THE CITY OF WINNIPEG
GEOTECHNINCAL INVESTIGATION
FOR NORTH WEST DISTRICT
OUTFALL RECONSTRUCTION
BURROWS AVENUE OUTFALL

DECEMBER, 1995

PREPARED BY
UMA ENGINEERING LTD.
ENGINEERS AND PLANNERS
1479 BUFFALO PLACE
WINNIPEG, MANITOBA
R3T 1L7

UMA JOB NO. 41 02 3483 034 01 01

INTER OFFICE MEMO

UMA ENGINEERING LTD.

TO	C. Macey	**DATE**	December 18, 1995
FROM	K. Skaftfeld	**FILE NO.**	41 02 3483 034 01 01
RE	**BURROWS AVENUE OUTFALL**		

Attached is our geotechnical report summarizing the results of our slope stability analysis at the referenced site. Although continued creep movements of the riverbank can be expected, reconstruction of the end of the outfall pipe and possibly the inclusion of an outfall structure, will not adversely impact on existing bank stability. Recessed erosion protection and the incorporation of slip joints to accommodate ongoing movements are considered more cost effective than the construction of slope stabilization measures.

K. Skaftfeld, P. Eng.
Senior Project Engineer
UMA Environmental

KS/pmb

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SECTION 1.0
INTRODUCTION

This report summarizes the results of our geotechnical investigation and slope stability analyses for the Burrows Avenue Outfall. The purpose of the investigation was to assess the existing riverbank stability as it relates to performance and upgrading of the existing outfall pipe. The report also addresses foundation considerations for a proposed outlet structure and erosion protection along the shoreline. The terms of reference for this project are outlined in a memo to Chris Macey, P.Eng. dated December 21, 1992.

SECTION 2.0
FIELD INVESTIGATION

Test holes (TH) 92-1 and 92-2 were drilled on December 16, 1992 to Standard Penetration Test (SPT) refusal and auger refusal, respectively. Drilling was performed by Paddock Drilling using a track-mounted Mobile Drill Rig (Nodwell) equipped with 125 mm solid stem augers. General site supervision and test hole logging were performed by Darren Yarechewski, P.Eng. of UMA Engineering Ltd. Representative undisturbed (Shelby Tube) and disturbed (Split Barrel Sampler and Bag) samples were collected and returned to UMA Engineering Ltd.'s soils laboratory for further testing.

Pocket penetrometer and Torvane tests were performed at the ends of Shelby Tube samples to measure undrained shear strengths of cohesive soils. Soil consistency in the non-cohesive silt till was determined using Standard Penetration Tests. Test results are shown on the test hole logs in Appendix A. A standpipe piezometer was installed in TH 92-1 to measure short term groundwater levels in the silt till. Both test holes were backfilled upon completion.

TH's 92-3 and 92-4 were hand augered to a maximum depth of 2.7 m near the end of the outfall pipe to determine soil conditions below the edge of the Normal Summer Water Level (NSWL). Undrained shear strengths were measured with a field vane at regular depth intervals. Disturbed (bag) samples were collected for moisture content determination.

SECTION 3.0
LABORATORY TESTING

All soil samples were transported to UMA Engineering Ltd.'s soils laboratory in Winnipeg for testing. Tests included visual classification, moisture content, undrained shear strength (laboratory vane) and bulk density. The laboratory test results are presented on the individual test hole logs in Appendix A.

SECTION 4.0
SITE AND SUBSURFACE CONDITIONS

4.1 Site Conditions

The Burrows Avenue Outfall is located on an outside bend of the Red River south (upstream) of the Redwood Street Bridge. An arcuate failure scarp extends across the City right-of-way and across the properties to the north and south of the outfall and generally defines the top of the bank. Downslope of the top of the bank, the ground slopes towards the river at approximately 3H:1V to elevation 227.0 m and then 20H:1V to the edge of the NSWL at elevation 225.0 metres. The edge of the NSWL is defined by the limit of vegetation and a change in grade to about 1H:1V to elevation 223.5 metres. From this point to the edge of the ice the river channel slopes at approximately 13H:1V.

The bank geometry to the north and south of the outfall pipe is much more irregular with evidence of recent instabilities (trees leaning). This was confirmed by area residents who reported a significant slope failure downslope of an apartment building to the north of the outfall pipe upon fill placement along the bank several years ago. Slight cracking is evident around the northeast upper storey windows of the apartment building. Comparison of the City ROW and adjacent properties suggests that some regrading may have been performed within the City owned property where the outfall pipe is located.

4.2 Outfall Inspection

The outfall pipe was inspected by Amalgamated Pipe Services Inc. via video camera on January 27, 1993. The 2400 mm diameter outfall pipe, heading west, consists of corrugated steel pipe (CSP) between the river and pipe bend, followed by monolithic concrete between the bend and the gate chamber. The pipe bend is located at about Station 0+30 on Section A-A (Drawing 01). The pipe did not contain a slip joint at the bend. Noted pipe separations are shown on the Drawing 01. Circumferential pipe separations ranging from 3 to 15 mm wide were encountered along the monolithic concrete portion of the pipe. The CSP contained one joint separation at the lower north quadrant. The pipe was found to be 40% full of sediment in the vicinity of the river and 30% full at the gate chamber to the west.

4.3 Subsurface Conditions

4.3.1 Soil Stratigraphy

In descending order, the soil stratigraphy at TH's 92-1 and 92-2 consists of brown clay, grey clay, and silt till. The brown and grey clays are considered to be lacustrine deposits. Past disturbance is evident in the brown clay layers as evidenced by numerous slickensides. The surficial river channel soils at test holes 92-3 and 92-4 consist of black to brown silty lacustrine clay. Soil units are discussed in more detail as follows:

a) Upper Brown Clay (Lacustrine Deposit)

The upper brown clay encountered in TH's 92-1 and 92-2 is about 4 m and 3 m thick, respectively. The plasticity of the clay is estimated to be low to intermediate (CL to CI). Organic traces (roots) were encountered throughout. The structure of the clay deposit is generally homogenous with no signs of disturbance in the samples. Moisture contents range from 31 to 43 percent with an average of 34 percent. Undrained shear strengths range from 33 to 51 kPa with an average of 42 kPa indicating a firm consistency. The average bulk density is approximately 17 kN/m^3.

b) Lower Brown Clay (Lacustrine Deposit)

The lower portion of the brown clay layer extends from a depth of 4 m to 8.5 m in TH 92-1 and was not encountered in TH 92-2. The plasticity of the clay is estimated to be high (CH). Numerous slickensides were evident throughout, as indicated on the test hole log in Appendix A. The slickensides, which indicate the location of a well defined failure plane through the clay, ranged from 0 to 50 degrees from the horizontal. The structure was blocky from 4 to 5.2 m, and homogeneous below this depth. Moisture contents range from 47 to 56 percent with an average of 52 percent. Undrained shear strengths range from 54 to 129 kPa with an average of 91 kPa, indicating a stiff consistency. The average bulk density is in the order of 17 kN/m^3.

c) Grey Clay (Lacustrine Deposit)

A layer of highly plastic grey clay underlies the brown clay to depths of 10.4 and 10.2 metres in TH's 92-1 and 92-2, respectively. Horizontal slickensides were observed at a depth of 5.6 metres in TH 92-2. Slickensides were not evident in the samples collected from TH 92-1. The structure of the clay is generally homogenous, although the upper portion in TH 92-2 appeared blocky. Moisture contents ranged from 41 to 64 with an average of 52 percent. Undrained shear strengths range from 37 to 76 kPa with an average of 58 kPa. The average bulk density of the grey clay is approximately 17 kN/m^3.

d) Silt Till (Glacial Deposit)

Silt till was encountered in TH's 92-1 and 92-2 at a depth of 10.4 m and 10.2 m, respectively. The silt till contains increasing percentages of sand and gravel with increased depth. SPT results ranging from 11 blows per 300 mm near the till surface to 50 blows per 80 mm at a depth of 13.8 metres in TH 92-1, suggests that the consistency of the till increases from loose to dense with depth. Moisture contents in the till decreased from 12 to 9 percent in TH 92-1 and remained constant at 11 percent in TH 92-2. Auger refusal was encountered at 18.6 metres in TH 92-2. Seepage and sloughing conditions prevented sample recovery from 11-18 metres in TH 92-2.

e) River Channel Soils

The surficial river channel soils below NSWL consist of soft black to brown silty clay (test holes 92-3 and 92-4). Moisture contents range from 46 to 47 percent with an average of 51 percent. Undrained shear strengths range from 17 to 26 kPa with an average of 20 kPa.

4.3.2 Groundwater Conditions

Seepage was encountered within the clay in test hole 92-2 at a depth of 5.6 m and within the silt till in TH's 92-1 and 92-2. A standpipe piezometer was installed in TH 92-1 to measure groundwater levels in the silt till unit. A stabilized groundwater elevation of 222.0 metres was reached within 24 hours. It is important to recognize that the groundwater conditions encountered at the time of our drilling may fluctuate with seasonal and climatic changes or as the result of construction activities on the site.

SECTION 5.0
STABILITY ANALYSIS

Although a visual inspection of the slope and outfall pipe indicates that slope adjustments have occurred since the pipe was installed, a review of 1988 aerial photographs suggest that slope failures have historically occurred along this stretch of riverbank. The presence of numerous slickensides observed in undisturbed samples from TH's 92-1 and 92-2 supports the historical observations. Therefore, the slope is considered to be marginally stable and the existing factor of safety (FS) against slope instabilities is assumed to be close to 1.0. With this marginal stability, it is likely that creep movements of the bank will continue. However, the magnitude and rate of these movements cannot be accurately estimated without instrumentation (slope indicators).

Because of the retrogressive nature of local bank instabilities, creep movements can often trigger large and often sudden bank failures. However, local experience has shown that a significant capital expenditure, in excess of $250,000, is generally required to mitigate these instabilities through the construction of slope stabilization measures. It is our understanding that this level of effort is not likely justified in this particular case, where only the outfall pipe is immediately jeopardized by continued bank movements. A more cost effective approach would be to provide some minor erosion protection at the toe of the slope and incorporate slip joints along the length of the pipe to accommodate the anticipated movement of the riverbank.

To verify our assessment of marginal slope stability, a computerized analysis was performed to back-calculate the operating shear strengths of the soils. These values were then compared with traditional residual strengths developed from local experience. The operating soil strengths determined from the analysis could also be used for the design of remedial measures, if required in the future.

analysis could also be used for the design of remedial measures, if required in the future.

The Morgenstern-Price Method was used to back-calculate the operating residual soil strength parameters. Several iteratives were performed until a combination of cohesive and frictional strengths produced a factor of safety of approximately 1.0. A representative cross section through the riverbank was used for the analysis (Section A-A, Drawing 01). Failure surface geometries were fitted to coincide with the existing scarp and the depth and inclination of the slickensides. The piezometric surface was assumed to be about 3 metres below ground surface at the top of the bank. Below elevation 227 metres, the piezometric surface is coincident with the ground surface.

Two possible circular failure surfaces were evaluated; a circular failure surface tangent to the till surface (Case A) and a composite circular failure surface running preferentially along the till surface (Case B), as shown in Drawings 02 and 03, respectively. Both failure surfaces enter the slope at the existing scarp location and exit downslope of the edge of the NSWL. Using a residual cohesive intercept of 1 kPa and an angle of internal friction of 12 degrees, the calculated factor of safety for case A and B is 1.02 and 1.04, respectively. These shear strength parameters are within the range of published data and are consistent with local experience by UMA Engineering Ltd.

SECTION 6.0
OUTFALL CONSTRUCTION

6.1 Outfall Structure

Should an outfall structure be considered as part of the overall upgrading a number of factors must be considered relative to the development of an adequate foundation. The upper 2 metres of the channel clay are soft and not considered a competent bearing layer for shallow foundations. Rather, shallow foundations should be placed on the underlying firm clay, where an allowable bearing capacity of 36 kPa (750 psf) should be used for the design of shallow foundations. The strength of the foundation soils should be verified by qualified geotechnical personnel at the time of construction. Granular fills used to support the outfall structure should be compacted to a minimum of 98% of Standard Proctor Dry Density. A non-woven geotextile should be used to line the excavation.

Any disturbed, softened, loose or frozen material at the base of the excavation should be removed and replaced with compacted soil or weak concrete. Based on the allowable bearing capacity provided, settlement of the structure is expected to be in the order of 25 mm, providing the granular backfill is properly compacted. Grout pipes could be incorporated beneath the structure to allow future levelling, if required. Vertical deformations of the foundation soils and hence, outfall structure, can also be expected from frost heaving after fall drawdown. Flexibility should therefore be incorporated in the design of the structure, in particular at the connection with the outfall pipe, to allow for the anticipated vertical movement.

If post-construction settlement associated with shall foundations cannot be tolerated, consideration should be given to supporting the outfall on driven concrete or timber piles. However, the mobilization costs for pile driving equipment may be significant. Design capacities and installation recommendations for deep foundations can be provided if required.

6.2 Outfall Pipe

Should the outfall pipe be replaced as part of this year's work, or considered in the future, slip joints should be provided to allow continued longitudinal pipe movements. Slip joints should be installed at pipe transitions and where existing separations are evident. Design guidelines for slip joint locations can be provided once the alignment, pipe material, etc. have been finalized.

6.3 Erosion Protection

The outfall structure should be recessed into the bank as far as required to conform with the existing bank geometry. Riprap should be provided at the discharge point and on either side of the structure. If grouted riprap is used, a transition section of random riprap (not grouted) is recommended. A non-woven geotextile should be used in conjunction with the riprap.

SECTION 7.0
CONCLUSIONS AND RECOMMENDATIONS

- Visual inspections of the riverbank and undisturbed soil samples indicate that slope instabilities have occurred along this stretch of riverbank.

- Continued creep movements of the bank can be expected. However, the magnitude and rate of these movements cannot be accurately estimated without instrumentation (slope indicators).

- At this time, long term maintenance of the outfall pipe and structure are considered to be more cost effective than construction of slope stabilization measures.

- Shoreline protection is recommended to reduce the rate of erosion at the edge of the NSWL.

- If an outfall structure is proposed either shallow or deep foundations could be used. An allowable bearing capacity of 36 kPa (750 psf) can be used for shallow foundations placed on firm river channel soils at a depth of $\pm$ 2 metres. Qualified geotechnical personnel should inspect foundation soils prior to placement of granular backfill beneath the outfall structure to confirm design capacities.

- Slip joints should be incorporated in any outfall pipe replacement to accommodate continued bank movements.

Should you have any questions regarding this report, please contact either of the undersigned.

Respectfully Submitted,
UMA ENGINEERING LTD.

K. Skaftfeld, P.Eng.
Senior Geotechnical Engineer
Environmental Division

PROVINCE OF MANITOBA
K. M. SKAFTFELD
19-12-95
REGISTERED ENGINEER

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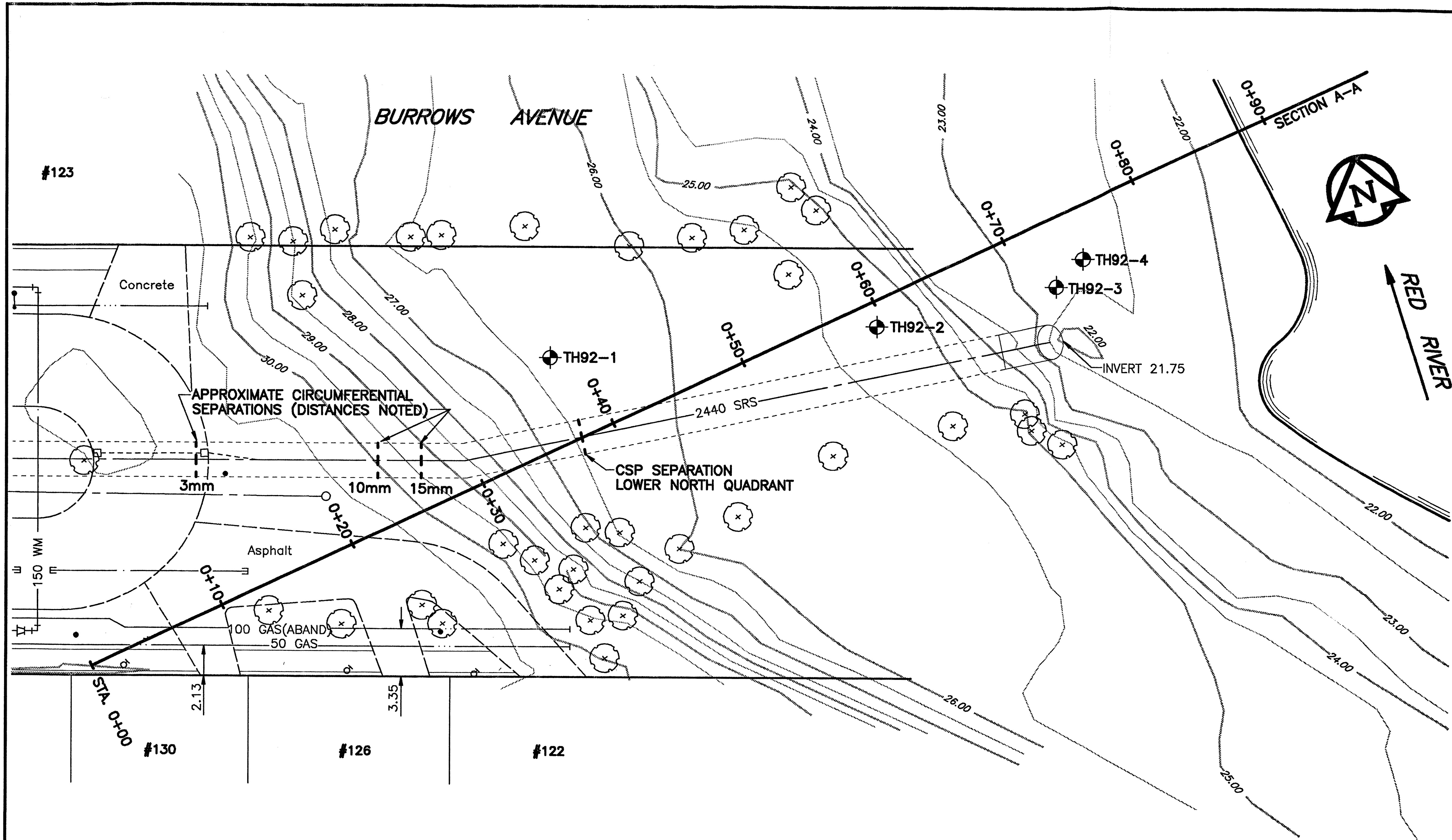
DRAWINGS



SCALE 1:250

UMA Engineering Ltd. Engineers & Planners 1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7	THE CITY OF WINNIPEG – Northwest District Burrows Avenue Outfall
TITLE:	Test Hole Location Plan
JOB No. 06-3483-034-01	DATE: March 2, 1993
DRAWN: DML	DWG. No. 01
CHECKED: DSY	

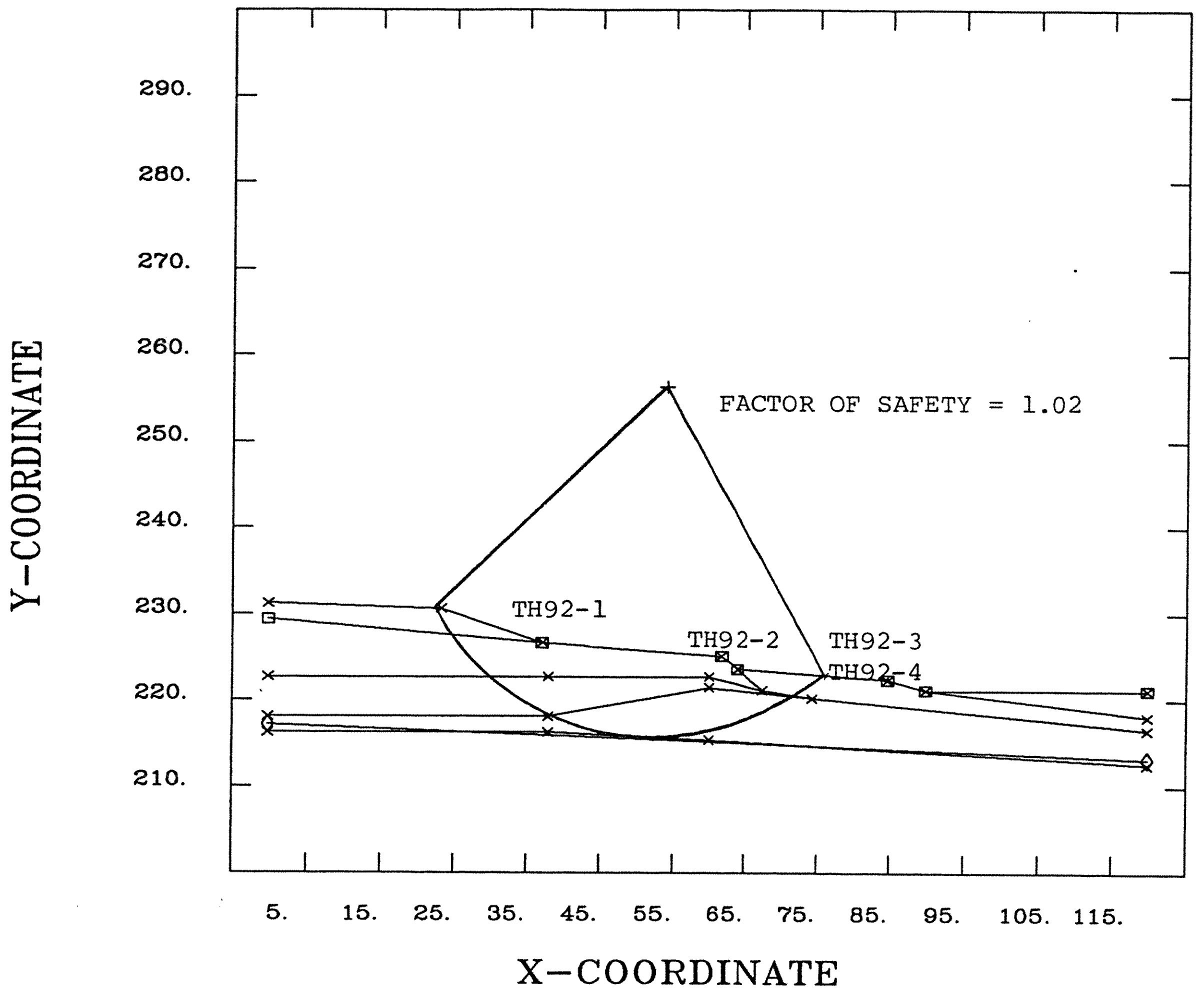
BUR02TH.dwg

CROSS-SECTION OF GEOMETRY

BURROWS AVENUE OUTFALL

JAN. 15, 1993

3483-034-01-01



UNIT WEIGHT	COHESION	PHI	DESCRIPTION
9.81	.00	.00	WATER
16.00	1.00	12.00	ALLUVIAL CLAY
17.00	1.00	12.00	UPPER BROWN CLAY
17.00	1.00	12.00	LOWER BROWN CLAY
18.00	1.00	12.00	GREY CLAY
−1.00	.00	.00	GLACIAL TILL

File name : B:BURROWS7.SET

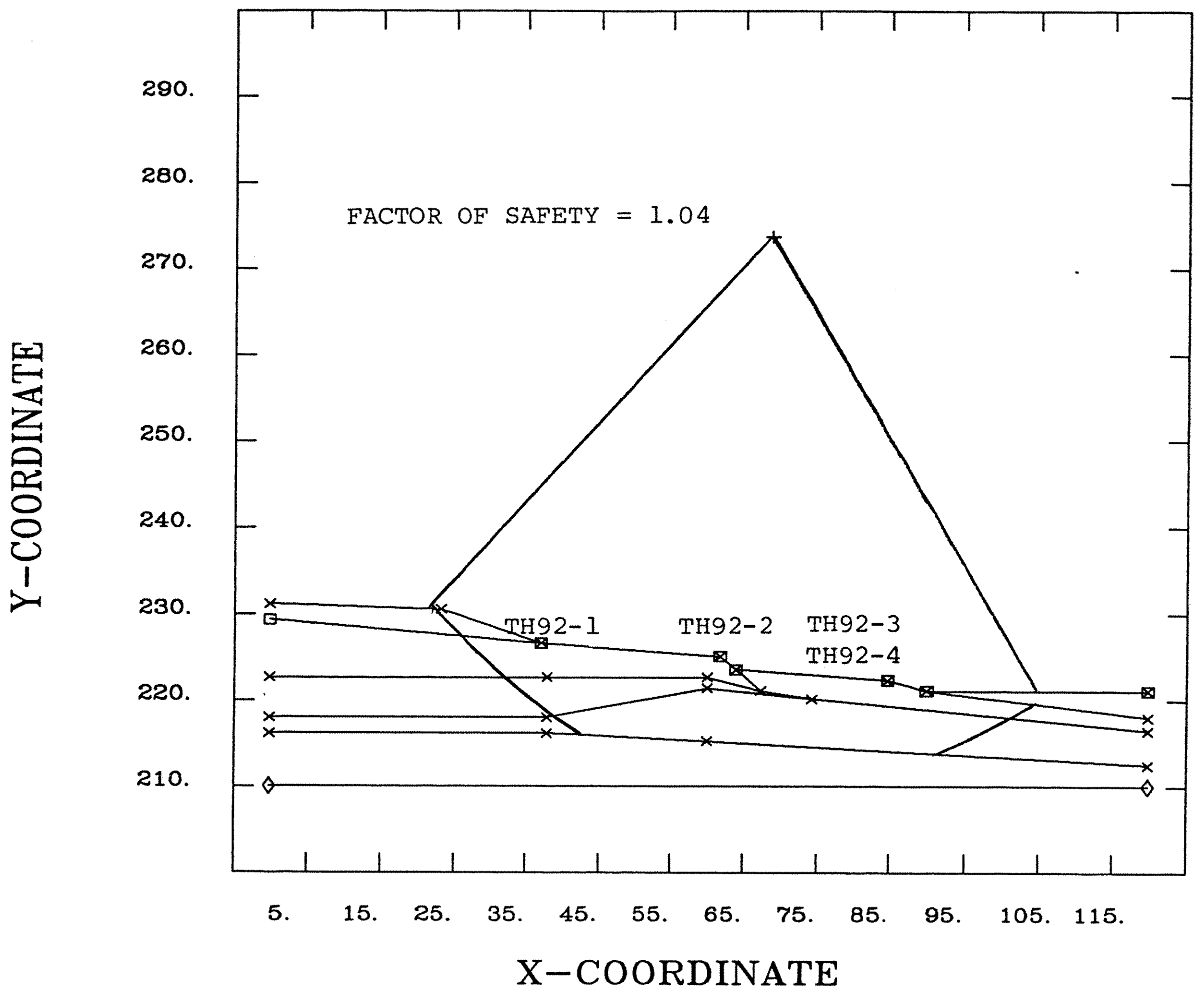
DRAWING 02

CROSS-SECTION OF GEOMETRY

BURROWS AVENUE OUTFALL

JAN. 15, 1993

3483-034-01-01



UNIT WEIGHT	COHESION	PHI	DESCRIPTION
9.81	.00	.00	WATER
16.00	1.00	12.00	ALLUVIAL CLAY
17.00	1.00	12.00	UPPER BROWN CLAY
17.00	1.00	12.00	LOWER BROWN CLAY
18.00	1.00	12.00	GREY CLAY
−1.00	.00	.00	GLACIAL TILL

File name : B:BURROWS8.SET

DRAWING 03

APPENDIX A
TEST HOLE LOGS

UMA ENGINEERING LTD.
EARTH SCIENCES DIVISION
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 represented 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.

APPENDIX

EXPLANATION OF FIELD & LABORATORY TEST DATA

The information presented on individual Test Hole Log Data Sheets is briefly described below:

NATURAL MOISTURE CONDITIONS & ATTERBERG LIMITS

The natural moisture content and Atterberg Limits are important in identifying soil types and properties. The water contents at the boundaries between the plastic and liquid states are termed the plastic and liquid limit respectively.

The liquid limit (LL) is determined by identifying the water content associated with the number of blows required to close the bottom of a 3 mm groove over a 13 mm length. The plastic limit (PL) is determined by identifying the water content of the soil when a 3 mm diameter thread of soil begins to crumble.

SOIL PROFILE & DESCRIPTION

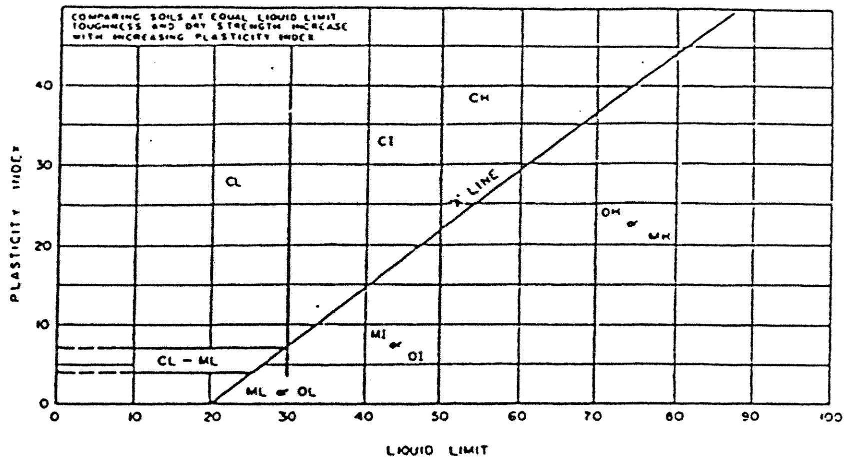
Each soil strata is classified and described noting any special conditions. The Unified Soil Classification system is used, and the soil profile is normally referenced to the existing ground elevation. When available the ground elevation is shown.

UNIFIED SOIL CLASSIFICATION SYSTEM

Description			Group symbols	Laboratory criteria			
				Fines (%)	Grading	Plasticity	Notes
Coarse grained (More than 50% larger than 63 μm BS sieve size)	Gravels (More than 50% of coarse fraction of gravel size)	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 $4 < PI < 7$
		Poorly-graded gravels, sandy gravels, with little or no fines	GP	0–5	Not satisfying GW requirements		
		Silty gravels, silty sandy gravels	GM	> 12		Below 'A' line or $PI < 4$	
		Clayey gravels, clayey sandy gravels	GC	> 12		Above 'A' line and $PI > 7$	
	Sands (More than 50% of coarse fraction of sand size)	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		
		Silty sands	SM	> 12		Below 'A' line or $PI < 4$	
		Clayey sands	SC	> 12		Above 'A' line and $PI > 7$	
Fine grained (More than 50% smaller than 63 μm BS sieve size)	Silts and clays (Liquid limit less than 50)	Inorganic silts, silty or clayey fine sands, with slight plasticity	ML	Use Plasticity Chart			
		Inorganic clays, silty clays, sandy clays of low plasticity	CL	Use Plasticity Chart			
		Organic silts and organic silty clays of low plasticity	OL	Use Plasticity Chart			
	Silts and clays (Liquid limit greater than 50)	Inorganic silts of high plasticity	MH	Use Plasticity Chart			
		Inorganic clays of high plasticity	CH	Use Plasticity Chart			
		Organic clays of high plasticity	OH	Use Plasticity Chart			
Highly organic soils		Peat and other highly organic soils	Pt				

NOTE: CI identified as an intermediate or medium plastic clay where the liquid limit lies between 30 and 50%.

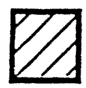
PLASTICITY CHART

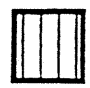


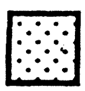
Individual test hole logs are detailed with one or a combination of the soil symbols shown below:


ORGANICS

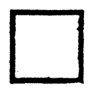

PEAT


CLAY


SILT


SAND


GRAVEL


BEDROCK

TESTS ON SOIL SAMPLES

Laboratory and field tests are identified by the following symbols:

q_u - unconfined compressive strength usually expressed in kilonewtons per square metre or alternatively kilopascals (kPa). This value is used in determining the allowable bearing capacity of the soil.

γd - dry unit weight expressed in grams per cubic centimetre. This value indicates the density or consistency of the in-situ soil.

T_v - undrained shear strength (kPa) using a Torvane.

pp - undrained shear strength (kPa) derived from pocket penetrometer testing.

L_v - undrained shear strength (kPa) using a lab vane.

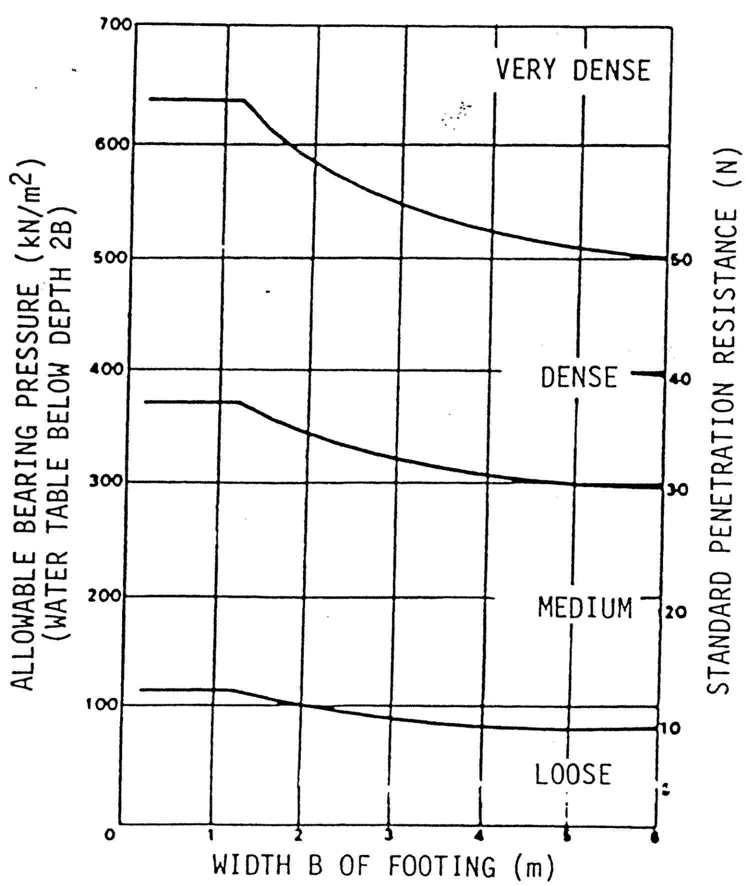
F_v - undrained shear strength (kPa) derived in-situ using a field vane.

N - standard penetration field test. This test is conducted in the field to determine the in-situ consistency of soil strata. The "N" value recorded is the number of blows 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 metres into the soil.

The resistance and unconfined compressive strength of a cohesive soil can be related to its consistency as follows:

N - BLOWS/0.30 m	q_u - kPa	CONSISTENCY
2	24	very soft
2 - 4	24 - 48	soft
4 - 8	48 - 96	medium or firm
8 - 15	96 - 192	stiff
15 - 30	192 - 383	very stiff
30	383	hard

The resistance of a non-cohesive soil (sand) can be related to its consistency as follows:



In addition, consolidation data sheets and gradation analysis sheets may be separately enclosed. The consolidation data sheets summarize test results on the consolidation characteristics of cohesive soils. The gradation analysis sheets graphically show grain size distribution of soil samples either by hydrometer or mechanical sieve tests or combination thereof.

GROUNDWATER TABLE

Groundwater conditions recorded at the time of site drilling may or may not represent static groundwater levels. Seasonal variations in the groundwater regime may be recorded by the installation and long time monitoring of standpipe or pneumatic type piezometers.

SAMPLE TYPE

A:	Split Spoon
B:	Shelby Tube
C:	Piston Sampler
D:	Core Barrel
E:	Auger
F:	Wash
G:	Bulk Sample
H:	Block Sample

CLASSIFICATION CODE

Clay	<0.005 mm
Silt	0.005 mm - #200 Sieve
Sand	#200 - #4 Sieve
Gravel	#4 Sieve - 75 mm
Cobbles	75 - 300 mm
Boulders	>300 mm

GRADATION DESCRIPTIVE TERMS

and	40 - 50%
with	30 - 40%
some	20 - 30%
little	10 - 20%
trace	0 - 10%

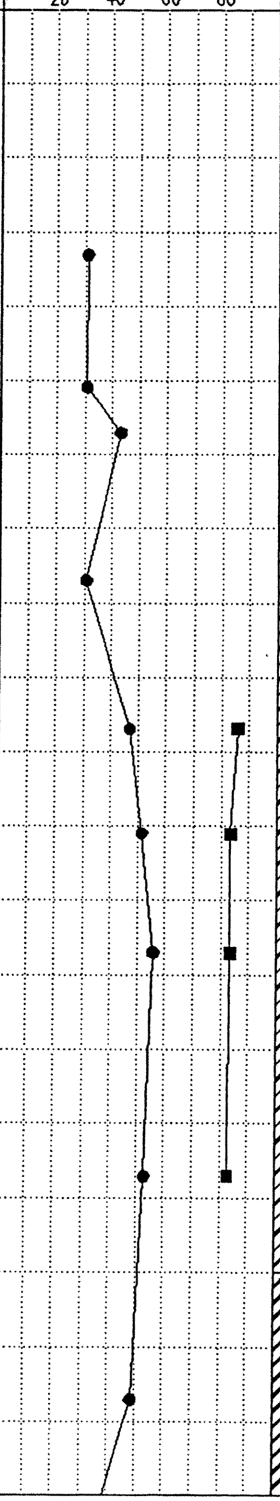
APPENDIX A

TEST HOLE LOGS

BURROWS AVENUE OUTFALL	DRILLED BY: PADDOCK DRILLING	TESTHOLE No: 92-1
CITY OF WINNIPEG	DRILL TYPE: NODWELL, 125 mm SS AUGERS	Project No: 3483-034-01-01
PROJECT ENGINEER: KS		ELEVATION: 226.400 (m)

SAMPLE TYPE: GRAB SAMPLE / SHELBY TUBE / SPT / NO RECOVERY / CORE BARREL / WIRELINE-TYPE

DEPTH (m) | BULK DENSITY (kN/m3) 4 8 12 16 | PLASTIC M.C. LIQUID 20 40 60 80 | SOIL SYMBOL | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NO | SPT(N) | SLOTTED PIEZOMETER | PP SHEAR STRENGTH (kPa) 30 60 90 120 | TV SHEAR STRENGTH (kPa) 30 60 90 120 | LV SHEAR STRENGTH (kPa) 30 60 90 120 | FV SHEAR STRENGTH (kPa) 30 60 90 120 | ELEVATION(m)



0.0 m – 4.0 m

CLAY – light to dark brown
- silty, trace sand and gravel
- trace of organics
- moist, firm
- low to medium plasticity

4.0 m – 8.5 m

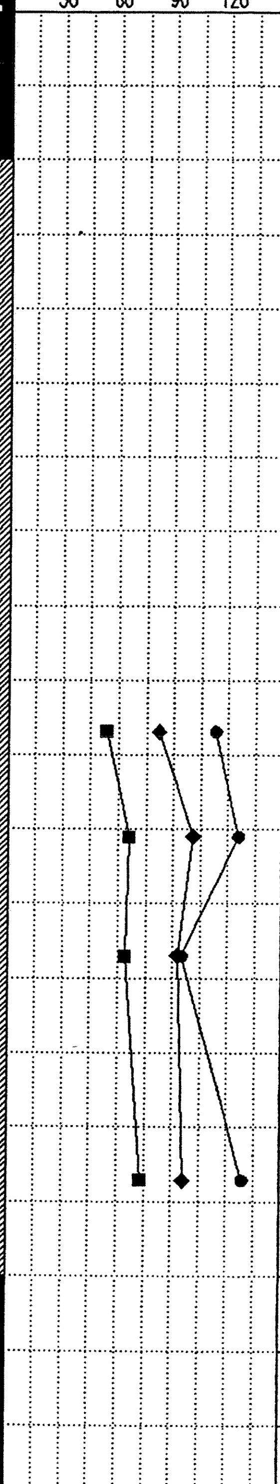
CLAY – brown, some grey mottling at 6.5 m
- silty, trace of sand and gravel
- moist, stiff, medium plasticity
- some carbonate inclusions
- upper 1.2 m blocky structure
- homogeneous below 5.2 m
- near vertical laminations with 1–2 mm spacing at 7.6 m
- slickensides as follows:

Sample No.	Inclination (from horizontal)
5	1 at 35 degrees
6	3 at 50 degrees
7	1 at 10 degrees
	1 at 45 degrees
8	1 at 15 degrees

8.5 m – 10.0 m

CLAY – grey
- silty
- moist, firm
- homogeneous
- high plasticity

Sample No: 1, 2, 3, 4, 5, 6, 7, 8, 9



Depth (m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0

Elevation (m): 226.4, 224.4, 222.4, 220.4, 218.4, 216.4

UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 13.8 m
	REVIEWED BY: KS	COMPLETE: 92/12/16
	Fig. No:	Page 1 of 2

BURROWS AVENUE OUTFALL	DRILLED BY: PADDOCK DRILLING	TESTHOLE No: 92-1
CITY OF WINNIPEG	DRILL TYPE: NODWELL, 125 mm SS AUGERS	Project No: 3483-034-01-01
PROJECT ENGINEER: KS		ELEVATION: 226.400 (m)

SAMPLE TYPE: GRAB SAMPLE · SHELBY TUBE · SPT · NO RECOVERY · CORE BARREL · WIREL‌INE-TYPE

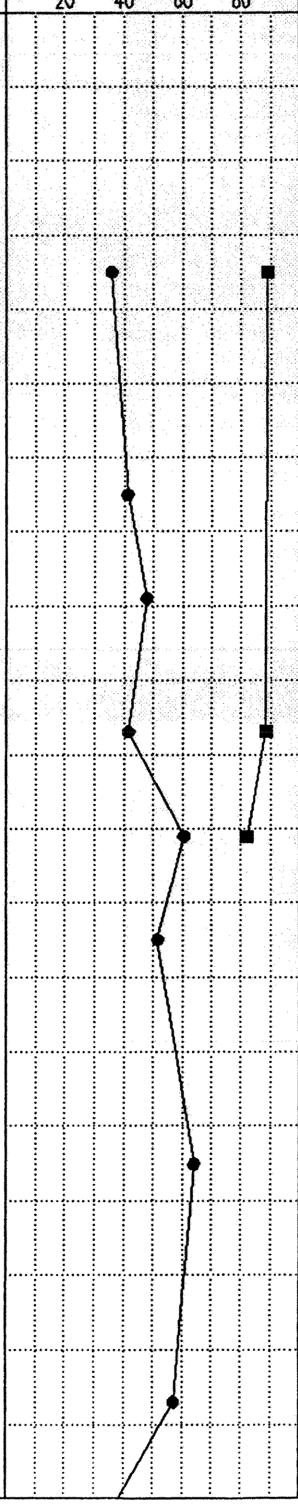
DEPTH (m)	BULK DENSITY (kN/m3) 4 8 12 16 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT(N)	SLOTTED PIEZOMETER	PP SHEAR STRENGTH (kPa) 30 60 90 120 / TV SHEAR STRENGTH (kPa) 30 60 90 120 / LV SHEAR STRENGTH (kPa) 30 60 90 120 / FV SHEAR STRENGTH (kPa) 30 60 90 120	ELEVATION (m)
10.0									216.4
10.4–13.8			SILT TILL – light brown – with sand and gravel – dense						
11.0									
11.7				SPT	10	11			
12.0									214.4
13.0									
13.8			NOTE:–FOR SAMPLE NO. 11, N = 50 FOR 8 cm	SPT	11	50			
14.0			END OF HOLE AT 13.8 m NOTE:–STANDPIPE PIEZOMETER INSTALLED IN SILT TILL						212.4
15.0									
16.0									210.4
17.0									
18.0									208.4
19.0									
20.0									206.4

UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 13.8 m
	REVIEWED BY: KS	COMPLETE: 92/12/16
	Fig. No:	Page 2 of 2

BURROWS AVENUE OUTFALL	DRILLED BY: PADDOCK DRILLING	TESTHOLE No: 92-2
CITY OF WINNIPEG	DRILL TYPE: NODWELL, 125 mm SS AUGERS	Project No: 3483-034-01-01
PROJECT ENGINEER: KS		ELEVATION: 225.300 (m)

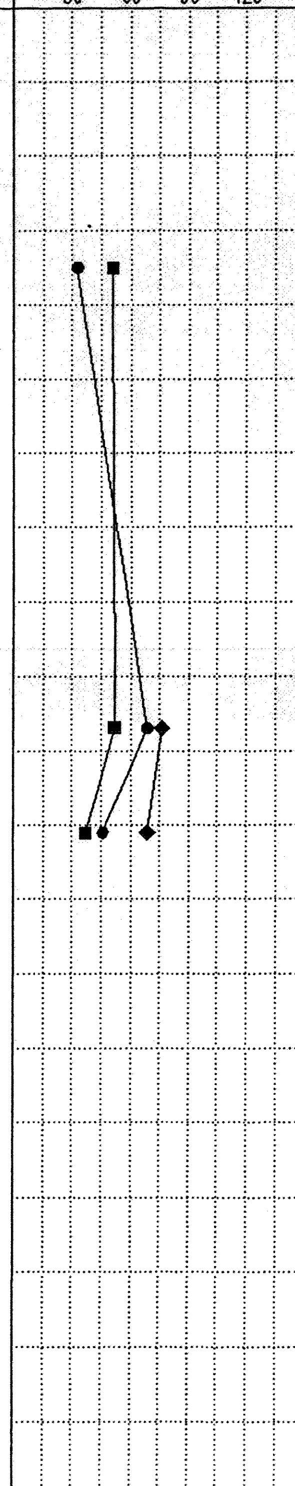
SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPT, NO RECOVERY, CORE BARREL, WIRELINE-TYPE

DEPTH (m) | ■ BULK DENSITY (kN/m3) ■ 4 8 12 16 / PLASTIC M.C. LIQUID 20 40 60 80 | SOIL SYMBOL | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NO | SPT(N) | ● PP SHEAR STRENGTH (kPa) ● / ■ TV SHEAR STRENGTH (kPa) ■ / ◆ LV SHEAR STRENGTH (kPa) ◆ / ▲ FV SHEAR STRENGTH (kPa) ▲ 30 60 90 120 | ELEVATION (m)



Depth (m)	Soil Description	Sample No
0.0	CLAY – reddish brown – silty, trace of fine sand – trace of organics (roots) – moist, firm – medium to low plasticity – homogeneous	1
3.0	CLAY – grey – silty – moist, firm – blocky structure and mottled brown to 4 m – homogeneous and grey below 4 m – trace of silt inclusions – medium to high plasticity – horizontal slickensides at 5.6 m	2, 3, 4, 5, 6, 7, 8

Depth scale: 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0



Elevation scale: 225.3, 223.3, 221.3, 219.3, 217.3, 215.3

UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 18.6 m
	REVIEWED BY: KS	COMPLETE: 92/12/16
	Fig. No:	Page 1 of 2

BURROWS AVENUE OUTFALL	DRILLED BY: PADDOCK DRILLING	TESTHOLE No: 92-2
CITY OF WINNIPEG	DRILL TYPE: NODWELL, 125 mm SS AUGERS	Project No: 3483-034-01-01
PROJECT ENGINEER: KS		ELEVATION: 225.300 (m)

SAMPLE TYPE: GRAB SAMPLE | SHELBY TUBE | SPT | NO RECOVERY | CORE BARREL | WIRELINE-TYPE

DEPTH (m)	■ BULK DENSITY (kN/m3) ■ 4 8 12 16 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT(N)	● PP SHEAR STRENGTH (kPa) ● / ■ TV SHEAR STRENGTH (kPa) ■ / ◆ LV SHEAR STRENGTH (kPa) ◆ / ▲ FV SHEAR STRENGTH (kPa) ▲ 30 60 90 120	ELEVATION(m)
10.0			SILT TILL – light brown – some sand and gravel (% increasing with depth) – dense					215.3
11.0				SPT	9	40		
12.0								213.3
13.0								
14.0								211.3
15.0								
16.0								209.3
17.0								
18.0				GRAB SAMPLE	10			207.3
19.0			AUGER REFUSAL AT 18.6 m NOTE:–GROUNDWATER SEEPAGE AT 5.6 m –NO SAMPLE RECOVERY FROM 11 TO 18 m DUE TO SLOUGHING CONDITIONS					
20.0								205.3

UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 18.6 m
	REVIEWED BY: KS	COMPLETE: 92/12/16
	Fig. No:	Page 2 of 2

BURROWS AVENUE OUTFALL	DRILL TYPE: 50 mm HAND AUGER	TESTHOLE No: 92-3
CITY OF WINNIPEG		Project No: 3483-034-01-01
PROJECT ENGINEER: KS		ELEVATION: 222.900 (m)

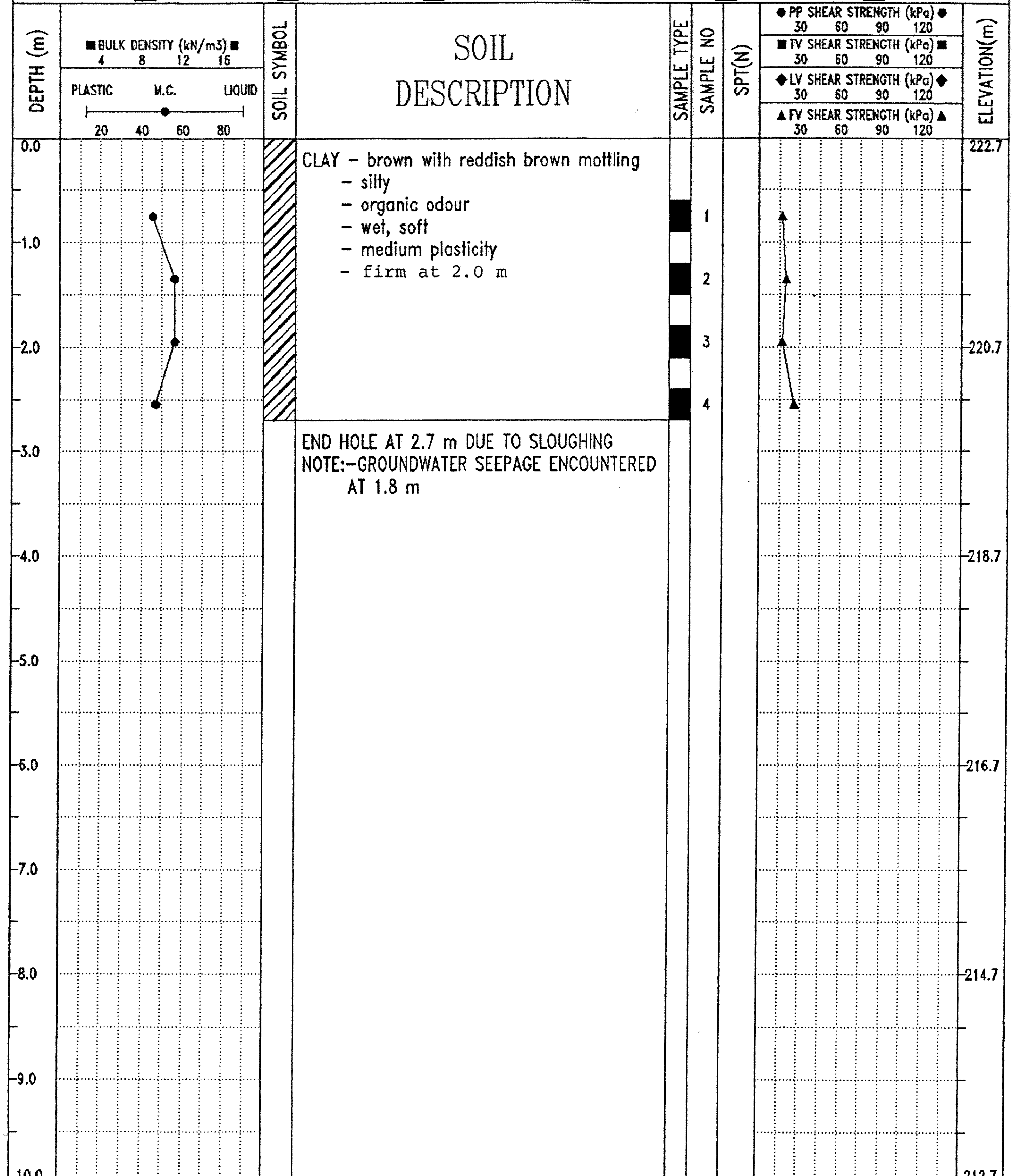
SAMPLE TYPE: GRAB SAMPLE | SHELBY TUBE | SPT | NO RECOVERY | CORE BARREL | WIRELINE-TYPE

DEPTH (m)	■ BULK DENSITY (kN/m3) ■ 4 8 12 16 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT(N)	● PP SHEAR STRENGTH (kPa) ● / ■ TV SHEAR STRENGTH (kPa) ■ / ◆ LV SHEAR STRENGTH (kPa) ◆ / ▲ FV SHEAR STRENGTH (kPa) ▲ 30 60 90 120	ELEVATION(m)
0.0			CLAY - black - silty - organic odour - wet, soft - medium plasticity REFUSAL AT 0.5 m ON SILTY SAND AND GRAVEL NOTE:-NO GROUNDWATER SEEPAGE ENCOUNTERED					222.9
-1.0								
-2.0								220.9
-3.0								
-4.0								218.9
-5.0								
-6.0								216.9
-7.0								
-8.0								214.9
-9.0								
10.0								212.9

UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 0.5 m
	REVIEWED BY: KS	COMPLETE: 92/12/17
	Fig. No:	Page 1 of 1

BURROWS AVENUE OUTFALL	DRILL TYPE: 50 mm HAND AUGER	TESTHOLE No: 92-4
CITY OF WINNIPEG		Project No: 3483-034-01-01
PROJECT ENGINEER: KS		ELEVATION: 222.700 (m)

SAMPLE TYPE: GRAB SAMPLE | SHELBY TUBE | SPT | NO RECOVERY | CORE BARREL | WIRELINE-TYPE



CLAY - brown with reddish brown mottling
- silty
- organic odour
- wet, soft
- medium plasticity
- firm at 2.0 m

END HOLE AT 2.7 m DUE TO SLOUGHING
NOTE:-GROUNDWATER SEEPAGE ENCOUNTERED AT 1.8 m

UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 2.7 m
	REVIEWED BY: KS	COMPLETE: 92/12/17
	Fig. No:	Page 1 of 1