strength

ASTM D2166

Project No. 0022 011 00
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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	94	6.0960	4.0624	0.004222	309.9	73.41	36.71
250	88	6.3500	4.23	0.004229	290.2	68.61	34.31
260	84	6.6040	4.40	0.004236	276.9	65.37	32.69
270	81	6.8580	4.57	0.004244	267.1	62.93	31.46
280	78	7.1120	4.74	0.004252	257.2	60.48	30.24
290	75	7.3660	4.91	0.004259	247.3	58.06	29.03
300	73	7.6200	5.08	0.004267	240.7	56.41	28.21