

APPENDIX A
GEOTECHNICAL REPORT

Memorandum

To	Barry Biswanger	Page	1
CC			
Subject	Pembina-Ducharme Culvert Replacement		
From	Omer Eissa		
Date	November 30, 2011	Project Number	60221826 (400)

Introduction

This memorandum summarizes the findings of additional geotechnical investigations and provides geotechnical recommendations for the Pembina-Ducharme Culvert Replacement project to be undertaken by the City of Winnipeg. This memo supplements the geotechnical investigation report included as Appendix B in AECOM's report "Pembina Highway Culvert Study Report" dated May 2011.

The above report has been completed as part of the design development process. It discusses subsurface conditions at the site and geotechnical concerns related to the design and construction of a new culvert and flood control chamber as well as recommendations to protect against further deterioration of the existing instabilities of the west side slope of Pembina Highway and the north bank of the Coulee Creek west of Pembina Highway.

This memo provides additional recommendations regarding the geotechnical aspects of the proposed trenchless pipe installation and stabilization alternatives to the previously proposed 9H:1V Pembina side slope.

Subsurface Investigation

Two (2) test holes were advanced at the east and west sides of Pembina Highway. Details of the test holes locations and termination depths are provided in Table 1. Test hole TH11-02 was advanced 1.5 m into the till and terminated at elevation 217.6 m, while TH11-01 was terminated in clay at elevation 215.3 m upon auger refusal on suspected boulder.

Table 1: Details of Test Holes

Test Hole #	Northing (m)	Easting (m)	Termination Depth (m)	Standpipe Piezometer
TH11-01	0632771	5514569	10.4	No
TH11-02	0632714	5514572	12.7	Yes

The soil profile at the test hole locations in descending order consists of:

- Topsoil
- Clay (alluvial)
- Clay (Lacustrine)
- Till

Topsoil

Topsoil was encountered at the surface of both test holes. The topsoil extended to approximately 0.3 metres below ground surface. The topsoil is black, dry to moist and contains trace rootlets and organics.

Clay (Alluvial)

Alluvial clay was encountered below the topsoil and extended to an approximate elevation of 225 m. The clay is silty and contains some sand, trace gravel and pockets of organics. The clay is brown to olive brown, firm to stiff and based on visual observation, is classified as of intermediate to high plasticity.

Clay (Lacustrine)

Lacustrine clay of high plasticity was encountered from an approximate elevation of 225 m to the till interface at an approximate elevation of 217 m. The clay is grey, homogenous and contains trace to some amount of silt. It is generally moist, and of soft to firm consistency.

Till

Till was encountered in TH11-02 at elevation 216.7 m. The till is predominantly silt and contains variable amounts of sand, clay and gravel.. The silt till is light brown to brown, wet, non plastic and of dense to very dense consistency.

Stability Assessment

Slope instability and head scarp along the crest of the north bank of the Coulee are visible at the site and were documented in AECOM's 2011 report. On the west side of Pembina Highway, slope instabilities have visibly undermined the existing fence, lamp standard and forced sidewalk reconstruction. It was reported that the instabilities have damaged the existing 1200 mm diameter concrete land drainage sewer that discharges into the Coulee from the north. The sidewalk has been

reconstructed in recent years by adding base material and replacing the sidewalk concrete. The sidewalk has remained unaffected by the instabilities since the last repairs.

To address the existing instabilities, AECOM's 2011 report provided recommendations for crest unloading, grading works, and flat slopes 9H:1V. These design components are associated with longer culvert and would require easement and encroachment into the private property on the north-west quadrant which is currently under consideration for development. The final development plans are not finalized yet by the Developer and it is expected that the above design components would not fit the intended use of the property. Therefore, the stabilization measures have been reviewed to accommodate the future site conditions. It is expected that the existing instabilities at the Coulee north bank will be addressed as part of the development of the private property on the north-west quadrant and therefore no further discussion related to the Coulee bank stabilization is provided in this report.

Slope stability analysis was completed for Pembina side slope with the following objectives:

- Improve the stability to attain factor of safety (FS) of 1.5.
- Provide slope no steeper than 5H:1V to maintain easy access for culvert maintenance.
- Reduce the length of the proposed culvert as much as practical.

The soil strength parameters used for the analysis are provided in **Table 2**. These strength parameters were established in AECOM's 2011 report based on the results of back analysis using fitted slip surfaces on the existing cross sections. .

Table 2: Soil Parameters for Slope Stability Assessment

Material	Unit Weight (kg/m ³)	Angle of Internal Friction (ϕ)	Cohesion (kPa)
Clay (intact)	18	16	7
Clay (residual)	18	10	0
Granular (Shear Key)	16.5	37	0
Granular (2" down)	18	34	0

Computer-aided numerical modeling incorporating Slope/W (geoslope International) was used to complete the slope stability analysis. The soil stratigraphy used in the analysis was based on test hole information from AECOM's investigation carried out on September 30th, 2009 and September 29, 2011. A process was devised for the design of the new west side slope of Pembina Highway as shown in **Figure 1**.



Figure 1

The stability analysis was conducted by first using a systematic reduction of the slope from the originally proposed 9H:1V to 5H:1V without any stability measures to assess the associated factors of safety. A slope of 6H:1V was selected based on the preliminary results shown in **Table 3**. This slope will result in reasonable culvert length and would be relatively less affected by the future development at the north side of the Coulee.

Table 3: Factors of Safety for Filling Side Slope with no Stabilization Measures

Slope	Factor of Safety (FS)
9:1	1.4
8:1	1.28
7:1	1.15
6:1	1.0
5:1	0.89

The analysis was carried forward to evaluate alternatives for stabilization measures including MSE retaining walls, rock columns and shear key to improve FS against slope instability and attain the design objective of FS = 1.5. The shear key was considered the most suitable alternative for this project. A shear key downstream the existing 3H:1V slope would be relatively easy to construct and would be cost effective without the need for specialized equipment/contractors. MSE retaining wall along the sidewalk of Pembina Highway would require deep foundation excavation/preparation so that the wall is bearing on competent soil below the shear zone associated with the observed instability. Such deep excavation would require extensive shoring requirements, pavement repairs and would adversely impact traffic operation. Rock columns would be more complicated and costly to install for a project of this size. The mobilization and access for the specialized equipment to a small site with limited access would be complex. Therefore, both MSE wall and rock columns were not considered suitable applications for this site.

The case was further analyzed to optimize the dimensions and location of the shear key. The analysis considered 3 and 4 m wide shear key at varying locations down the existing slope. The stability analysis takes into account global stability in terms of a factor of safety against deep seated slip surface as well as local instabilities and shallow slides. A schematic representation of global and local slip surfaces is illustrated on Figure 2.

The results of the analysis are presented in Table 4.

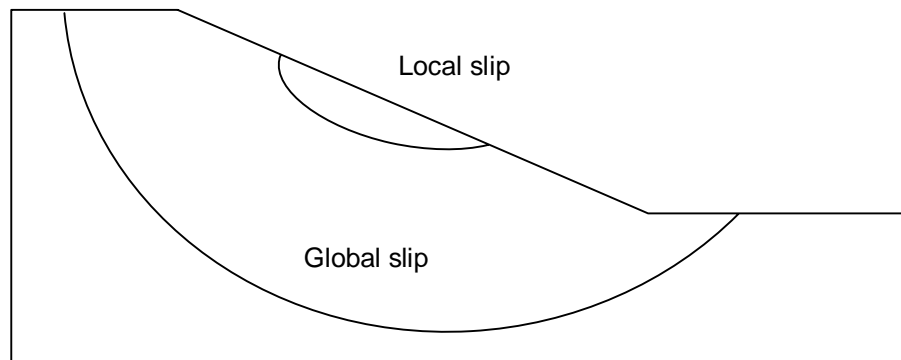


Figure 2: Schematic for global and local slips.

Table 4: Results of Stability Analysis for a Section Incorporating 3 and 4 m Wide Shear Key

Location* (m)	Factor of Safety			
	3 m wide shear key		4 m wide shear key	
	Local	Global	Local	Global
14	1.32	1.53	1.32	1.57
15	1.34	1.49	1.34	1.57
16	1.38	1.51	1.37	1.54
17	1.39	1.50	1.46	1.56
18	1.43	1.42	1.49	1.43
19	na	1.41	na	na

* Horizontal distance from slope crest to the near edge of Shear Key

Based on the results of the stability analysis, a 6H:1V granular fill slope and 4 metre wide shear key excavated to elevation 224 m at a distance of 17 metres from the slope crest is recommended. This slope/shear key configuration results in acceptable factors of safety against local and global slope instabilities.

The walls of the shear key can be excavated to near vertical provided the excavation remains safe and stable and confirms to all applicable safety regulations. The shear key excavation should be backfilled with 150 mm down crushed limestone. The granular material should be placed in lifts and packed using the bucket of suitably sized excavator or tracked by heavy construction equipment where possible. Excavation and backfill operations may need to be completed in staged sequence to maintain stability of the trench and facilitate construction. The excavation should be backfilled immediately after inspection by geotechnical personnel. No excavation should be left open overnight. Construction water control may be required to facilitate construction. The design should incorporate a compacted clay seal 600 mm minimum thickness as a permanent measure to protect against surface water infiltration into the shear key zone. The clay should be placed in layers not exceeding 200 mm and compacted to at least 95 percent of maximum standard proctor density before placement of subsequent layers.

Crushed limestone 150 mm down or 50 mm down can be used as granular fill for the new slope construction. The existing slope should be benched to provide key-in between the new and existing fill. No fill operation should take place before the completion of the shear key construction. The material should be placed in layers and compacted to the satisfaction of the Engineer. Details of the shear key are shown in Drawing 1, attached in Appendix A of this report.

Trenchless Pipe Installation Methods

There are two trenchless methods of pipe installation practiced locally. One method 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 responses. A brief description of each method is provided herein:

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 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.

Drive lengths up to 120 m have been successfully achieved in the Winnipeg area using this method. However, since the method requires personnel working inside the pipe, the method is limited to man entry size boring. Locally, 1050 mm diameter is considered the minimum size installed using this method.

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 centre 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 opposite 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 sections are added and the installation process continued.

Face Stability for Trenchless Installation

The face stability index, frequently referred to as the Overload Factor (OF), is the ratio of the difference between the vertical pressure at the tunnel axis and the pressure applied to the tunnel face, and the undrained shear strength of the soil. In cohesive soils, the tunnel face is considered stable when the index is less than 6. While the limiting OF value of 6 represents a threshold, an OF value of 5 represents a practical limit below which tunnelling can generally be carried out without unusual difficulties.

Using the selected design values of 30 kPa for undrained shear strength and 18 kN/m³ for bulk unit weight, the variation of the OF values with respect to the depth to pipe invert is shown in **Figure 3** for the proposed 1650 mm diameter culvert. Values of OF for the culvert design depth were calculated to be in the range between 2.5 to 3.5 suggesting satisfactory tunnel face stability. However, difficulties in face stability could be experienced if wet silt layers or seams are encountered within the clay along the culvert alignment.

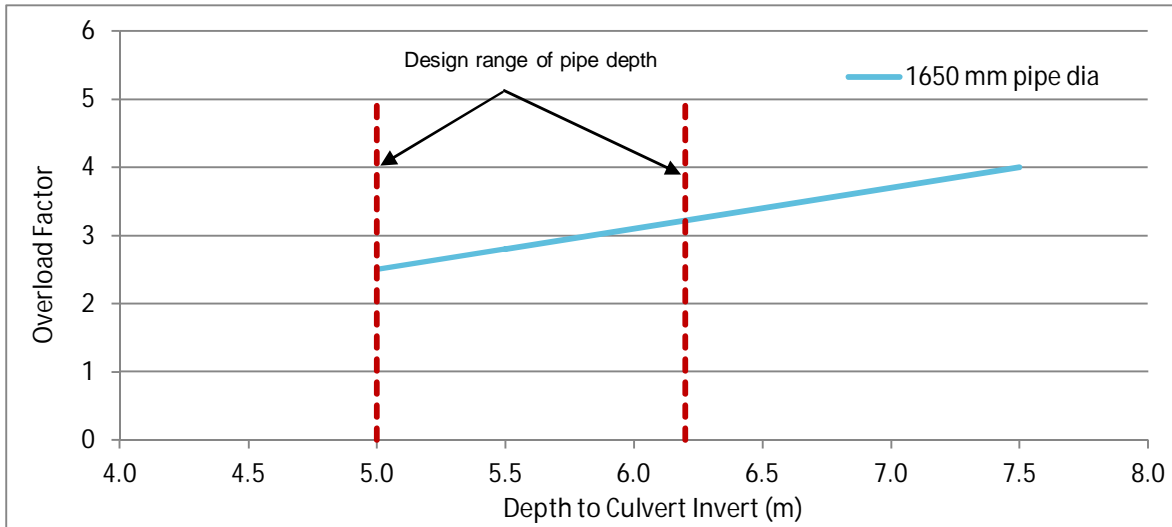


Figure 3: Overload Factor against Depth

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

Like other tunnelling methods, pipe jacking will result in a change in the state of stress in the ground with 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 tunnelling 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. It is suggested that a provision for pavement restoration at the crossing location be included in the project construction budget and schedule.

Excavation

Earth Pressure

Cantilevered shoring in Winnipeg clays is limited to depths of about 4 m. Beyond this depth, the shoring will generally have to be braced or tied back. In this regard, the earth pressure distribution shown in Figure 4 should be used to design the shoring. The design should account for all applicable surcharge loads. Shoring is usually designed to keep movements around the perimeter of the

excavation within acceptable limits. Avoidance of ground movements entirely is not possible. The amount of movement that will occur cannot be accurately predicted mainly because the movements are more a function of excavation procedures and workmanship than they are of theoretical considerations. Settlements of the ground surface adjacent to braced excavation are often estimated using the design chart developed by Peck (1969) as shown in Figure 5. It is recommended that the boundary between Zone II and III be used to estimate vertical ground movements at the site. It should be recognized that the predicted ground movements are associated with standard soldier piles and lagging or sheet piles with cross bracing or tie back anchors, assuming they are installed with a normal quality of workmanship. Good contact between the lagging and retained soil should be maintained throughout the construction period. Free draining sand should be used to fill the voids behind the lagging or sheet piles.

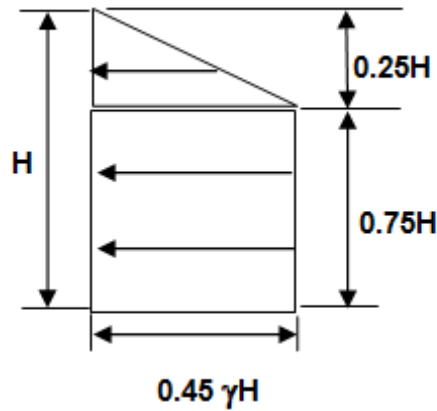


Figure 4: Earth Pressure Distribution for Shored Excavation

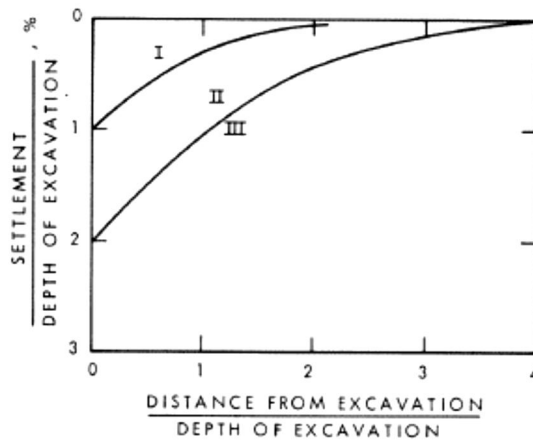


Figure 5: Ground Settlement Estimate Adjacent to Excavation

Temporary unsupported excavations up to a depth of 2 m could be cut with back slopes not steeper than 1H:1V. A detailed stability assessment should be carried out for excavations greater than 2 m in depth or if they are to remain open for an extended period of time. If soft zones or perched groundwater are encountered, flatter slopes may be required. Where a combination of open excavation and shoring is planned, 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.

Base Heave

The factor of safety against base heave (FS) is defined as the ratio of the bulk unit weight of the clay to the groundwater head in the till acting on the base of the clay layer. A minimum FS of 1.3 is recommended for short term conditions during construction (i.e., excavations for shear key and receiving/driving shafts, if required). The FS is a function of the groundwater level (GWL) in the till, depth to the till and the excavation depth. A range of GWL's from 224 to 228 m was considered for the base heave analysis. The till was encountered at an approximate elevation of 216.8 m in the investigations carried out at the site in 2009 and 2011. Excavation base elevation is expected to be at 224 m. Under these parameters, a sensitivity analysis for the FS against base heave with respect to GWL was performed with the results shown in Figure 6 . For the anticipated excavation elevation is 224 m and encountered till depth of 216.8 m, the design objective of FS = 1.30 is satisfied for GWL below EL 227. Recent groundwater level measurement in October 2011 was 225.5 m. Groundwater monitoring is recommended before and during construction to confirm that groundwater levels do not exceed the critical value of 227 m . During construction, there is a potential for groundwater flow into the excavation from silt layers and along existing fractures in the clay. Should this condition occur, it is expected that the seepage will be at a rate which can be handled by conventional construction dewatering equipment.

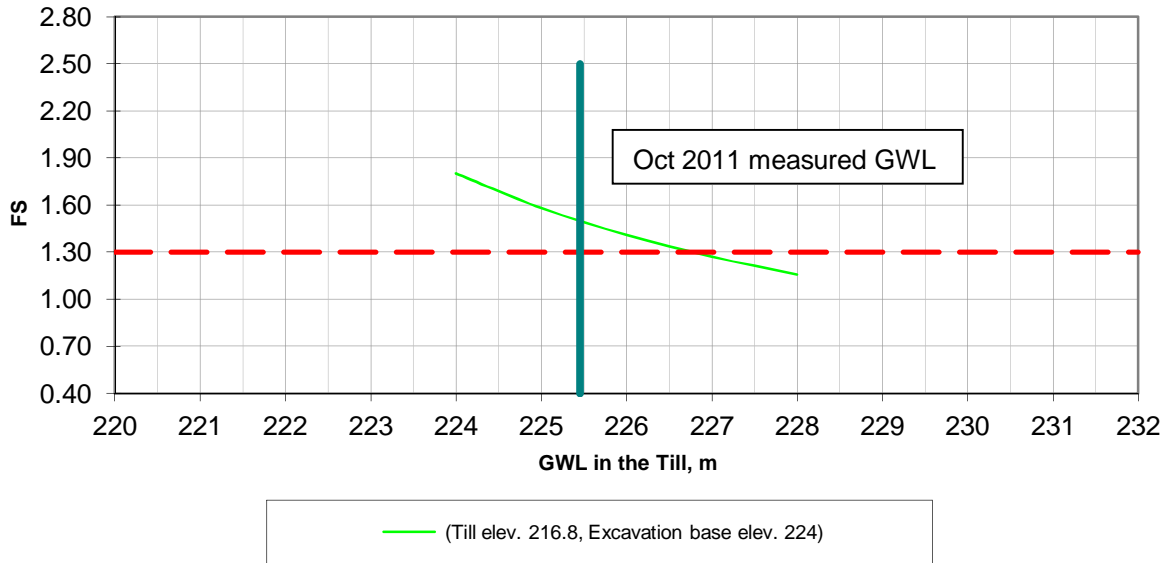


Figure 6: Factor of Safety Against Base Heave Potential

Base Instability

The potential of base instability must be considered 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

Based on the site topography, ground elevation at the inlet and outlet locations of the culvert are within 1.2 metres of the culvert inlet. It is expected that no access shafts will be needed for the pipe installation.


Minor cuts may be needed to create sufficient space for the equipment alongside the excavation face, mainly at the inlet location. The anticipated maximum depth of the shear key excavation is about 4 m. A minimum factor of safety of 1.50 is recommended for design purposes using an undrained shear strength of 30 kPa and a bulk unit weight of 18 kN/m³ for the clay. The calculated factor of safety against base instability for the shear key excavation is 2.8 satisfying the design objective, provided no surcharge is allowed within a distance equal to the depth of the excavation from the excavation face.

AECOM Canada Ltd.

Prepared by:


for Omer Eissa, B.Eng., EIT
Geotechnical Engineer-In-Training

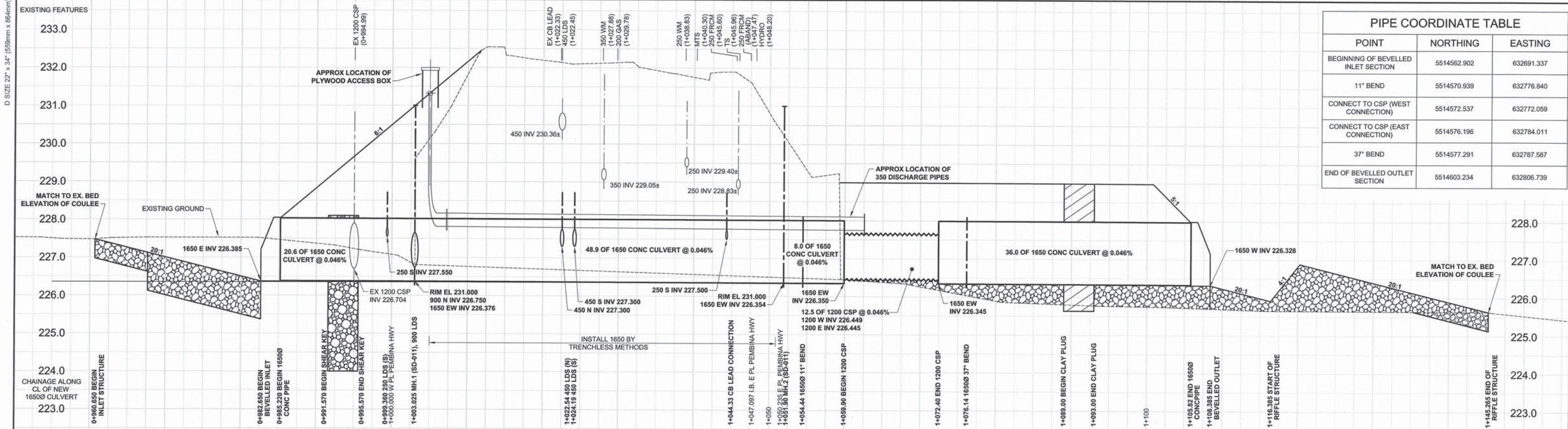
Reviewed by:


Faris Khalil, P.Eng.
Manager, Geotechnical Engineering

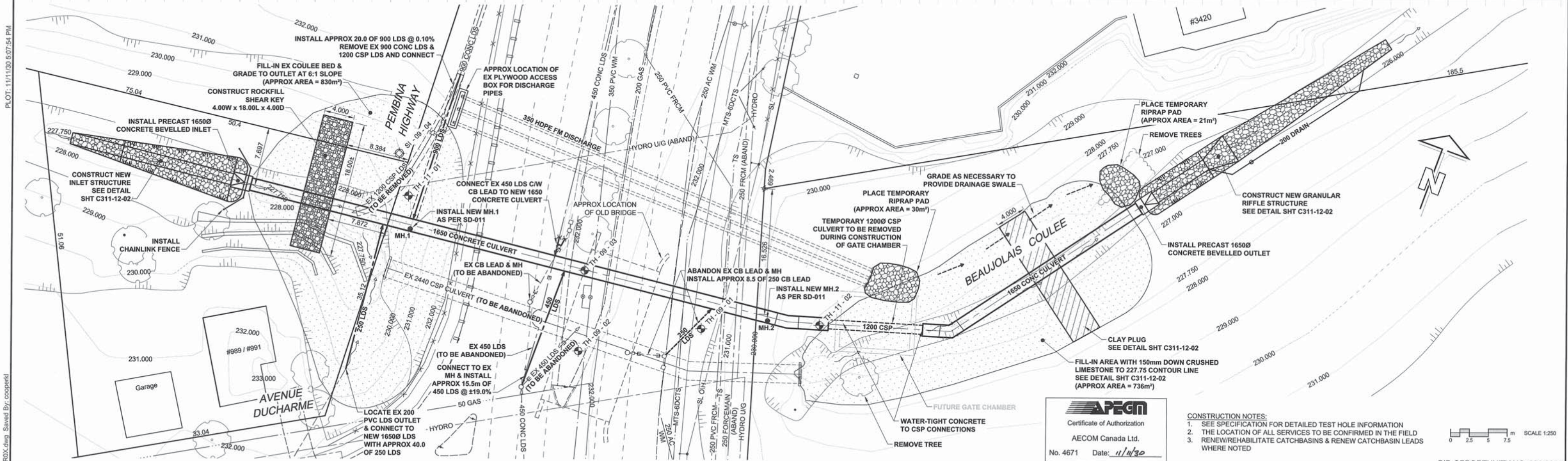
Appendix A

Drawing 1

D SIZE 22" x 34" (559mm x 864mm)

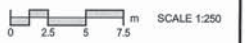


PIPE COORDINATE TABLE		
POINT	NORTHING	EASTING
BEGINNING OF BEVELLED INLET SECTION	5514562.902	632691.337
11" BEND	5514570.939	632776.840
CONNECT TO CSP (WEST CONNECTION)	5514572.537	632772.059
CONNECT TO CSP (EAST CONNECTION)	5514576.196	632784.011
37" BEND	5514577.291	632787.587
END OF BEVELLED OUTLET SECTION	5514603.234	632806.739



AECOM
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date: 11/11/20

- CONSTRUCTION NOTES:
- SEE SPECIFICATION FOR DETAILED TEST HOLE INFORMATION
 - THE LOCATION OF ALL SERVICES TO BE CONFIRMED IN THE FIELD
 - RENEW/REHABILITATE CATCHBASINS & RENEW CATCHBASIN LEADS WHERE NOTED



BID OPPORTUNITY NO. 980-2011

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	150 WM	WATERMAIN	150 WM	300 LDS	LAND DRAINAGE SEWER	300 LDS
300 LDS	LAND DRAINAGE SEWER	300 LDS	300 LDS	LAND DRAINAGE SEWER	300 LDS	250 WWS	WASTE WATER SEWER	250 WWS
250 WWS	WASTE WATER SEWER	250 WWS	250 WWS	WASTE WATER SEWER	250 WWS	○	MANHOLE	○
○	MANHOLE	○	○	MANHOLE	○	□	CATCH BASIN	□
△	CURB INLET	△	△	CURB INLET	△	---	CULVERT	---
+	COMMUNITY MAILBOX	+	+	COMMUNITY MAILBOX	+	+	SURVEY BAR	+
+	SURVEY BAR	+	+	SURVEY BAR	+	---	MODIFIED BARRIER CURB	---
---	MODIFIED BARRIER CURB	---	---	MODIFIED BARRIER CURB	---	---	RAMP CURB	---
---	RAMP CURB	---	---	RAMP CURB	---	---	CURB STOP	---
---	CURB STOP	---	---	CURB STOP	---	---	REDUCER	---
---	REDUCER	---	---	REDUCER	---	---	COUPLING	---
---	COUPLING	---	---	COUPLING	---	---	HYDRO	---
---	HYDRO	---	---	HYDRO	---	---	MTS	---
---	MTS	---	---	MTS	---	---	GAS	---
---	GAS	---	---	GAS	---	---	ASPHALT	---
---	ASPHALT	---	---	ASPHALT	---	---	CONCRETE	---
---	CONCRETE	---	---	CONCRETE	---	---	LIP CURB	---
---	LIP CURB	---	---	LIP CURB	---	---	CONCRETE	---
---	CONCRETE	---	---	CONCRETE	---	---	PAVEMENT CROWN	---
---	PAVEMENT CROWN	---	---	PAVEMENT CROWN	---	---	GROUNDLINE	---
---	GROUNDLINE	---	---	GROUNDLINE	---	---		---

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES DATE COMMITTEE

NOTE: LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	DATE	BY
0	ISSUED FOR CONSTRUCTION	11/11/20	KLC

AECOM

DESIGNED BY: JAT
DRAWN BY: KMB/KLC
HOR. SCALE: 1:250
VERT. SCALE: 1:50

CHECKED BY: [Signature]
APPROVED BY: [Signature]
RELEASED FOR CONSTRUCTION: [Signature]
DATE:

PROFESSIONAL'S SEAL
A. NAGY
REGISTERED PROFESSIONAL ENGINEER
CONSULTANT DRAWING NO. 60221826-01-C-101-ROX.dwg

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT

INSTALLATION OF CONCRETE CULVERT AT PEMBINA HIGHWAY & BEAUJOLAIS COULEE

CIVIL
PLAN/PROFILE OF CULVERT

SHEET 1 OF 3
CITY DRAWING NUMBER
C311-12-02
REV 0

Test Hole Logs

AECOM Canada Ltd.

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 groundwater 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 differ 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.

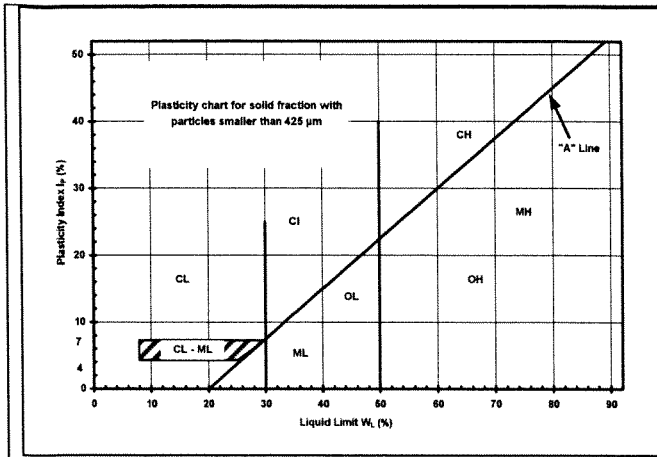
Since it is possible for conditions to vary from those assumed in the analysis and upon which our conclusions and recommendations are based, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction 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 $4 < W_p < 7$ $C_u = \frac{D_{60}}{D_{10}}$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$	
			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 ORGANIC SOILS		Peat and other highly organic soils		Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	
	Asphalt		Till			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	
Silt (non-plastic) or Clay (plastic)	< 0.075 mm		10-20	some
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 (S_u) of a cohesive soil can be related to its consistency as follows:

S_u (kPa)	CONSISTENCY
<12	very soft
12 – 25	soft
25 – 50	medium or firm
50 – 100	stiff
100 – 200	very stiff
200	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: Pembina Hwy Culvert Replacement	CLIENT: City of Winnipeg	TESTHOLE NO: TH 11-01
LOCATION: West Side of Culvert (N 0632714 / E 5514569)		PROJECT NO.: 60221826
CONTRACTOR: Maple Leaf Drilling Ltd.	METHOD: Acker MP-5, 125 mm SSA	ELEVATION (m): 228.00
SAMPLE TYPE	<input checked="" type="checkbox"/> GRAB <input type="checkbox"/> SHELBY TUBE <input checked="" type="checkbox"/> SPLIT SPOON <input type="checkbox"/> BULK <input type="checkbox"/> NO RECOVERY <input type="checkbox"/> CORE	

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS		UNDRAINED SHEAR STRENGTH		COMMENTS	ELEVATION
						* Becker * ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) ■ Total Unit Wt ■ (kN/m ³)	+ Torvane + × QU × □ Lab Vane □ △ Pocket Pen. △ ⊕ Field Vane ⊕ (kPa)				
0		TOPSOIL - trace rootlets, organics - black, dry	GRAB	G1							
1		CLAY (Alluvial) silty, trace to some gravel, trace rootlets - brown to grey, dry, stiff - intermediate to high plasticity - pockets of organics	SPLIT SPOON	S2	16	◆				- 4, 8, 8 blows	227
2		- moist - intermediate plasticity	GRAB	G3							226
3		Clay - - moist, brown, firm to soft	SPLIT SPOON	S4	11	◆		△		- 3, 4, 7 blows	225
4			SPLIT SPOON	T5							224
5		- silt inclusions - moist to wet, brown, soft - intermediate to high plasticity	SPLIT SPOON	S7	4	◆		△		- 1, 1, 3 blows	223
6			GRAB	G8							222
7		- dark brownish grey	GRAB	G9							221
8		- grey below 7.9 m	GRAB	G10							220
10											218
11		END of TEST HOLE AT 10.4 m ON SUSPECTED BOULDER Notes: 1. Auger refusal at 10.4 m. 2. Hole found dry after drilling. 3. Test hole backfilled with bentonite and auger cuttings upon completion.									217
12											216
13											216

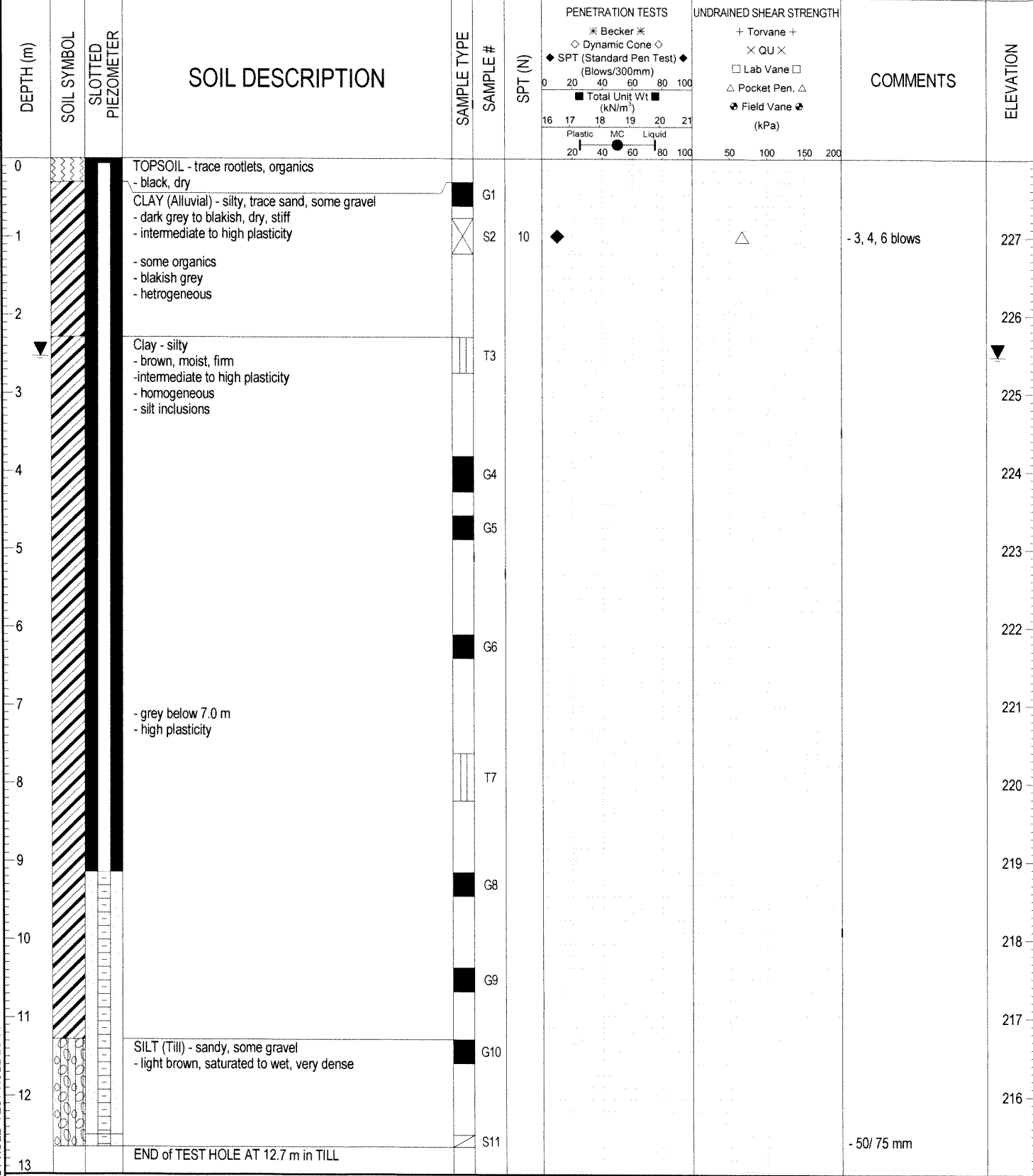
LOG OF TEST HOLE TEST HOLE LOGS - PEMBINA HWY.GPJ UMA WINN.GDT 11/25/11



LOGGED BY: O.Eissa	COMPLETION DEPTH: 10.36 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 9/29/11
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Pembina Hwy Culvert Replacement CLIENT: City of Winnipeg TESTHOLE NO: TH 11-02
 LOCATION: East Side of Culvert (N 0632771 / E 5514572) PROJECT NO.: 60221826
 CONTRACTOR: Maple Leaf Drilling Ltd. METHOD: Acker MP-5, 125 mm SSA ELEVATION (m): 228.00

SAMPLE TYPE GRAB SHELBY TUBE SPLIT SPOON BULK NO RECOVERY CORE
 BACKFILL TYPE BENTONITE GRAVEL SLOUGH GROUT CUTTINGS SAND



LOG OF TEST HOLE TEST HOLE LOGS - PEMBINA HWY.GPJ UMA WINN.GDT 11/25/11



LOGGED BY: O.Eissa COMPLETION DEPTH: 12.50 m
 REVIEWED BY: Faris Khalil COMPLETION DATE: 9/29/11
 PROJECT ENGINEER: Faris Khalil Page 1 of 2

PROJECT: Pembina Hwy Culvert Replacement		CLIENT: City of Winnipeg		TESTHOLE NO: TH 11-02		
LOCATION: East Side of Culvert (N 0632771 / E 5514572)				PROJECT NO.: 60221826		
CONTRACTOR: Maple Leaf Drilling Ltd.		METHOD: Acker MP-5, 125 mm SSA		ELEVATION (m): 228.00		
SAMPLE TYPE <input checked="" type="checkbox"/> GRAB		<input type="checkbox"/> SHELBY TUBE	<input checked="" type="checkbox"/> SPLIT SPOON	<input type="checkbox"/> BULK	<input type="checkbox"/> NO RECOVERY	<input type="checkbox"/> CORE
BACKFILL TYPE <input checked="" type="checkbox"/> BENTONITE		<input type="checkbox"/> GRAVEL	<input type="checkbox"/> SLOUGH	<input type="checkbox"/> GROUT	<input type="checkbox"/> CUTTINGS	<input type="checkbox"/> SAND

DEPTH (m)	SOIL SYMBOL	SLOTTED PIEZOMETER	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS		UNDRAINED SHEAR STRENGTH		COMMENTS	ELEVATION	
							* Becker * ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) ■ Total Unit Wt ■ (kN/m ³)	+ Torvane + × QU × □ Lab Vane □ △ Pocket Pen. △ ⊕ Field Vane ⊕ (kPa)					
13			Notes: 1. Water level measured at 4.6 m below surface upon completion of drilling. 2. Water level measured at 2.5 m below surface on October 23rd, 2011 2. Installed 25 mm diameter standpipe piezometer well at 12.7 m. Complete with 3.35 m of screen and 0.9 m stick up with above ground metal protector. Backfilled with sand to 9.14 m, bentonite to surface.									214	
14													213
15													212
16													211
17													210
18													209
19													208
20													207
21													206
22													205
23													204
24													203
25													203
26												203	

LOG OF TEST HOLE TEST HOLE LOGS - PEMBINA HWY.GPJ UMA WINN.GDT 11/25/11



LOGGED BY: O.Eissa	COMPLETION DEPTH: 12.50 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 9/29/11
PROJECT ENGINEER: Faris Khalil	Page 2 of 2