APPENDIX ‘C’

SUBSURFACE INVESTIGATION AND ENGINEERING REPORT ON THE PROPOSED PIPE/TRACK CROSSING

Dillon Consulting Ltd.

2006 Regional Street Renewal Program

Panet Road Reconstruction – New LDS 900 mm ϕ Crossing within CP Right-of-Way at Callsbeck Avenue

Subsurface Investigation and Engineering Report on the Proposed Pipe/ Track Crossing

Prepared by:
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Project Number: F504 023 00 (4.6.1)

November 2007

UMA | AECOM

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November 30, 2007

UMA Project No: F504 023 00 (4.6.1)

Mr. David Wiebe, P. Eng.
Dillon Consulting Ltd.
200 – 895 Waverley Street
Winnipeg, Manitoba
R3T 5P4

Dear Mr. Wiebe:

Reference **2006 Regional Street Renewal Program**
Panet Road Reconstruction – New LDS 900 mm ϕ
Crossing within CP Right-of-Way at Callsbeck Avenue
Subsurface Investigation and Engineering Report on the
Proposed Pipe/Track Crossing

UMA Engineering Ltd. is pleased to submit our report for the above noted project.

Should you have any questions or require additional information, please contact Mr. Faris Khalil, P.Eng. directly.

Yours truly,

UMA Engineering Ltd.

R.V. Typliski

Ron Typliski, P.Eng.
Regional Manager
Earth and Environmental
FK/dh

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1.0 Introduction

As part of the Panet Road Reconstruction Project a new 900 mm diameter concrete sewer pipe will be installed using a trenchless construction method to cross under the CP Keewatin Subdivision at Callsbeck Avenue. The pipe invert will be about 6.8 m below base of rail (BOR) at the crossing location.

This report summarizes the results of our geotechnical investigation and provides a geotechnical assessment of the potential impact of the proposed pipe installation on the existing track. It has been prepared according to the principles outlined in "CPR Geotechnical Protocol for Pipeline and Utility Installations within Railway Right of Way" dated March 2006 to address CP's needs and to be part of CP's approval process. An intermediate review process is identified based on the proposed work conditions as outlined in the above Protocol

2.0 Geotechnical Investigation

Three test holes were drilled on June 21st, 2007 at the locations shown on the test holes location plan, Drawing 01in Appendix A. Test Hole (TH) 07-01 was drilled on the south side of the track to a depth of 16.8 m below ground surface to identify the till contact and enable piezometer installation in the till unit. TH's 07-02 and 07-03 were drilled on the north side of the track to 10.7m below ground surface.

Drilling was carried out by Paddock Drilling Ltd. using an RM30 drill rig equipped with 125 mm diameter solid stem augers. Disturbed soil samples from the auger cuttings and relatively undisturbed (Shelby tube) were collected at regular intervals in each test hole. All soils observed during drilling were logged and visually classified on site by UMA personnel. Two standpipe piezometers equipped with Casagrande tips were installed in TH07-01 to facilitate ground water measurement in the clay and till units. One standpipe piezometer was installed in TH07-03 in the clay unit. Piezometers construction details are shown on the test hole logs in Appendix B.

Soil samples recovered during drilling were transported to UMA's Materials Testing Laboratory in Winnipeg for further visual examination and testing. Laboratory testing consisted of determination of moisture contents and Atterberg limits, and grain size distribution. Undrained shear strengths and unit weights were determined on all Shelby tube samples. Shear strength testing consisted of unconfined compression tests and Lab vane, Torvane and pocket penetrometer measurements.

A detailed test hole log has been prepared for each test hole to record the description and the relative position of the various soil strata, location of samples obtained, field and laboratory test results, and other pertinent information. The test hole logs are provided in Appendix B.

3.0 Subsurface Conditions

3.1 Soil Profile

In descending order, the general soil profile is as follows:

- Topsoil
- Glacio-Lacustrine Clay
- Glacial Till

These soils are described as follows:

Topsoil

Topsoil was encountered at the ground surface at all test holes. The topsoil is about 150 mm thick, black, moist and contains trace organics.

Glacio-Lucstrine Clay

High plastic glacio-lacustrine silty clay was encountered beneath the topsoil. In TH07-01 the clay extended to the depth of 13.4 m below ground surface. TH 07-02 and 07-03 were terminated in the clay unit at 10.7m below ground surface. The clay is generally stiff, moist and brown / dark grey. Moisture contents typically fall close to an average value of 43 percent. The clay is classified as highly plastic based on an average liquid limit and plasticity index of 90 and 63 percent, respectively. Moisture contents fall closer to plastic limit and the liquidity index is determined to be between 0.5 and 0.7 indicating the firm to soft nature of the clay. Undrained shear strengths, as measured from unconfined compression tests, range from 20 to 70 kPa. Bulk unit weights of the clay range from 16.4 to 17.4 kN/m^3 .

A thin silt layer approximately 0.3 m thick was encountered in all test hole at elevations ranging from 230 to 230.7 m. The silt is brown, soft and low plastic. Moisture contents range from 23 to 29 percent. A profile of the laboratory test results illustrating the location (depth) of the proposed pipe is shown on Figure 01.The recommended design values (discussed under Sections 6.0 and 7.0 of this report) are also illustrated. Clay and silt deposits up to 0.6m thick were encountered below the silt layer in TH 07-02 and 07-03 between elevations 229.3 to 230.3. The top 0.3 to 1.2 m of the clay unit is predominantly black and containing trace rootlets and decomposed plant material.

Glacial Till

Glacial till was encountered beneath the clay in TH 07-01 at 13.4 m below ground surface at elevation 218.8 m. The upper portion of the unit is loose to compact silt till, while more granular material was encountered in the lower dense portion of the till unit. Moisture contents are consistent through the depth drilled and fall close to 11 percent.

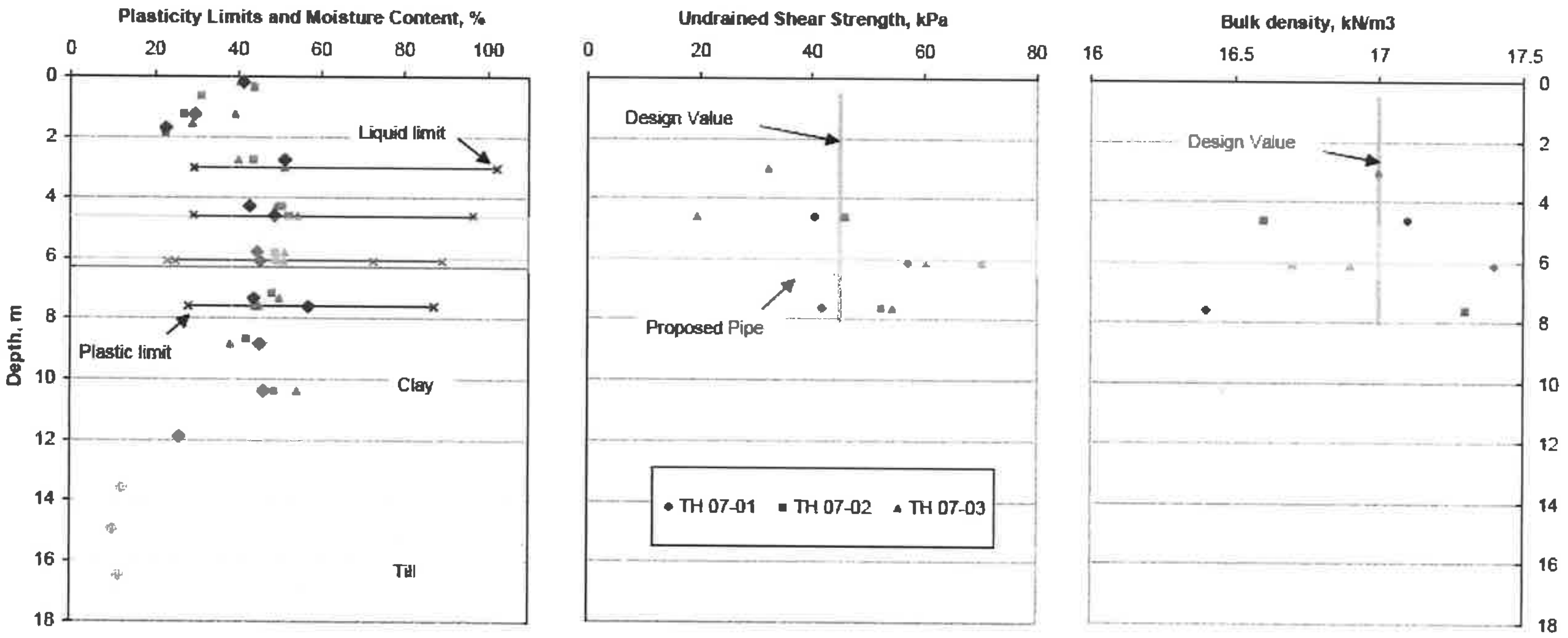


Figure 01: Profile of Laboratory Test Results

3.2 Groundwater Conditions

Groundwater levels in till and clay were measured using the standpipe piezometers installed in TH 07-01 and 07-03. The groundwater level in the till was measured at 9.85 below ground surface or at elevation 222.39. The groundwater level in the clay was measured at 4.55 and 1.90 m below ground surface or at elevation 227.69 and 229.83 in TH 07-01 and 07-03, respectively. Groundwater levels may not have stabilized over the monitoring period and could vary seasonally or as a result of construction activities.

4.0 Pipe Crossing Plans

As stipulated by CP geotechnical protocol, the stratigraphic section on Drawing 02 in Appendix A was prepared to depict the pipe/track crossing plan and cross section and longitudinal section along the track. The Drawing was prepared in accordance with Item 5.4 of the above protocol and shows the details requested where applicable.

5.0 Pipe Installation Methods

There are two methods of pipe jacking practiced locally. One utilizes the Akkerman system while the other is a variation of the Atkins coring system. Both methods follow a similar construction approach and result in similar ground response. A brief description for each method is provided herein:

5.1 Akkerman System

The Akkerman installation method requires a jacking shaft from which the pipe installation starts and a receiving shaft at the end of the pipe length to retrieve the Tunnel Boring Machine (TBM) which would be used to excavate underground along the pipe alignment. The TBM has a rotating cutterhead that rotates and excavates the soil which comes inside the cutting head. The spoil is transferred to the rear of the shield through conveyers which dump it into muck carts or conveys it out of the tunnel or the pipe being installed. Thrust power of hydraulic jacks is utilized to force the TBM and the following string of pipes forward. The hydraulic pressures overcome face resistance and friction forces on the exposed surfaces of the shield and installed pipes.

The proposed pipe length of 80 m is within the capability of this system in one jacking phase (single drive). Drive lengths up to 120 m have been successfully achieved in Winnipeg area using this method. However, since the method requires personnel working inside the pipe, the method is limited to man entry size boring. Even though it is theoretically possible for a person to enter a 900 mm diameter bore, it is practically difficult for the person to work in it. Locally, 1050 mm diameter pipes are the minimum size installed using this method and to our knowledge there is no Akkerman system available locally can be used to install the proposed pipe size of 900 mm diameter. Upgrading the proposed pipe size to 1050 mm diameter should be considered if this method is to be utilized.

5.2 Atkins System

The Atkins jacking method is a variation of Atkins traditional coring method. This method requires a shaft on both ends of the pipe length to be installed. Three steel rods are driven through from shaft to shaft along the center of the proposed pipe alignment. A push-pull earth coring knife is attached to the center rod and front cutting and shielding rim is attached to the two outer rods. The first pipe section is placed so that it abuts to the front cutting and shielding rim securely. A pulling and holding rim connected to the outer rods and secured against the back of the pipe section is used to advance the pipe forward. The rods are pulled, or jacked, towards the apposite shaft to move the whole assembly through the soil. The spoil removed from the coring knife as necessary by pushing the knife forward. Once a pipe section is installed, additional section are added and the installation process continued.

This method can be utilized to install the proposed pipe size. However the drive length between shafts is limited to 30 to 35 m. For the proposed pipe length of 80 m, three to four shafts may be required. The shafts should be located as far as practical from the track to protect against potential impact of the excavation. The approximate locations of the access shaft are shown on Drawings 02 in Appendix A. The final location will be determined by the Contractor.

6.0 Geotechnical Concerns and Potential Impacts During Pipe Jacking

6.1 Face Stability

The Face Stability Index, frequently referred to as the overload factor (OF), is the ratio of the difference between the vertical pressure at tunnel axis and the pressure applied to the tunnel face, and the undrained shear strength. In cohesive soils, the tunnel face is considered stable when the index is less than six. While the limiting value of OF=6 represents a threshold of serious problems, a value of OF=5 represents a practical limit below which tunnelling may be carried out without unusual difficulties.

Using the selected design value of 45 kPa for undrained shear strength and 17 kN/m3 for bulk unit weight, the estimated OF is between 2 and 2.5 which suggests that tunnel face stability is satisfactory. However, difficulties in face stability are expected if wet silt layers or seams are encountered within the clay along the pipe alignment.

Caution should be exercised to monitor the face and minimize the time period associated with the tunnelling operations. A contractual requirement for a continuous jacking operations under the track and visual observation of the cuttings to confirm no silt zone has been encountered will allow remedial action to be undertaken in the unlikely event of experiencing face instabilities.

6.2 Ground Subsidence

Like other tunnelling methods, pipe jacking will result in a change in the state of stress in the ground with the corresponding displacements. Ground subsidence can be caused by several factors such as ground loss at the tunnel face, behind the tail of the shield and through the tunnel support or linings. Based on having a stable tunnelling face, the only significant contribution to ground loss is the closure of the over-cut. The over-cut is the annular space between the tunnel boring walls and the installed pipe.

Some degree of ground surface subsidence can be expected from tunneling although in many instances its effects, from a practical perspective are negligible. Empirical methods of predicting settlement due to tunnelling induced ground movements have been used extensively and successfully over the years. Most methods derived for estimating surface or subsurface subsidence are empirical in nature and based on field observations in the UK although the same computational methods have been successfully applied locally. The most common method is estimating the value of (i), a parameter used to define the distance from the tunnel centre line to the point of inflexion of the settlement trough of a normal probability curve as shown in Figure 02. The distribution of the settlements or settlement trough approximates a normal probability distribution function described as:

$S_x = S_{max}\ exp\ [-x^2/2i^2]$ ....Equation 1

where

S_x = surface settlement at a transverse distance (x) from the tunnel centre line
S_{max} = maximum settlement at $x = 0$
i = location of maximum settlement gradient or point of inflexion.

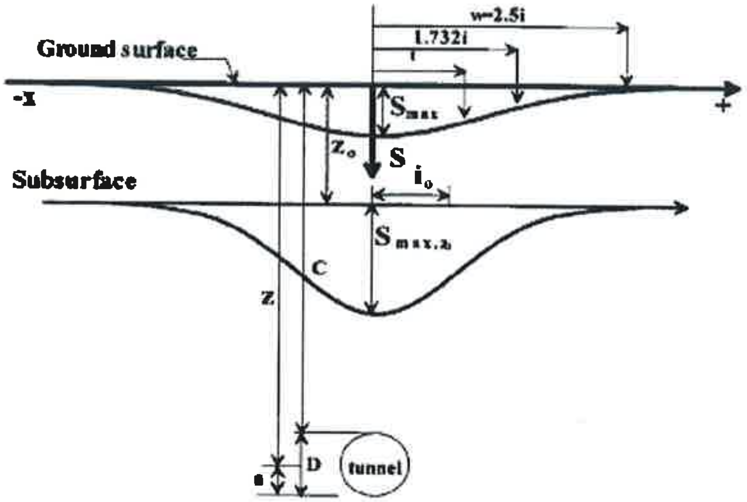


Figure 02: Form of Surface and Subsurface Settlement Trough

Based on Equation 1, the estimated i parameter, width of settlement trough and max settlement at (BOR) elevation and selected subsurface elevations are shown in Table 1. In estimating these values, the volume of settlement trough, per unit length, was considered equal to the ground loss from the closure of 13mm over-cut between the excavated tunnel bore and the outer pipe wall. The over-cut size used in the above estimation is consistent with the local construction practice. As shown in Table 1 subsurface settlement troughs are narrower with larger settlement as compared to surface settlement.

Table 1: Estimated Surface and Subsurface Settlement Trough Parameters

Elevation (m)	i parameter (m)	Total trough width (approx. 5 i) (m)	Max. settlement (mm)
BOR (El. 232.80)	3.82	19.1	5
3.0 m below BOR (EL. 229.80)	2.53	12.7	7
4.5 m below BOR (EL. 228.30)	1.89	9.4	10
5.0 m below BOR (EL. 227.80)	1.67	8.4	11

To put these maximum anticipated values in some perspective they are presented graphically using an exaggerated vertical scale on Figure 03. The maximum estimated ground subsidence at the BOR elevation is in the order of 5 mm and it diminishes to zero across the width of the settlement trough which is estimated to be about 20 meters. The estimated extent and amount of the ground subsidence is not expected to be of concern and unlikely to impose adverse impact on the operation of the existing track. However, continuous monitoring during construction is recommended to monitor actual ground subsidence and protect against development of unanticipated conditions.

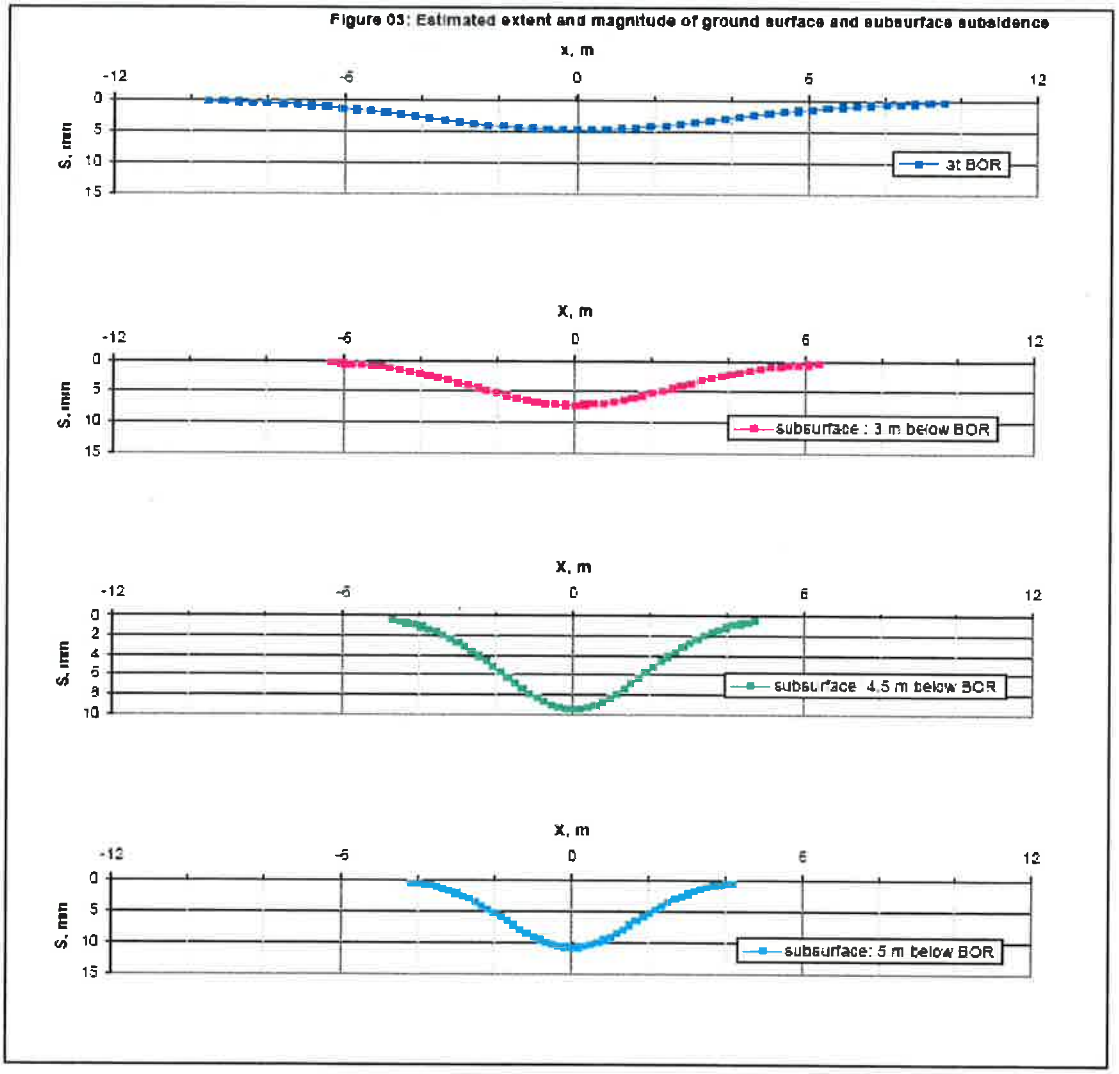


Figure 03: Estimated Extent and Amount of Ground Surface and Subsurface Subsidence

If used, unsupported excavation shall be limited to the top 3m of the clay and can be cut with back slopes not steeper than (1 H: 1 V). If soft zones or perched groundwater are encountered, flatter slopes may be required. The toe of the cut slope should be at least half the depth of the shored excavation from the shoring face. A perimeter ditch should be provided to intercept surface runoff and/or any groundwater from entering the excavation. All excavations should be completed in accordance with Manitoba Workplace Health and Safety Regulations.

The potential of base instability and associated ground displacement adjacent to the excavation must be recognized in excavation design. The factor of safety against base instability should be determined using the equation:

$$F_{sb} = (N_b\, S_u) / \sigma_z$$

Where:

F_{sb} = Factor of Safety with respect to base instability
N_b = stability factor depending on the geometry of the excavation
S_u = Undrained shear strength of the clay below base level
σ_z = Total overburden pressure at base level

A minimum factor of safety of 1.50 is recommended for design purposes using undrained shear strength of 45 kpa and a bulk unit weight of 17 kN/m3 for the clay. The anticipated maximum depth of the access shaft excavation is about 7.5 m. The factor of safety against base instability for a range of excavation dimensions is shown on Figure 06. The calculated factor of safety exceeds the design objective of 1.50, provided no surcharge is allowed within a distance equal to half the depth of the excavation from the shoring face.

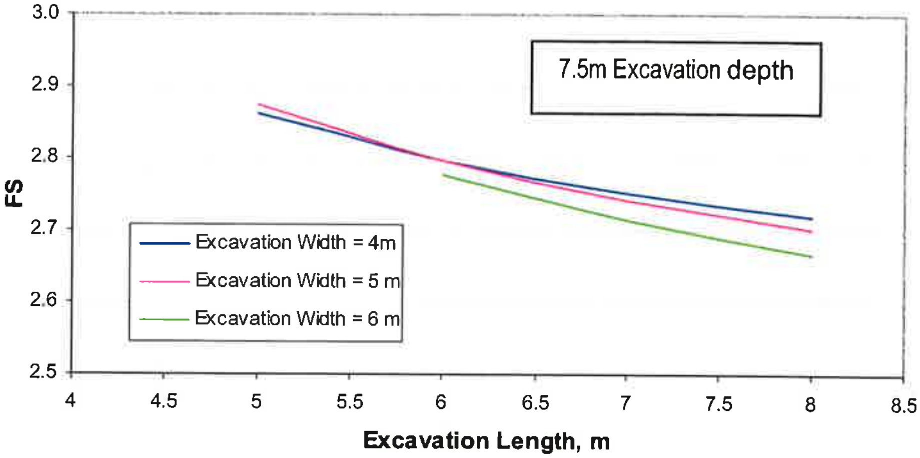


Figure 06: Factor of Safety Against Base Instability

The factor of safety against base heave is defined as the ratio of the bulk unit weight of the clay to the groundwater head acting on the base of the clay layer and should be a minimum of 1.5. For example, a factor of safety of xxx againstr base heave was calculated using a piezometric elevation of 222.39 in the till, a bulk unit weight of 17 kN/m^3 for the clay and the anticipated maximum excavation depth of 7.5 below ground surface (elevation 224.70). In this example the calculated factor of safety exceed the minimum requirement and base heave in not of concern during access shaft excavation. Similar calculations should be performed once final excavation depth are know and piezometric elevation in the till confirmed at the time of construction. Groundwater depressurization of the till and the underlying limestone aquifer will likely not be required, however groundwater monitoring is recommended before and during construction to confirm that groundwater levels do not exceed the values used in determination of the factor of safety against base heave.

During construction, the potential for groundwater flow into the excavation from silt layer and along existing vertical fractures in the clay if the head in the till exceeds the elevation of excavation base cannot be ruled out. Should such conditions occur, it is expected that the seepage will be at a rate which can be handled by conventional construction dewatering equipment.

8.0 Construction Monitoring Program

The CP geotechnical protocol, referenced in Section 1.0, stipulates the requirements for a surface and subsurface monitoring program. To our knowledge, there are no utilities buried at the proposed crossing that would be considered sensitive to the small ground displacement predicted. Therefore subsurface displacement monitoring can be waived. The ground surface subsidence can be monitored using standard survey points on the ground surface and on rail ties. However, because the precision of the standard levelling is in the order of +/- 5 mm it may not be sufficient to accurately measure settlement within the predicted range (< 5 mm). This precision however is capable of detecting 50 percent or less of surface subsidence that would be considered a reason for track class change.

The estimated cost estimate for the monitoring program implementation, data collection and interpretation is $ 31,000 excluding he GST and RST. The cost estimate is prepared based on the following assumptions:

1. Inspection of 30 by 30 m area at the proposed pipe crossing location and establishment of base lines and control points before construction.
2. Perform three monitoring events before construction to assess the survey precision and the impact of other factors such as train traffic on survey data.
3. Monitoring will commence when pipe installation takes place between the nearest shafts north and south the existing track.
4. Daily collection and distribution of the survey data.
5. Ten days of monitoring.

Details of the cost estimate are attached in Appendix C. Should the actual monitoring schedule differ than assumed, the unit rates provided in Appendix C will apply.

8.1 Proposed Notification and Action Plan

According to CP personnel, the track under consideration is TC class 3 track with a maximum tolerable relative displacement between rails of 1.75 inch (44 mm). The track class would be changed to TC class 2 if the displacement exceed 1.75 inch. This limit of track displacement, the estimated ground surface subsidence above pipe and the expected precision of the survey equipment were considered in

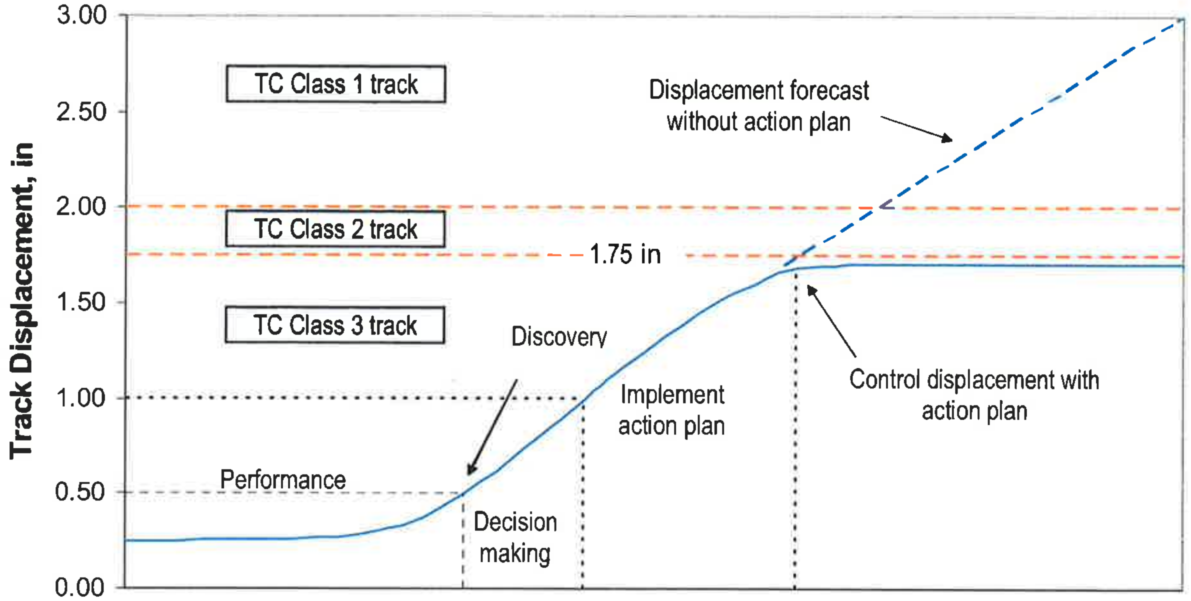
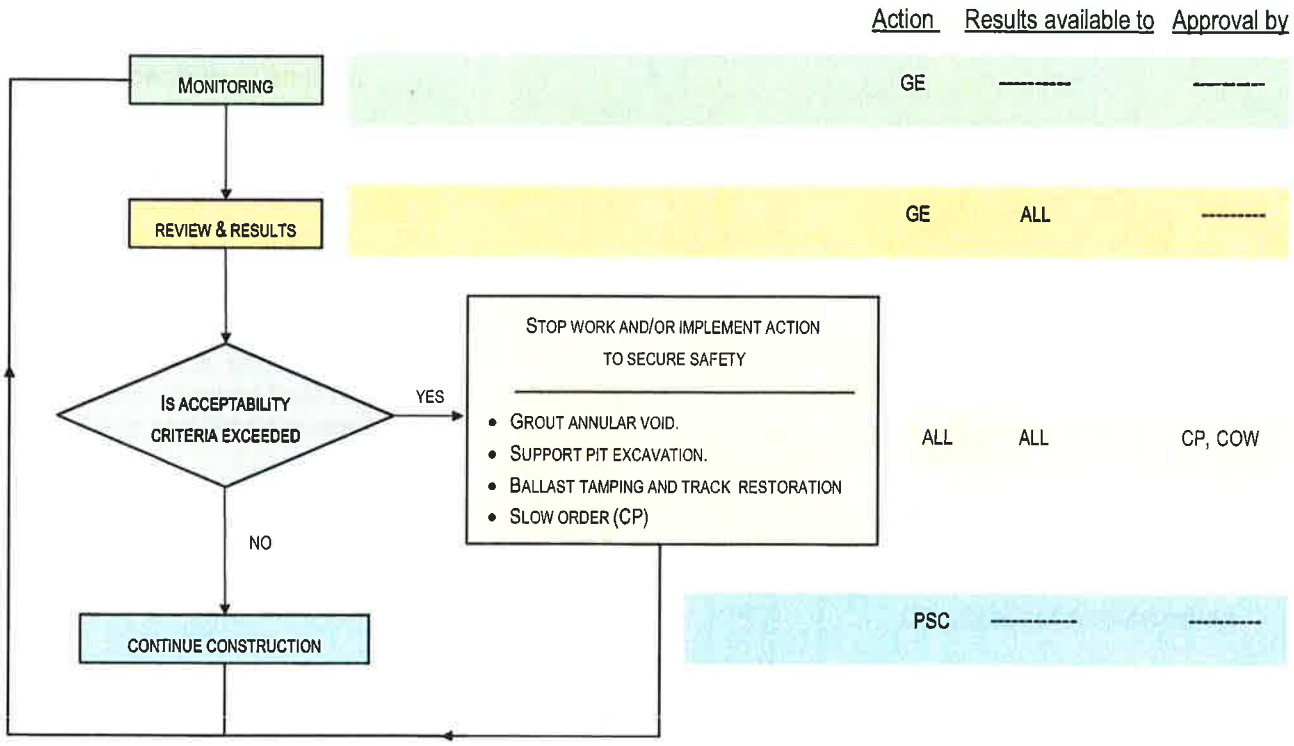


Figure 07: Proposed Numerical Values for Track Displacement Trigger Levels



Action	Results available to	Approval by
GE	---------	--------
GE	ALL	---------
ALL	ALL	CP, COW
PSC	-----------	----------

ALL : (CP, COW, DC, GE, PSC)
CP: Canadian Pacific Railway, COW: City of Winnipeg, DC: Dillon Consulting, GE: Geotechnical Engineer, PSC: Pipe Specialist Contractor

Figure 08: Proposed Notification and Action Plan

Appendix A

Test Holes Location Plan,
Pipe/Track Crossing Plan

B SIZE 11" x 17" (279.4mm x 431.8mm)

PLOT: 07/11/26 11:19:50 AM

Saved By: stevensc

UMA FILE NAME: F504-023-00_01-B-F001_RX.dwg

ISS/REV: A

PLAN

0 5 10 m SCALE 1:500

Dillon Consulting

Panet Road Reconstruction

New LDS Crossing Within CPR ROW at Callsbeck Ave.

Test Holes Location Plan

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Drawing - 01

B SIZE 11" x 17" (279.4mm x 431.8mm)

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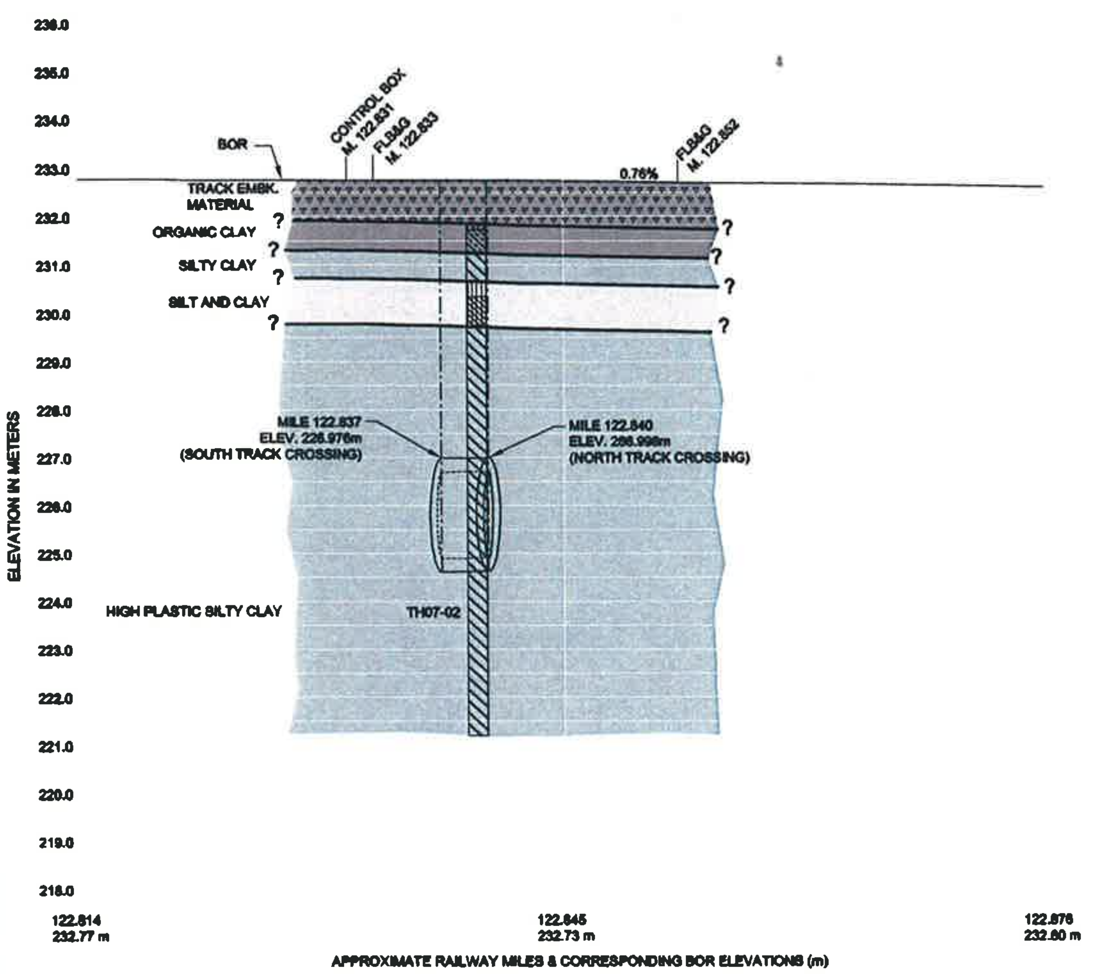
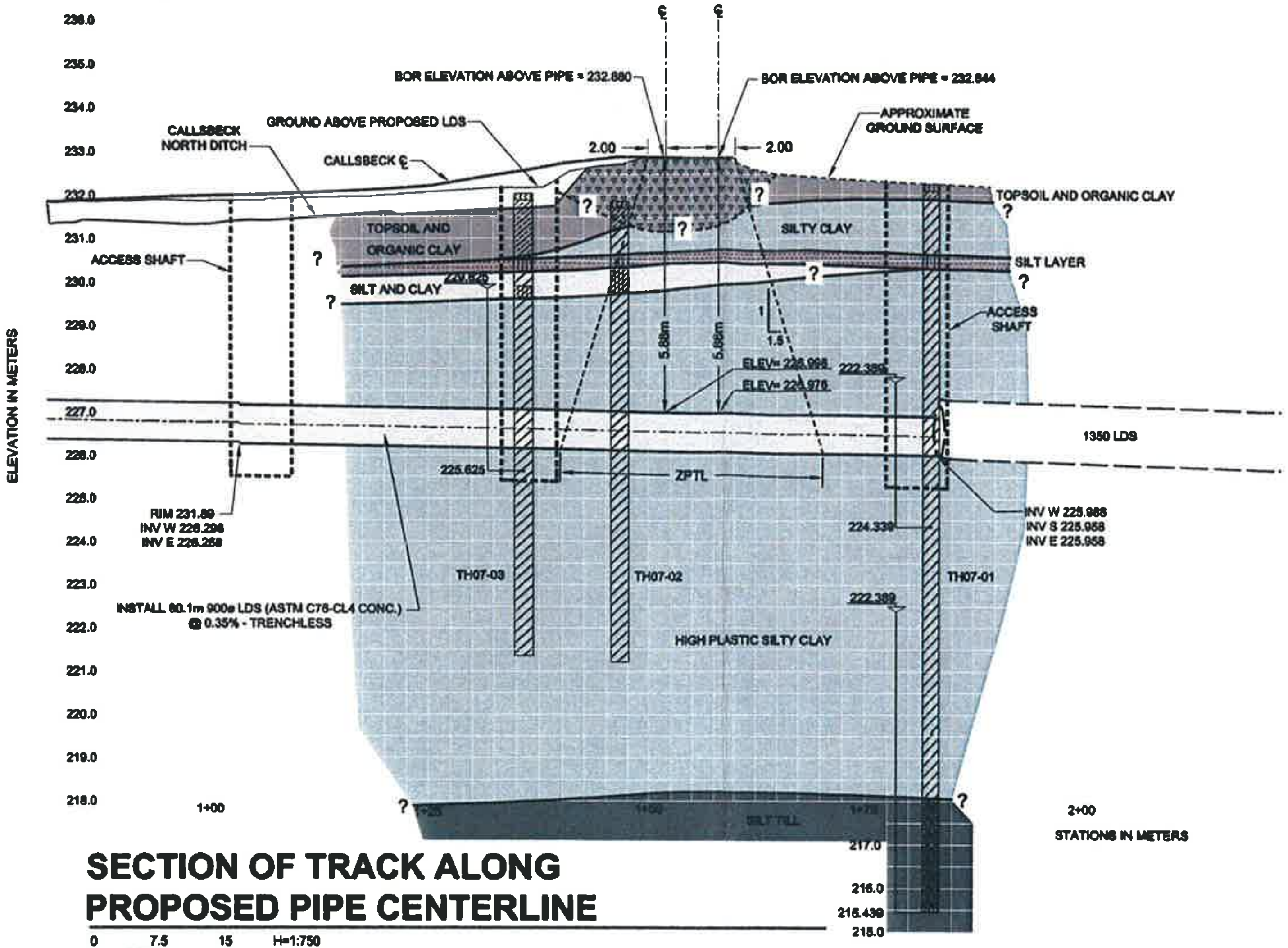


PROFILE OF THE TRACKS AND PROPOSED PIPE CROSSING (LOOKING SOUTH)

0 7.5 15 m H=1:750

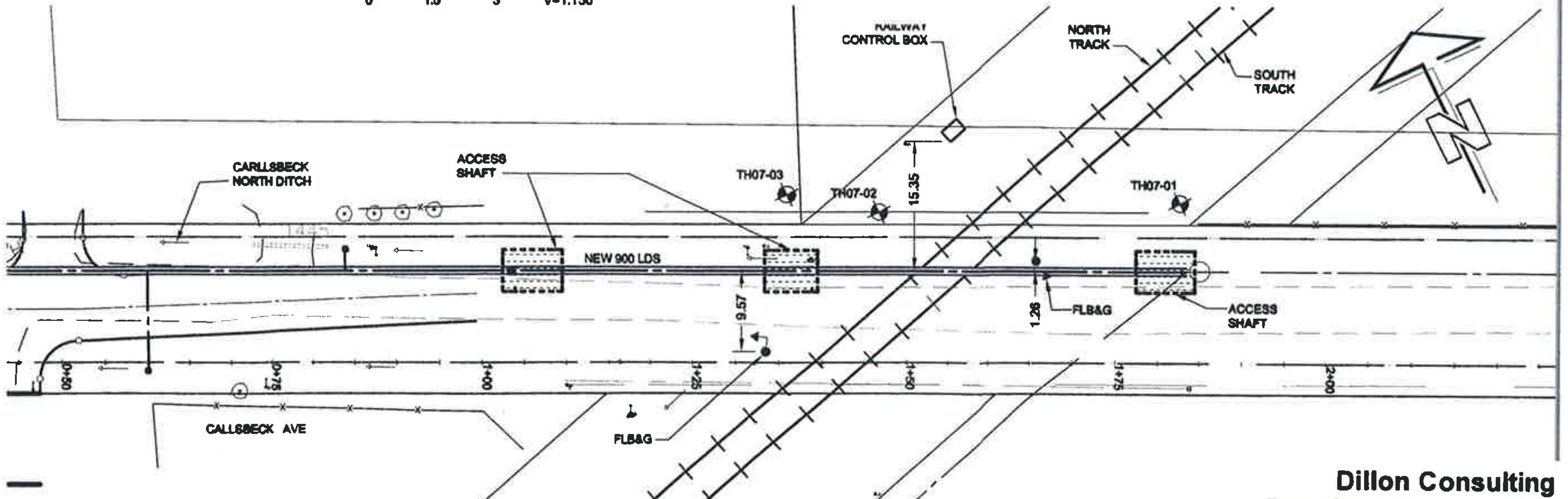
0 1.5 3 V=1:150



SECTION OF TRACK ALONG PROPOSED PIPE CENTERLINE

0 7.5 15 m H=1:750

0 1.5 3 V=1:150



PLAN OF THE PROPOSED PIPE CROSSING

0 1.5 3 m SCALE 1:750

Dillon Consulting
Panet Road Reconstruction
New LDS Crossing Within CP ROW at Callsbeck Ave.

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Drawing - 02

Appendix B

Test Hole Logs

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GENERAL STATEMENT

NORMAL VARIABILITY OF SUBSURFACE CONDITIONS

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability for the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of earth work, foundations and similar. In the event of any changes in the basic design or location of the structures as outlined in this report or plan, we should be given the opportunity to review the changes and to modify or reaffirm in writing the conclusions and recommendations of this report.

The analysis and recommendations presented in this report are based on the data obtained from the borings and test pit excavations made at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere are not significantly different from those disclosed by the borings and excavations. However, variations in soil conditions may exist between the excavations and, also, general ground water levels and conditions may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions different from those encountered in the exploratory borings and excavations are observed or encountered during construction or appear to be present beneath or beyond excavations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Since it is possible for conditions to vary from those assumed in the analysis and upon which our conclusions and recommendations are based, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction procedures.

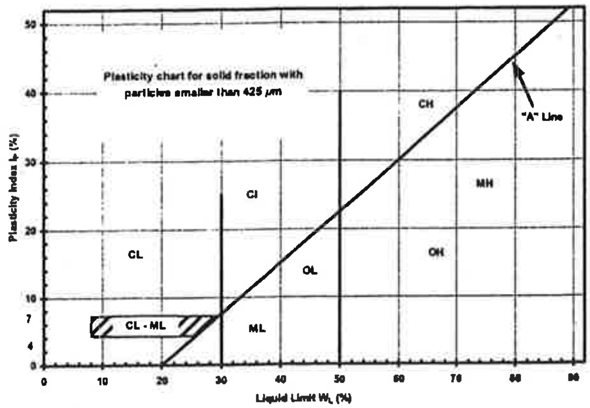
In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced soils engineer. We can be retained to provide these services for you during construction. In addition we can be retained to review the plans and specifications that have been prepared to check for substantial conformance with the conclusions and recommendations contained in our report.

EXPLANATION OF FIELD & LABORATORY TEST DATA

Description				UMA Log Symbols	USCS Classification	Laboratory Classification Criteria			
						Fines (%)	Grading	Plasticity	Notes
COARSE GRAINED SOILS	GRAVELS (More than 50% of coarse fraction of gravel size)	CLEAN GRAVELS (Little or no fines)	Well graded gravels, sandy gravels, with little or no fines	◬	GW	0-5	$C_U > 4$ $1 < C_C < 3$		Dual symbols if 5-12% fines. Dual symbols if above "A" line and $4<W_P<7$ $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10} x D_{60}}$
			Poorly graded gravels, sandy gravels, with little or no fines	◭	GP	0-5	Not satisfying GW requirements		
		DIRTY GRAVELS (With some fines)	Silty gravels, silty sandy gravels	▥	GM	> 12		Atterberg limits below "A" line or $W_P<4$	
			Clayey gravels, clayey sandy gravels	▨	GC	> 12		Atterberg limits above "A" line or $W_P<7$	
	SANDS (More than 50% of coarse fraction of sand size)	CLEAN SANDS (Little or no fines)	Well graded sands, gravelly sands, with little or no fines	⁘	SW	0-5	$C_U > 6$ $1 < C_C < 3$		
			Poorly graded sands, gravelly sands, with little or no fines	○	SP	0-5	Not satisfying SW requirements		
		DIRTY SANDS (With some fines)	Silty sands, sand-silt mixtures	▥	SM	> 12		Atterberg limits below "A" line or $W_P<4$	
			Clayey sands, sand-clay mixtures	▨	SC	> 12		Atterberg limits above "A" line or $W_P<7$	
FINE GRAINED SOILS	SILTS (Below 'A' line negligible organic content)	$W_L<50$	Inorganic silts, silty or clayey fine sands, with slight plasticity	▥	ML			Classification is Based upon Plasticity Chart	
		$W_L>50$	Inorganic silts of high plasticity	▮	MH				
	CLAYS (Above 'A' line negligible organic content)	$W_L<30$	Inorganic clays, silty clays, sandy clays of low plasticity, lean clays	▨	CL				
		$30<W_L<50$	Inorganic clays and silty clays of medium plasticity	▧	CI				
		$W_L>50$	Inorganic clays of high plasticity, fat clays	▨	CH				
	ORGANIC SILTS & CLAYS (Below 'A' line)	$W_L<50$	Organic silts and organic silty clays of low plasticity	▥	OL				
		$W_L>50$	Organic clays of high plasticity	▨	OH				
HIGHLY ORGAINIC SOILS			Peat and other highly organic soils	≋	Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

■	Asphalt	⁂	Till			UMA | AECOM
⁖	Concrete	▩	Bedrock (Undifferentiated)			
▩	Fill	▦	Bedrock (Limestone)			

When the above classification terms are used in this report or test hole logs, the designated fractions may be visually estimated and not measured.



FRACTION		SEIVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		Passing	Retained	Percent	Identifier
Gravel	Coarse	76	19	35-50	and
	Fine	19	4.75		
Sand	Coarse	4.75	2.00	20-35	"y" or "ey" *
	Medium	2.00	0.425		
	Fine	0.425	0.075	10-20	some
Silt (non-plastic) or Clay (plastic)		< 0.075 mm		1-10	trace

* for example: gravelly, sandy clayey, silty

Definition of Oversize Material

COBBLES: 76mm to 300mm diameter
BOULDERS: >300mm diameter

LEGEND OF SYMBOLS

Laboratory and field tests are identified as follows:

q_u -	undrained shear strength (kPa) derived from unconfined compression testing.
T_v -	undrained shear strength (kPa) measured using a torvane
pp -	undrained shear strength (kPa) measured using a pocket penetrometer.
L_v -	undrained shear strength (kPa) measured using a lab vane.
F_v -	undrained shear strength (kPa) measured using a field vane.
γ -	bulk unit weight (kN/m^3).
SPT -	Standard Penetration Test. Recorded as number of blows (N) from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 51 mm O.D. Raymond type sampler 0.30 m into the soil.
DPPT -	Drive Point Pentrometer Test. Recorded as number of blows from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 50 mm drive point 0.30 m into the soil.
w -	moisture content (W_L, W_P)

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

Su (kPa)	CONSISTENCY
25	very soft
25 – 50	soft
50 – 100	medium or firm
100 – 200	stiff
200 – 400	very stiff
400	hard

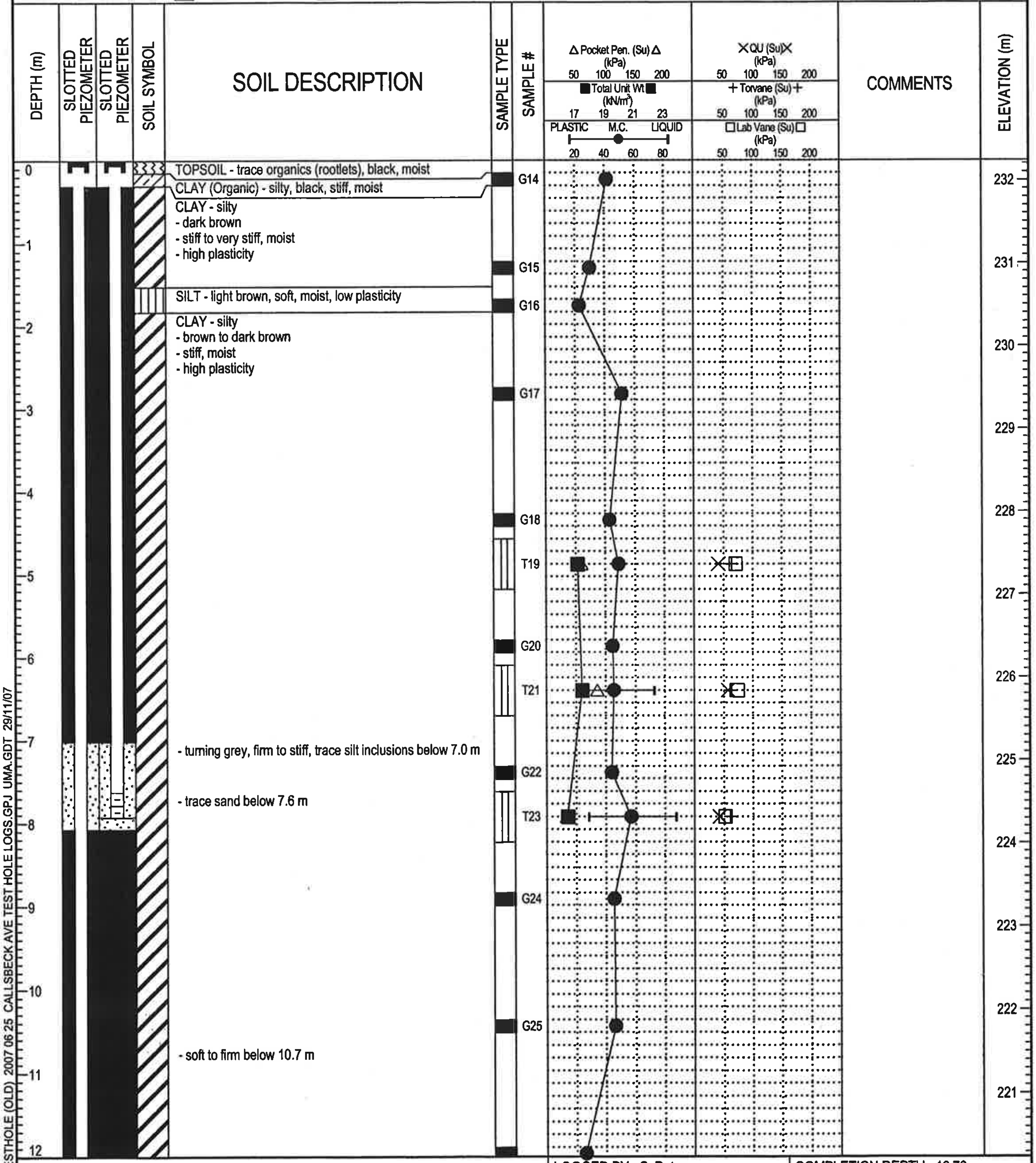
The resistance (N) of a non-cohesive soil can be related to compactness condition as follows

N – BLOWS/0.30 m	COMPACTNESS
0 - 4	very loose
4 - 10	loose
10 - 30	compact
30 - 50	dense
50	very dense

PROJECT: Panet Road Reconstruction - 900 mm LDS Crossing	CLIENT: Dillion Consulting	TESTHOLE NO: **TH07-01**
LOCATION: South of CP Keewatin Rail tracks, East shoulder of Callsbeck Ave. N 5,530,386.9 E 639,162.9		PROJECT NO.: F504 023 00
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125mm dia. SSA	ELEVATION (m): 232.239

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁙ GRAVEL ▥ SLOUGH ⁙ GROUT ▨ CUTTINGS ⁙ SAND



TOPSOIL - trace organics (rootlets), black, moist

CLAY (Organic) - silty, black, stiff, moist

CLAY - silty
- dark brown
- stiff to very stiff, moist
- high plasticity

SILT - light brown, soft, moist, low plasticity

CLAY - silty
- brown to dark brown
- stiff, moist
- high plasticity

- turning grey, firm to stiff, trace silt inclusions below 7.0 m

- trace sand below 7.6 m

- soft to firm below 10.7 m

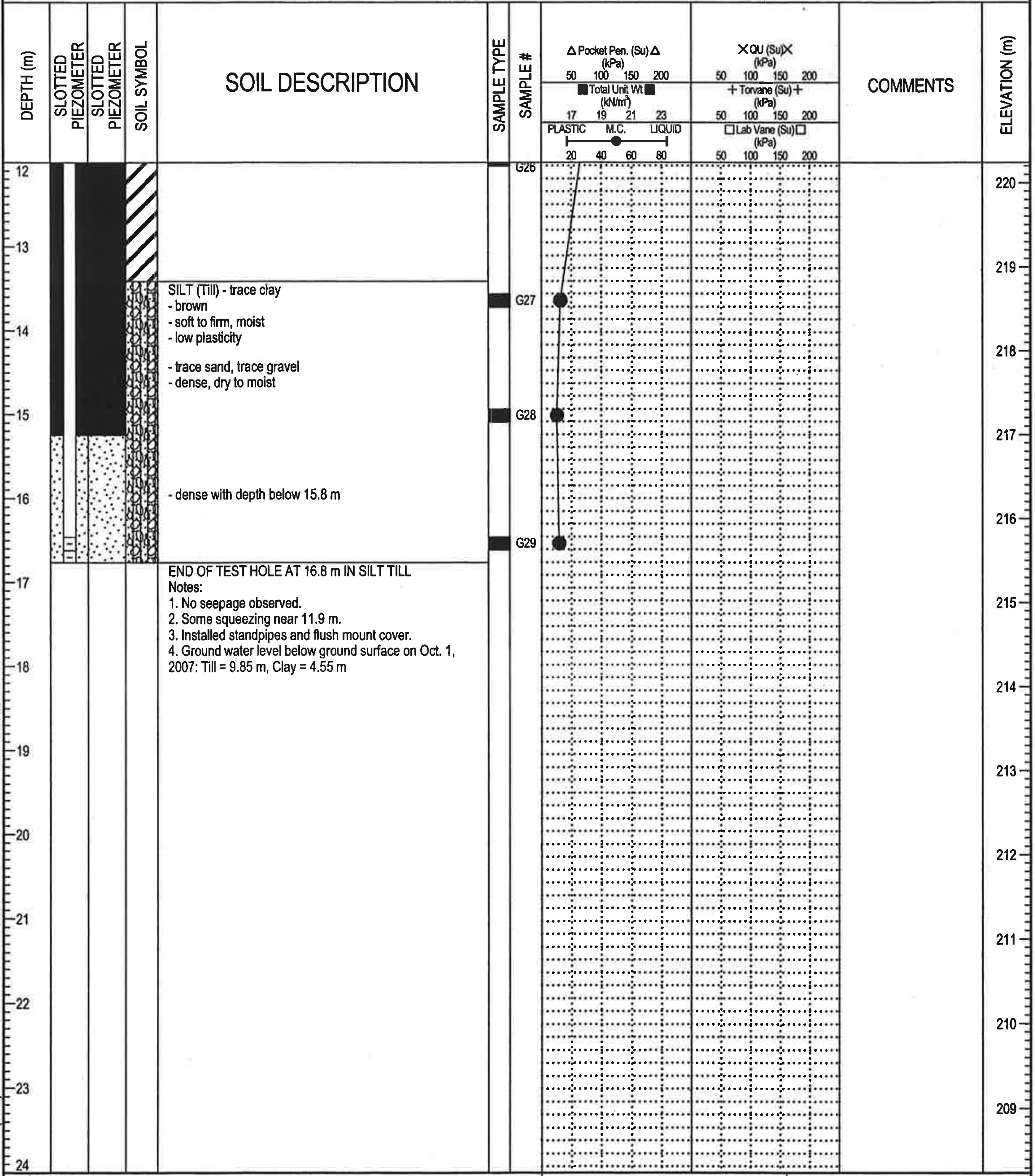
UMA | AECOM

LOGGED BY: S. Peters	COMPLETION DEPTH: 16.76 m
REVIEWED BY: F. Khalil	COMPLETION DATE: 21/6/07
PROJECT ENGINEER: F. Khalil	Page 1 of 2

LOG OF TESTHOLE (OLD) 2007 06 25 CALLSBECK AVE TEST HOLE LOGS.GPJ UMA.GDT 29/11/07

PROJECT: Panet Road Reconstruction - 900 mm LDS Crossing	CLIENT: Dillion Consulting	TESTHOLE NO: **TH07-01**
LOCATION: South of CP Keewatin Rail tracks, East shoulder of Callsbeck Ave. N 5,530,386.9 E 639,162.9		PROJECT NO.: F504 023 00
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125mm dia. SSA	ELEVATION (m): 232.239

SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▨ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	⁘ GROUT	▨ CUTTINGS	⁘ SAND



SILT (Till) - trace clay
- brown
- soft to firm, moist
- low plasticity

- trace sand, trace gravel
- dense, dry to moist

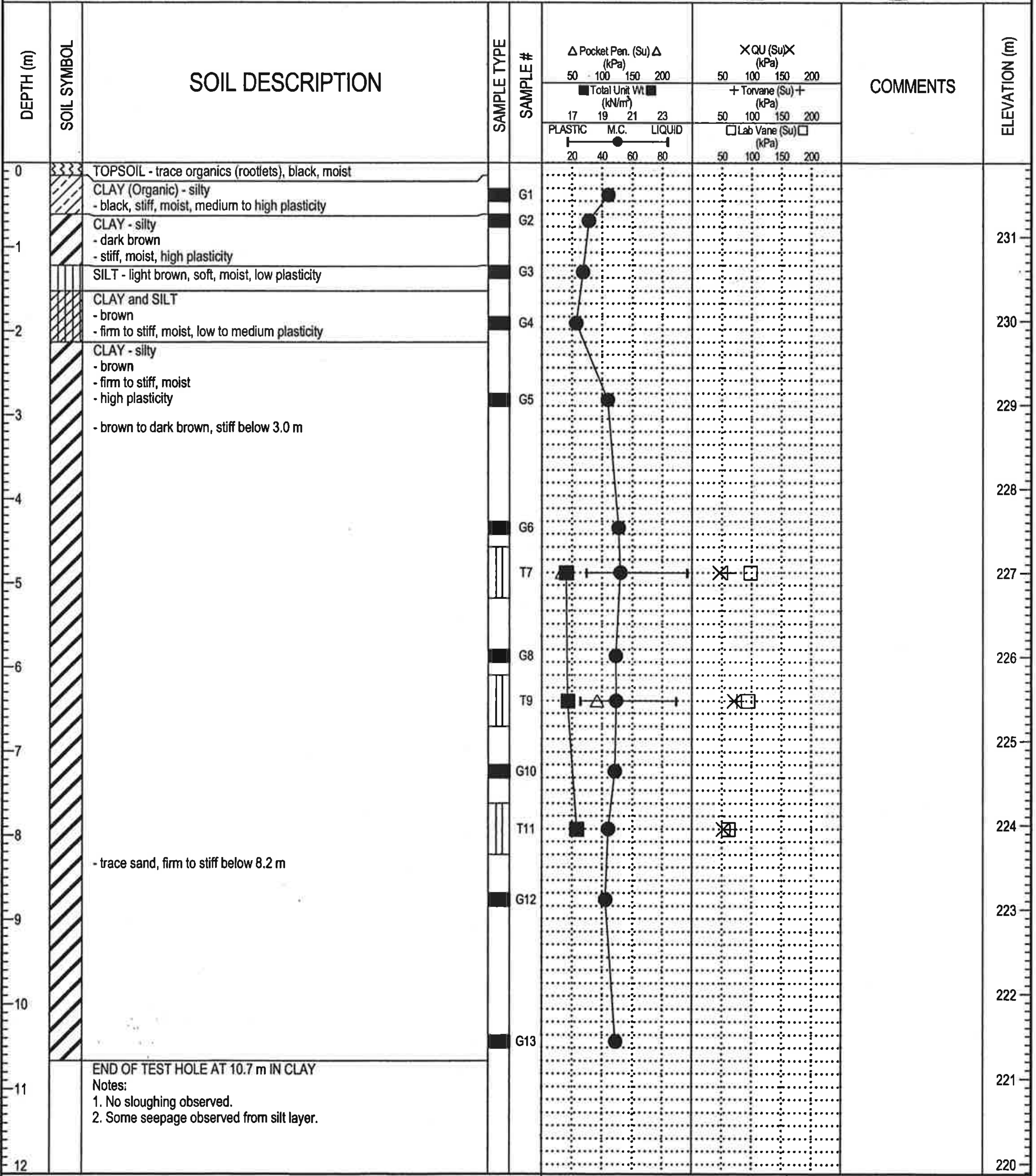
- dense with depth below 15.8 m

END OF TEST HOLE AT 16.8 m IN SILT TILL
Notes:
1. No seepage observed.
2. Some squeezing near 11.9 m.
3. Installed standpipes and flush mount cover.
4. Ground water level below ground surface on Oct. 1, 2007: Till = 9.85 m, Clay = 4.55 m

UMA | AECOM	LOGGED BY: S. Peters	COMPLETION DEPTH: 16.76 m
	REVIEWED BY: F. Khalil	COMPLETION DATE: 21/6/07
	PROJECT ENGINEER: F. Khalil	Page 2 of 2

PROJECT: Panet Road Reconstruction - 900 mm LDS Crossing	CLIENT: Dillion Consulting	TESTHOLE NO: **TH07-02**
LOCATION: North of CP Keewatin Rail tracks, East shoulder of Callsbeck Ave. N 5,530,401.5 E 639,130.2		PROJECT NO.: F504 023 00
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125mm dia. SSA	ELEVATION (m): 231.876

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◪ NO RECOVERY ▮ CORE



TOPSOIL - trace organics (rootlets), black, moist

CLAY (Organic) - silty
- black, stiff, moist, medium to high plasticity

CLAY - silty
- dark brown
- stiff, moist, high plasticity

SILT - light brown, soft, moist, low plasticity

CLAY and SILT
- brown
- firm to stiff, moist, low to medium plasticity

CLAY - silty
- brown
- firm to stiff, moist
- high plasticity

- brown to dark brown, stiff below 3.0 m

- trace sand, firm to stiff below 8.2 m

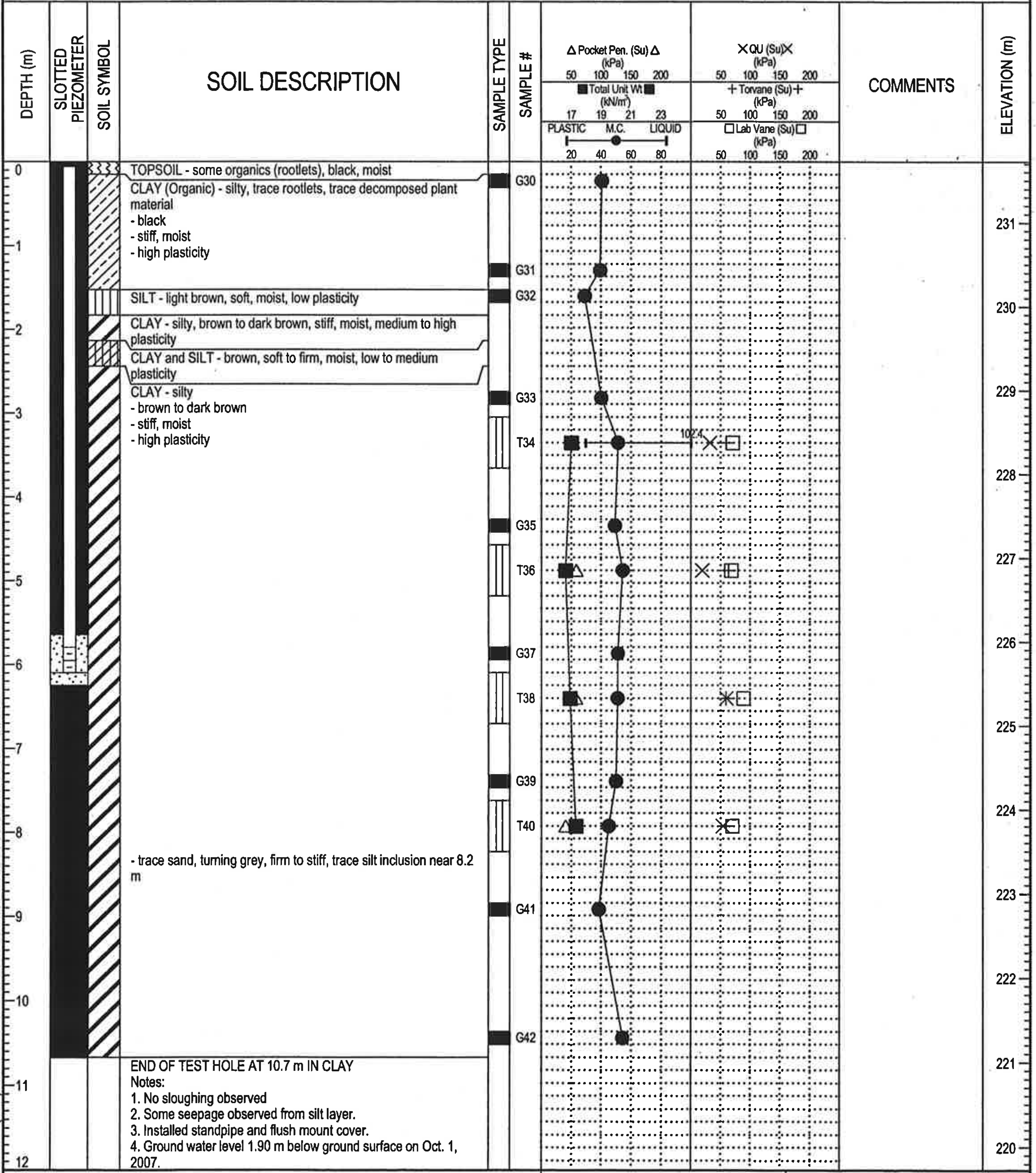
END OF TEST HOLE AT 10.7 m IN CLAY
Notes:
1. No sloughing observed.
2. Some seepage observed from silt layer.

LOGGED BY: S. Peters	COMPLETION DEPTH: 10.67 m
REVIEWED BY: F. Khalil	COMPLETION DATE: 21/6/07
PROJECT ENGINEER: F. Khalil	

PROJECT: Panet Road Reconstruction - 900 mm LDS Crossing	CLIENT: Dillion Consulting	TESTHOLE NO: **TH07-03**
LOCATION: North of CP Keewatin Rail tracks, East shoulder of Callsbeck Ave.		PROJECT NO.: F504 023 00
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125mm dia. SSA	ELEVATION (m): 231.725

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



SOIL DESCRIPTION

TOPSOIL - some organics (rootlets), black, moist

CLAY (Organic) - silty, trace rootlets, trace decomposed plant material
- black
- stiff, moist
- high plasticity

SILT - light brown, soft, moist, low plasticity

CLAY - silty, brown to dark brown, stiff, moist, medium to high plasticity

CLAY and SILT - brown, soft to firm, moist, low to medium plasticity

CLAY - silty
- brown to dark brown
- stiff, moist
- high plasticity

- trace sand, turning grey, firm to stiff, trace silt inclusion near 8.2 m

END OF TEST HOLE AT 10.7 m IN CLAY
Notes:
1. No sloughing observed
2. Some seepage observed from silt layer.
3. Installed standpipe and flush mount cover.
4. Ground water level 1.90 m below ground surface on Oct. 1, 2007.

UMA | AECOM

LOGGED BY: S. Peters	COMPLETION DEPTH: 10.67 m
REVIEWED BY: F. Khalil	COMPLETION DATE: 21/6/07
PROJECT ENGINEER: F. Khalil	Page 1 of 1