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Memorandum

To	Eric Loewen	Page	1
CC			
Subject	Elm Park Pedestrian Bridge – Geotechnical Investigation		
From	Faris Khalil		
Date	December 11, 2009	Project Number	60119229 2940 394 00 (4.6.1)

Introduction

The City of Winnipeg is planning condition improvements to the existing Elm Park pedestrian bridge. It is understood that the proposed improvements will include the abandonment of the last south span and the south abutment by extending the approach fill towards the south bent-pier. A soil retention structure will be required to provide adequate lateral support and containment of the approach fill. The existing bent-piers will be maintained and no additional foundation elements will be introduced. A new approach slab is to be constructed between the bridge and the proposed approach fill to accommodate the potential for differential displacement.

This memorandum summarizes the results of the geotechnical investigation and provides geotechnical assessment and recommendations pertaining to foundation, approach fill and riverbank stability. Also, this memorandum serves as part of the documents supporting Waterways permit application.

Available Information and Site Inspection

The south end of the bridge is supported by a concrete abutment, two bent-piers and a concrete pier (Pier No. 4). The bent-pier structures are shown on Photo 01, these structures are not shown on the drawing B-5986-2, attached, made available by the City of Winnipeg. The south abutment is mostly buried and only the upper 0.5 m is visible, Photo 02. The drawing B-5986-2 illustrates the general arrangement of the bridge and test holes data from the 1964 drilling. AECOM reviewed this drawing and completed a site inspection on October 1st 2009 to assess the site condition, define the scope of the geotechnical work and develop the field investigation program. A second site inspection was completed by AECOM's structural group on October 30th 2009 to investigate the foundation type and condition of the existing bent-piers. Further discussion of the inspection details and foundation assessment is provided below.

A community ring dyke has been constructed along the riverbank east and west of the bridge. The dyke terminates near the south abutment on both sides and changes alignment towards the south to integrate with the bridge approach fill.

Figure 01 shows a contour map for the site and provides cross sections at the vicinity of the south abutment based on recent survey completed by AECOM. The geometry of the existing riverbank consists of two slopes and a terrace 10 to 12 m wide. The upper slope is about 2.5 m high with a side slope of approximately 3 horizontal to 1 vertical (3H:1V). The lower slope is about 4 m above the water level in the river and at an inclination of 3.5 H:1V. The terrace is gently sloped towards the river and is wider west of the bridge than east of the bridge. Under the bridge, the slope in front of the existing abutment is significantly flatter at about 9H:1V. No visible signs of instability or disturbance were observed on the upper slope within 20 m on either side of the bridge. A rip-rap protection/stabilization layer was observed on the face of the lower slope along the riverbank. A head scarp about 0.8 m high was observed along the crest of the lower slope of the riverbank on the west side of the bridge, Photo 03. It could not be determined if the scarp is associated with localized instability of the riverbank or if it is the exposed riverbank above the riprap. No visible signs of instability were observed in front of Pier 4 and along the riverbank east of the bridge. The area is covered with short grass and occasional medium shrubs. Tall trees were observed on the east side of the bridge.

The October 30th 2009 site inspection was completed by AECOM's structural group to investigate the foundation type and condition of the existing bent-pier. An excavation 2.4 m below existing grade was dug around one of the steel columns as shown on Photo 04. The steel column is supported on a stepped footing consisting of a concrete pedestal 450 x450 x600 mm (length x width x depth) which in turn is supported on a larger concrete block 960 x 960 mm and greater than 1200 mm in depth. The excavation was terminated above the base of the lower concrete block to protect against undermining the footings. There were no visible indications of a tie beam between the footings. It could not be confirmed if the footing is supported on piles or directly on the soil. The conditions of the footing and the steel columns were reported as satisfactory. AECOM's structural group has confirmed with City's personnel that there have been no known performance issues related to the bent-piers nor are there any observed displacements of the section of the bridge supported on the bent-piers.

It is our assessment that the existing bent-piers are adequate to support the proposed improvement at the south end span based on the following:

- Historically, the performance of the bent-piers has been satisfactory, according to City personnel.
- The conditions of the steel columns and the foundation concrete are satisfactory as confirmed by the field inspection performed by AECOM's structural group.
- The proposed improvement works will reduce the loading on the bent-piers compared to the existing condition.

Geotechnical Investigation

On October 20th 2009 two test holes (TH09-01 and 09-02) were drilled at the locations shown on Figures 01. Drilling was carried out by Paddock Drilling Ltd. using a track mounted drill rig equipped with 125 mm solid stem augers. Disturbed samples from auger cuttings and relatively undisturbed samples were collected at regular intervals. All soils observed during drilling were logged and visually classified on site by AECOM personnel.

TH09-01 was located close to Pier No 4 (south pier) and TH09-02 was located in the vicinity of the south bent-pier, both test holes were on the west side of the bridge. Drilling was advanced to auger refusal into till at 11.3 and 11.6 m below existing grade for TH09-01 and 09-02, respectively. A standpipe piezometer equipped with Casagrande tip was installed in TH09-02 in the till at 11.3 m below ground surface to facilitate groundwater level measurements.

Soil samples recovered during drilling were transported to AECOM's Materials Testing Laboratory in Winnipeg for further visual examination and testing. Laboratory testing consisted of determination of moisture contents, Atterberg limits, unit weight, and undrained shear strength. Detailed test hole logs have been prepared to record the description and the relative position of the various soil strata, location of samples obtained, field and laboratory test results, piezometer installation details and other pertinent information. Observations of any occurrence of sloughing and seepage during drilling are also recorded. The test hole logs are attached.

Subsurface Conditions

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

- Fill
- Alluvial Clay
- Till

These soils are described as follows:

Fill

About 0.4 m of clay fill was encountered at ground surface in TH09-02.

Alluvial Clay

Alluvial clay was encountered at ground surface in TH09-01 and beneath the fill in TH09-02. The clay extends to the glacial till at 10.3 m below existing ground surface in both test holes. The alluvial clay is silty, sandy and contains inclusions, seams and pockets of sand, silt and organics at various elevations. At the bottom of the alluvial clay deposit (i.e., interface with the till) the clay contains a trace gravel. Generally, the clay is moist and firm. Undrained shear strength measured from unconfined compression test ranged from 30 to 42 kPa. The moisture content of the clay increases with depth from 33 to 54 percent. The clay is of medium to high plasticity based on average liquid limit and plasticity index of 58 and 36 percent, respectively.

Till

Till was encountered beneath the clay and extend to the depth explored. Auger refusal was encountered at 11.3 and 11.6 m below ground surface in TH 09-01 and 09-02, respectively. Predominantly, the till consists of silt and sand and it contains variable amounts of clay and gravel. The till is light brown, firm, moist to wet and of low plasticity to non plastic. Moisture contents were measured at 18.5 and 24 percent.

Sloughing and seepage were observed in the till during drilling. Immediately after drilling the groundwater levels were at 3.8 and 4.3 m below existing grade in TH09-01 and 09-02, respectively. Groundwater measurement in the standpipe piezometer installed in TH09-02 was at 3.5 m below existing grade or at El. 223.8 m on December 1st 2009. These levels may not have stabilized and it should also be recognized that groundwater levels may fluctuate annually, seasonally or due to construction activities.

Approach Fill

The proposed approach fill at the south end of the existing bridge will need to be retained by a retaining system. A retaining wall behind the bent-piers is preferable over a fill that slopes towards the river because this fill will reduce the stability of the riverbank. The retaining wall will not obstruct future inspections and maintenance works of the bridge's sub-structures. Also, the limited headroom beneth the end span would make the placement and compaction of the fill difficult and may lead to long term performance issues (e.g. head slope instability, settlement and erosion). Therefore a soil retaining system is considered the preferred application to provide lateral support and containment for the proposed fill.

The most feasible and less disturbing alternative among available soil retaining systems is a reinforced soil mass which will not require extensive foundation preparation and can be integrated with the existing riverbank and dyke. A geogrid reinforced soil mass can be used to construct vertical or near vertical wall that provide the required support for the proposed fill. The wall facing system could consist of concrete blocks or gabion cells that offer the desired objectives including stability, durability, erosion resistance and aesthetics.

It is understood that the fill will be less than 2.5 m high, therefore the settlement of the of the fill and compression of the riverbank soils below is expected to be relatively low (in the order of 100 mm) and can be accommodated by the new approach slab.

The detailed design and other construction details should be completed by the Contractor according to the Manufacturer's recommendations. AECOM request the opportunity to review the wall design and shop drawings. Also, it is recommended that AECOM provide construction inspection so that design assumptions can be confirmed.

Riverbank Stability

The proposed work is within 107 m (350 ft) of the Red River and therefore falls under the jurisdiction of the Waterways Authority and will require a Waterways permit.

Slope stability assessment was undertaken to investigate if the proposed work will have an adverse impact on the existing stability of the riverbanks. As part of the assessment the following documents have been reviewed:

- UMA/AECOM Report “City of Winnipeg – St. Vital Park – Riverbank Stability Study and Functional Design of Stabilization Measures” dated December 2006,
- KGS Group Report “City of Winnipeg - Community Ring (Secondary) Dike Sites – Conceptual Design Report” dated May 2000.

Stability analysis was completed using Geostudio software developed by GeoSlope International Ltd. The scope of the assessment is limited to the south riverbank section immediately under the existing bridge (approximately a 10 m long section). No assessment was undertaken for the stability of the riverbanks beyond this section because no additional fill is planned in these areas. The stability of the riverbank was analyzed for the current and proposed slope geometry using soil strength parameters based on correlation with measured soil indices. These parameters are within the range of the localy acceptable values. The groundwater conditions are based on the levels measured at site and on local experience. The strength parameters and groundwater conditions used in the analysis are summarized in Table 1. The analysis assume the minimum water level in the river at 222.3 (i.e., ice level). The geometry of the riverbank was modeled based on the recent survey completed by AECOM and from the information shown on the drawing B-5986-2 (for underwater geometry).

Table 01: Soil Strength Parameters for Slope Stability Analysis

Soil Type	Cohesion (kPa)	Friction Angle (degrees)	Bulk Unit Weight (kN/m^3)	Groundwater Condition (Piezometric Elevation) (m)
Granular Fill	0	33	20.0	Max 225.0 and linearly match river water level
Alluvial Clay	0	24	18.0	
Till	10	30	22.0	224.0
Rock)	Impenetrable			NA

Four slip surfaces (No 1, 2, 3 and 4) were selected at different set back distances from the top edge of the riverbank to assess stability under existing and future conditions. The analysis assumes no contribution from the existing piers or from the reinforcement of the proposed reinforced earth wall. The results of the stability analysis are attached (Figure 02 to 03) and summarized in Table 02. The results indicate that the stability of the slip surfaces encompass the proposed fill (i.e., No. 3 and 4) is greater than the stability of the slip surfaces between the fill and the riverbank (i.e., No. 1 and 2). The placement of fill (to El 230.0 m) underneath the existing bridge between the south bent-pier and the south abutment will not adversely impact the stability of the critical slip surfaces No. 1 and 2. The change in the stability of slip surfaces No. 3 and 4 is estimated to be 4 and 9 percent, respectively. The calculated factor of safety (FS) for slip surfaces No. 3 and 4 after fill placement is greater than 1.5 which is typically set as a design objective.

Table 02: Summary of the Slope Stability Analysis

Case		FS against slope instability				Figure No
		Slip Surface No.				
		1	2	3	4	
Intact soil parameters	Existing Geometry	1.36	1.51	1.67	1.79	02
	Proposed Geometry	1.36	1.51	1.61	1.64	03
	Percent Change	No change	No change	4	9	-

The stability analysis demonstrates that the proposed work will not adversely impact the stability of the riverbank. Should riverbank instability develop in the vicinity of the bridge, its impact on the existing bridge should be reviewed. It is recommended that an inspection and assessment of the riverbank be performed annually to determine if further work is required to protect against slope instability.

If we can be of further assistance, please contact the undersigned.

Reviewed by

Faris Khalil, M.Sc., P.Eng.
Senior Geotechnical Engineer
Environment
/dh

Jeff Tallin, P.Eng
Senior Geotechnical Engineer
Environment

Photo 01: South End of the Existing Bridge, Looking East

Photo 02: South Abutment and Bent-Piers

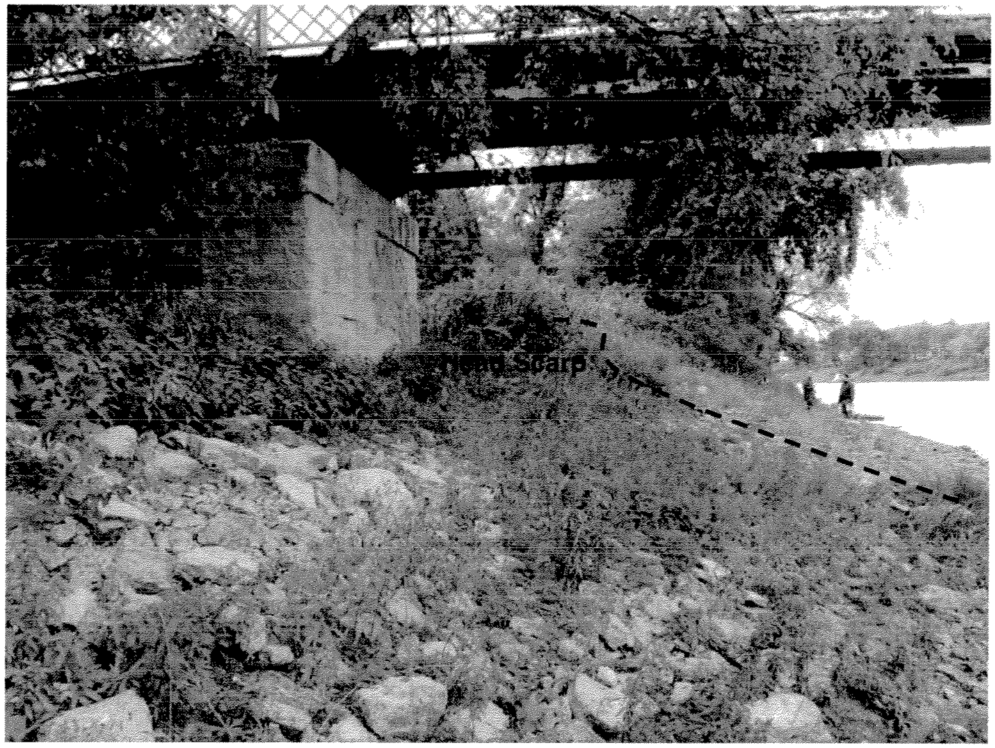


Photo 03: Head Scarp at the West Side, Looking West

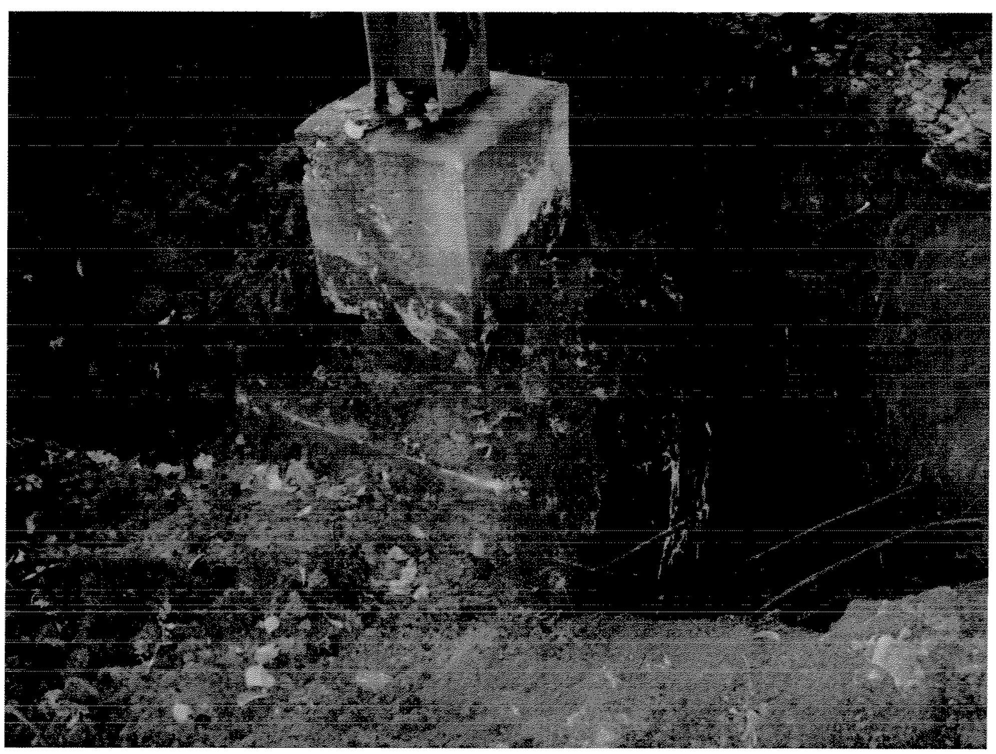
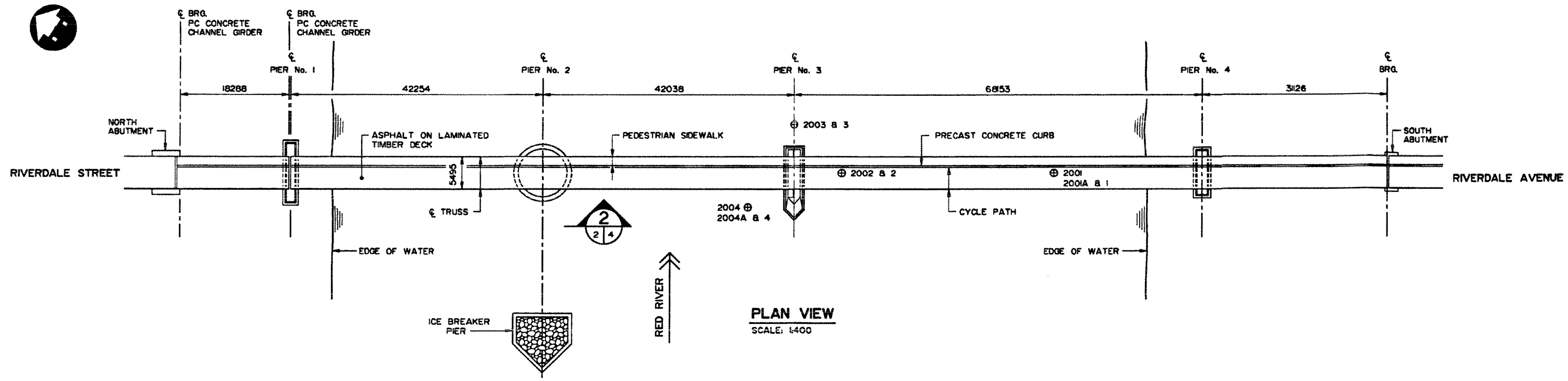


Photo 04: Concrete Footing at Bent-Pier

WO 101-84



PLAN VIEW
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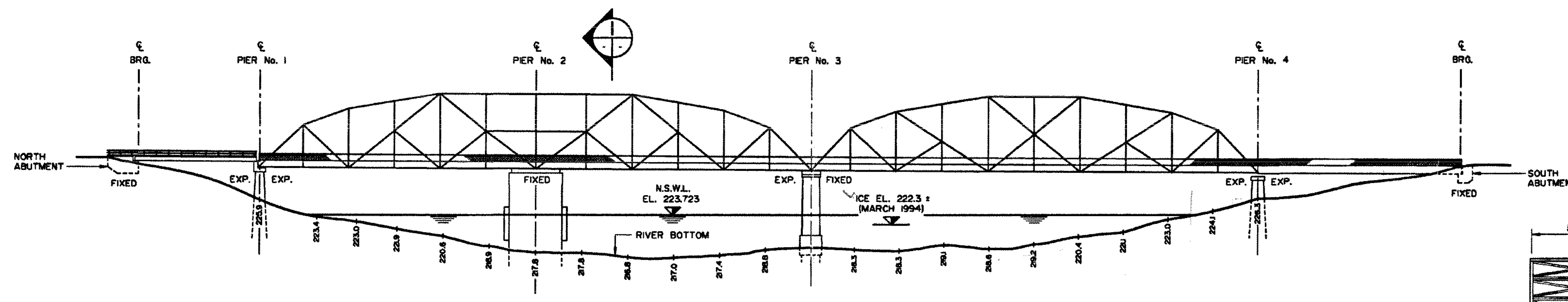
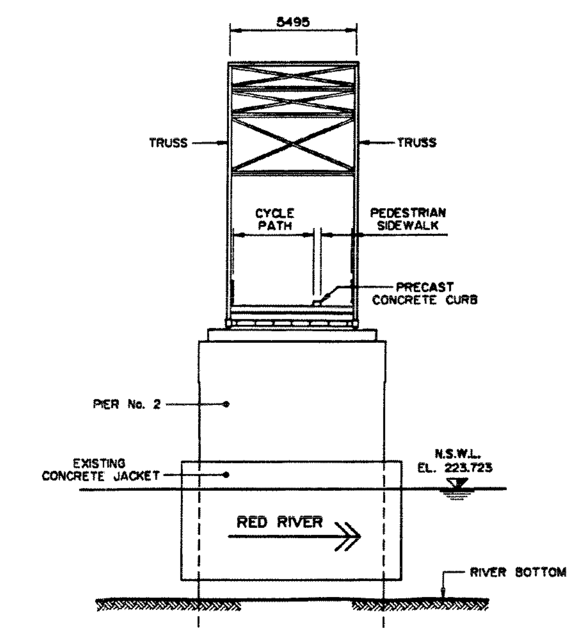
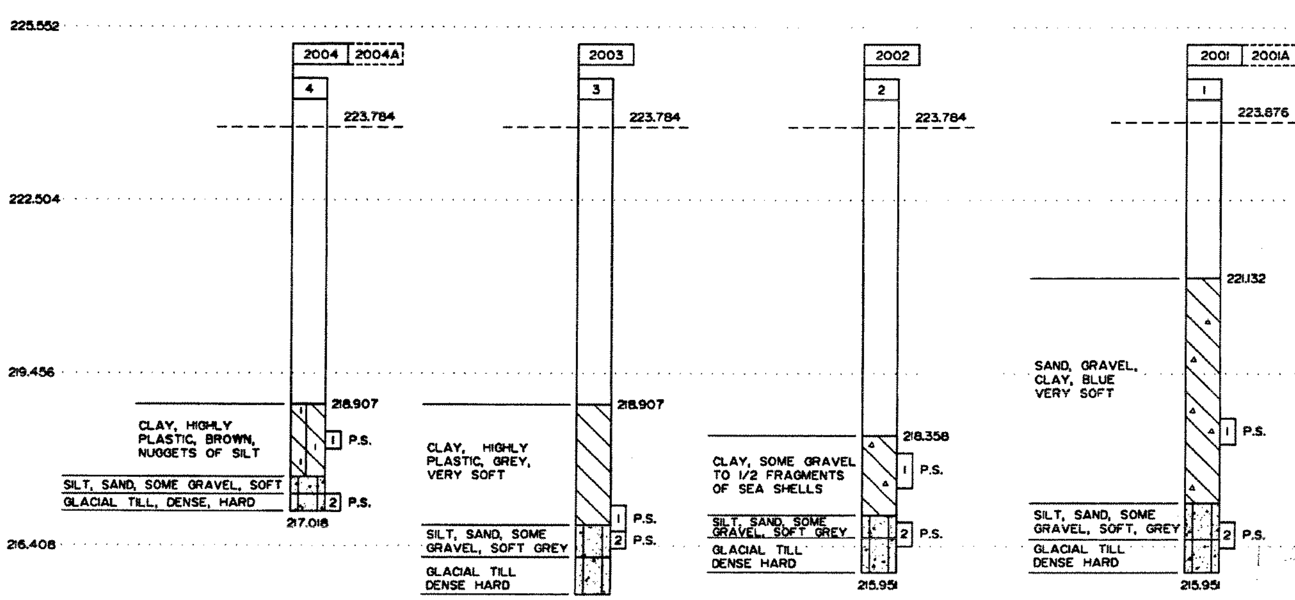


ELEVATION (LOOKING EAST)
SCALE: 1:400



SECTION
SCALE: 1:50

TYPICAL SECTION THROUGH BRIDGE



SUMMARY OF 1964 TEST HOLE DATA
N.T.S.

DESIGN CRITERIA

CONCRETE:
- PIER JACKETS: f'c = 32 MPa
- PIER No. 2 ROCK ANCHOR SEATS: f'c = 35 MPa

REINFORCING STEEL:
- CSA G30.12 GRADE 400
- CLEAR COVER TO REINFORCING STEEL IS 50mm UNLESS NOTED OTHERWISE

RECORD DRAWING
APPROVED BY: DATE 97.12.08

SOIL GROUP SYMBOLS & LEGEND

- THE SYMBOLS SHOWN MAY BE COMBINED TO DENOTE THE VARIOUS SOIL COMBINATIONS, THE PREDOMINENT BEING HEAVIER
- P.S. DENOTES 'PISTON SAMPLER'
- ⊕ DENOTES PENETRATION TEST & SAMPLE HOLE

B.M. ELEV.

NO.	REVISIONS	DATE	BY

DILLON
Consulting Engineers · Planners
Environmental Scientists

DESIGNED BY	M.B.C.	CHECKED BY	M.B.C.
DRAWN BY	M.C.R.	APPROVED BY	
HOR. SCALE	AS SHOWN	AUTHORIZED BY: R. SMITH / BRIDGE MAINTENANCE ENG.	DATE 960918
VERTICAL	AS SHOWN	ACCEPTED BY: S. OBERSPANGER / BRIDGE CONSTRUCTION ENG.	DATE 960918
DATE	AUGUST 1996		

ENGINEER'S SEAL
PROVINCE OF MANITOBA
M. B. CHISLETT
96/09/18
REGISTERED PROFESSIONAL ENGINEER

CONSULTANT DRAWING NO. 96-3295

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION
STREETS AND TRANSPORTATION DEPARTMENT

ELM PARK BRIDGE
RIVER PIER REPAIRS

GENERAL ARRANGEMENT
TEST HOLE DATA & DESIGN CRITERIA

CITY DRAWING NUMBER B131-96-02
SHEET 2 OF 6

B-5986-2

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.

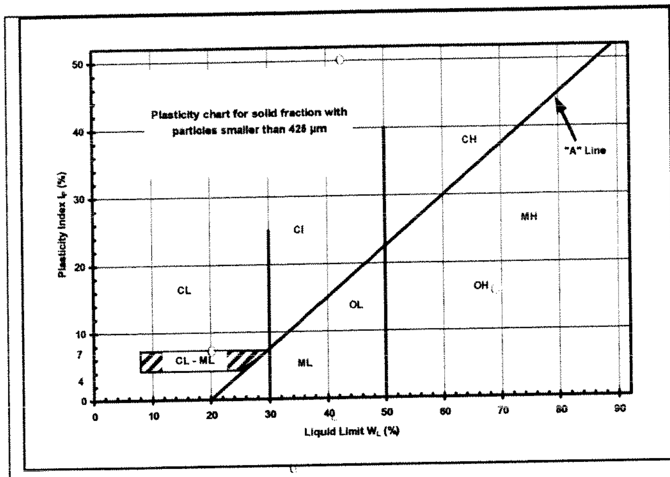
EXPLANATION OF FIELD & LABORATORY TEST DATA

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 $4<W_P<7$
			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	$C_U > 6$ $1 < C_C < 3$		$C_U = \dfrac{D_{60}}{D_{10}}$
			Poorly graded sands, gravelly sands, with little or no fines		SP	0-5	Not satisfying SW requirements		$C_C = \dfrac{(D_{30})^2}{D_{10} x D_{60}}$
		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
	Concrete		Bedrock (Undifferentiated)
	Fill		Bedrock (Limestone)

AECOM

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**
<12	very soft
12 – 25	soft
25 – 50	medium or firm
50 – 100	stiff
100 – 200	very stiff
200	hard

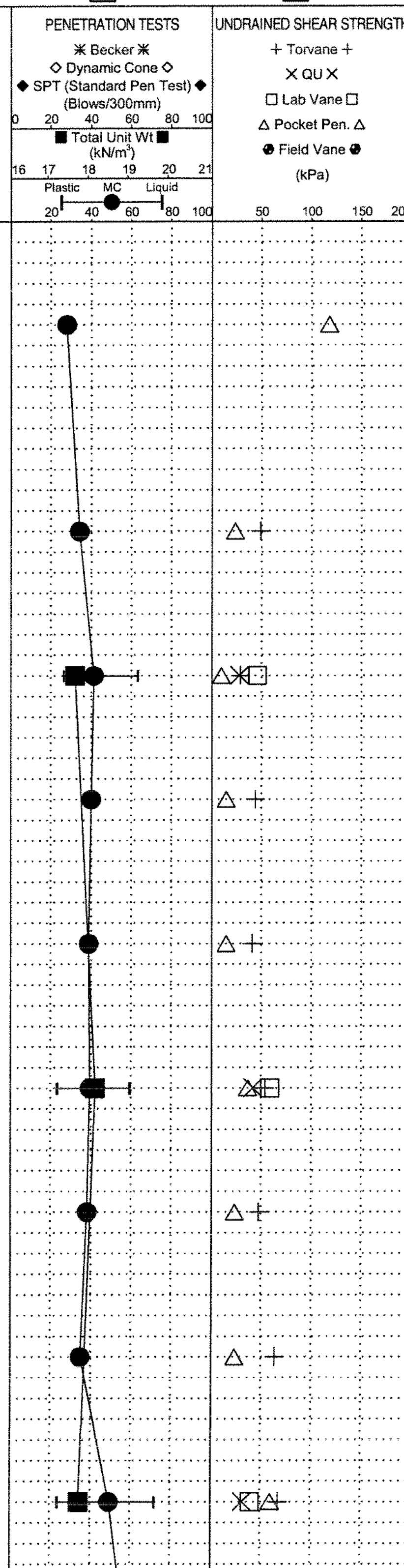
The resistance (N) of a non-cohesive soil can be related to compactness condition as follows

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

PROJECT: Elm Park Bridge	CLIENT: City of Winnipeg	TESTHOLE NO: **TH-09-01**
LOCATION: 1 m North of Pier No. 4, 6 m West of Bridge		PROJECT NO.: 60119229
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125 mm SSA	ELEVATION (m): 227.00

SAMPLE TYPE: GRAB | SHELBY TUBE | SPLIT SPOON | BULK | NO RECOVERY | CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	ELEVATION
0	CLAY (Alluvial) - silty, some sand, trace organics (rootlets) - brown - moist, firm - intermediate plasticity		
		G2	
1			226
2	- sandy, soft to firm below 1.8 m		225
		G3	
3	- high plasticity below 3.0 m - trace organic inclusions (<1 mm dia, black) below 3.6 m	T4	224
4			223
		G5	
5			222
		G6	
6	- some sand, grey, firm below 6.1 m		221
		T7	
7	- trace precipitates (<2 mm dia) below 7.0 m		220
		G8	
8			219
	- some organic inclusions (<5 mm dia.) between 8.4 and 8.7 m	G9	
9	- firm below 9.1 m		218
		T10	
10			



COMMENTS

LOGGED BY: Jared Baldwin	COMPLETION DEPTH: 11.28 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 20/10/09
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

AECOM

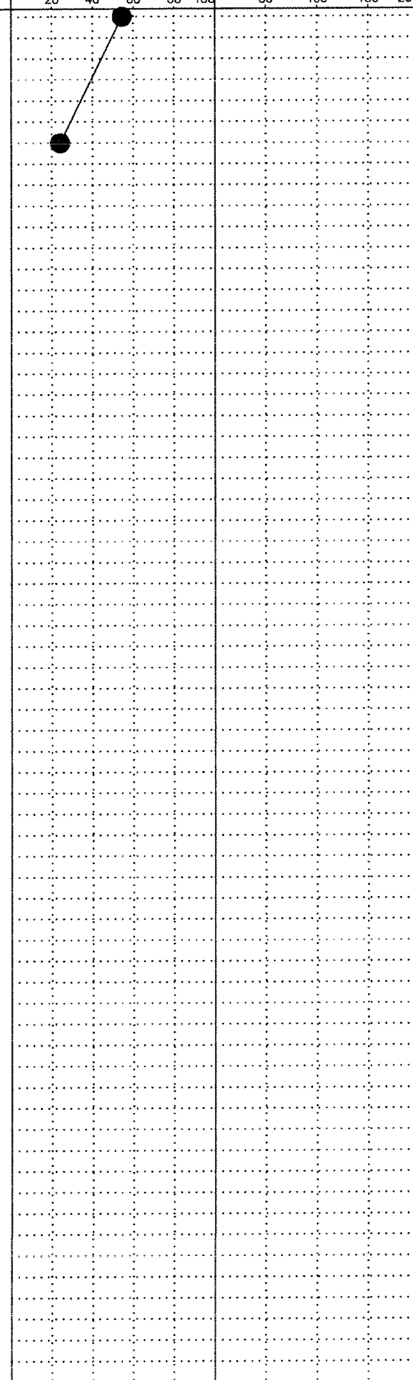
PROJECT: Elm Park Bridge	CLIENT: City of Winnipeg	TESTHOLE NO: **TH-09-01**
LOCATION: 1 m North of Pier No. 4, 6 m West of Bridge		PROJECT NO.: 60119229
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125 mm SSA	ELEVATION (m): 227.00

SAMPLE TYPE: GRAB | SHELBY TUBE | SPLIT SPOON | BULK | NO RECOVERY | CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS	UNDRAINED SHEAR STRENGTH	COMMENTS	ELEVATION
10			Grab	G11					
		SILT (Till) - sandy, trace to some gravel - light brown - moist to wet, firm - low plasticity - <25 mm dia., subrounded / subangular gravel							
11			Grab	G12					216
		SAND (Till) - silty, some clay, some gravel - light brown - wet - fine to coarse grained, well graded - <25 mm dia., subangular / subrounded gravel							
12		END OF TEST HOLE AT 11.3 m IN SAND (TILL) Notes: 1) Power auger refusal at 11.3 m below grade. 2) Sloughing observed below 10.8 m below grade. 3) Seepage observed in SAND (Till). 4) Water level observed at 3.8 m below grade immediately after drilling. 5) Test hole backfilled with auger cuttings. 6) Bentonite plug at 10.8 m below grade and at surface.							215
13									214
14									213
15									212
16									211
17									210
18									209
19									208
20									

PENETRATION TESTS: ✳ Becker ✳ ; ◇ Dynamic Cone ◇ ; ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ; ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 ; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane + ; × QU × ; □ Lab Vane □ ; △ Pocket Pen. △ ; ● Field Vane ● (kPa) 50 100 150 200



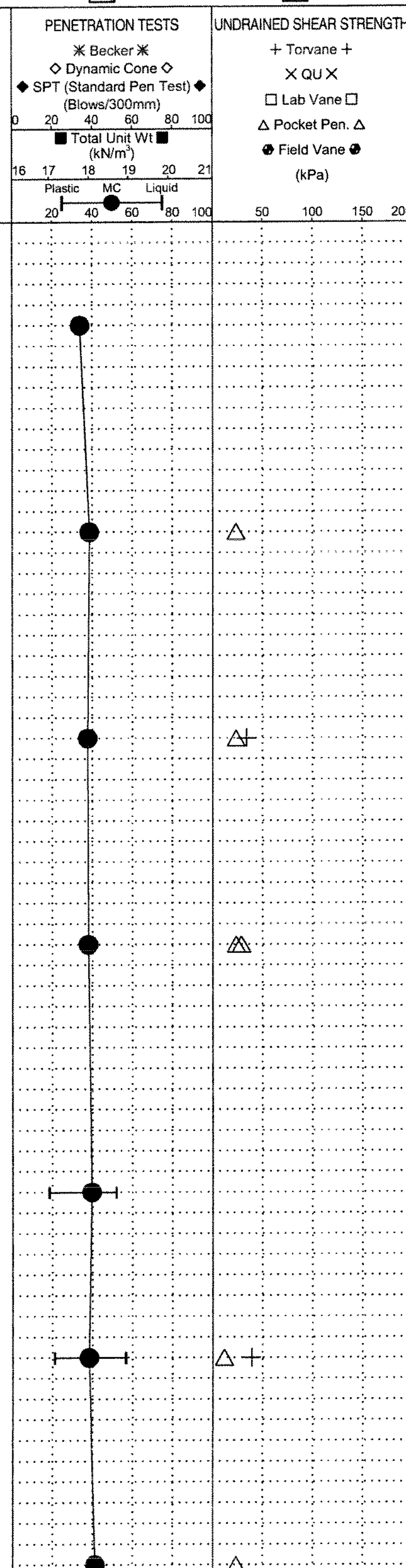
	LOGGED BY: Jared Baldwin	COMPLETION DEPTH: 11.28 m
AECOM	REVIEWED BY: Faris Khalil	COMPLETION DATE: 20/10/09
	PROJECT ENGINEER: Faris Khalil	Page 2 of 2

PROJECT: Elm Park Bridge	CLIENT: City of Winnipeg	TESTHOLE NO: **TH-09-02**
LOCATION: 8.5 m Soth of Pier No. 4, 3 m West of Bridge		PROJECT NO.: 60119229
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125 mm SSA	ELEVATION (m): 227.30

SAMPLE TYPE: GRAB, SHELBY TUBE, SPLIT SPOON, BULK, NO RECOVERY, CORE

BACKFILL TYPE: BENTONITE, GRAVEL, SLOUGH, GROUT, CUTTINGS, SAND

DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	ELEVATION
0	CLAY (Fill) - silty, sandy, trace to some gravel (<10 mm dia.), trace organics, brown and black, moist, firm intermediate plasticity		227
	CLAY - silty, sandy - brown - moist, firm - intermediate plasticity	G13	
1			226
2	- soft to firm below 1.8 m	G14	225
3	- firm below 3.4 m		224
4	- trace organic inlcusions (<2 mm dia., black), high plasticity below 4.0 m	G15	223
	- mottled grey and brown below 4.6 m		
5	- some sand to sandy, grey below 5.5 m	G16	222
6			221
7	- 150 mm sand pocket (fine to medium grained, brownish-grey, wet) between 6.7 and 6.9 m	G17	220
	- some organic inclusions (<5 mm dia., black) between 7.3 and 7.6 m - trace precipitates (<2 mm dia.), soft to firm below 7.6 m		
8		G18	219
9	- moist to wet below 9.4 m		218
10		G19	



COMMENTS

LOGGED BY: Jared Baldwin	COMPLETION DEPTH: 11.58 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 20/10/09
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

AECOM

LOG OF TEST HOLE ELM PARK BRIDGE - TEST HOLE LOGS.GPJ UMA WINN.GDT 7/12/09

PROJECT: Elm Park Bridge	CLIENT: City of Winnipeg	TESTHOLE NO: **TH-09-02**
LOCATION: 8.5 m Soth of Pier No. 4, 3 m West of Bridge		PROJECT NO.: 60119229
CONTRACTOR: Paddock Drilling Ltd.	METHOD: RM30, 125 mm SSA	ELEVATION (m): 227.30

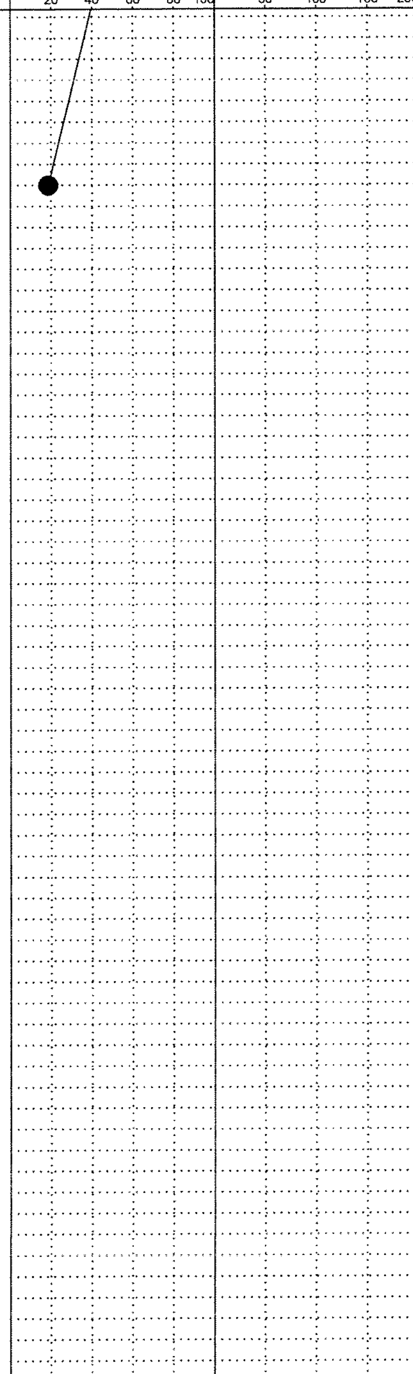
SAMPLE TYPE: GRAB | SHELBY TUBE | SPLIT SPOON | BULK | NO RECOVERY | CORE

BACKFILL TYPE: BENTONITE | GRAVEL | SLOUGH | GROUT | CUTTINGS | SAND

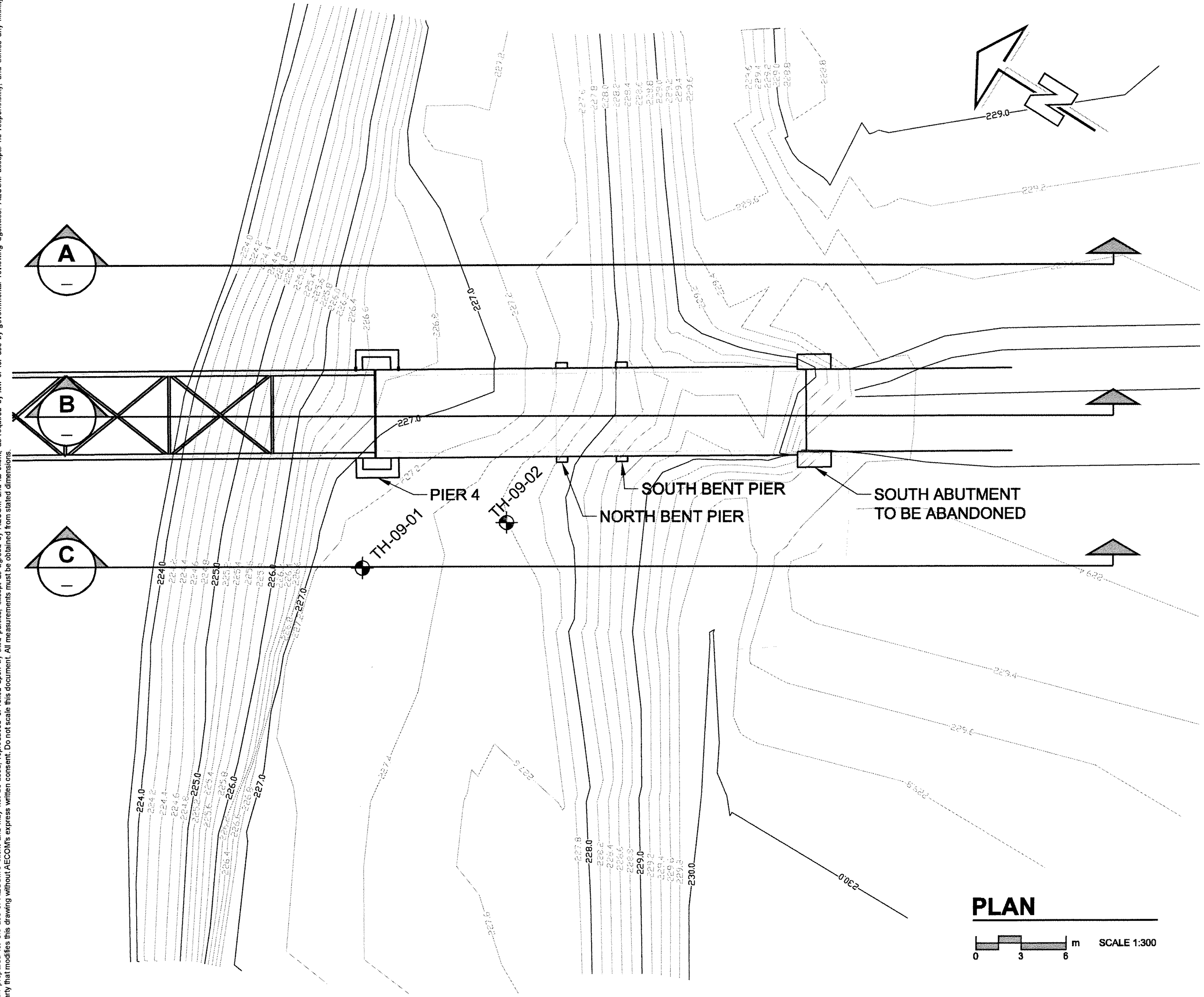
DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
10	- trace gravel (<25 mm dia., subrounded / subangular) below 10.1 m					217
	SILT (Till) - sandy, some gravel to gravelly (<25 mm dia., subrounded / subangular), light brown, wet, firm, low plasticity					
11		GRAB	G20			216
11.6	END OF TEST HOLE AT 11.6 m IN SILT (TILL) Notes: 1) Power auger refusal at 11.6 m below grade. 2) No sloughing observed. 3) Seepage observed in SILT (Till) 4) Water level observed at 4.3 m below grade immediately after drilling. 5) Insalled standpipe (SP-09-02) with casagrande tip to 11.3 m below grade with flush mount protective casing. 6) Water level in SP-09-02 observed at 3.5 m below top of pipe on December 1, 2009.					
12						215
13						214
14						213
15						212
16						211
17						210
18						209
19						208
20						

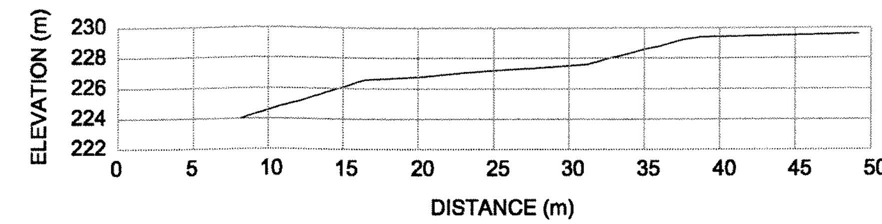
PENETRATION TESTS: Becker; Dynamic Cone; SPT (Standard Pen Test) (Blows/300mm) 0 20 40 60 80 100; Total Unit Wt (kN/m³) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane +; × QU ×; □ Lab Vane □; △ Pocket Pen. △; Field Vane (kPa) 50 100 150 200



	LOGGED BY: Jared Baldwin	COMPLETION DEPTH: 11.58 m
AECOM	REVIEWED BY: Faris Khalil	COMPLETION DATE: 20/10/09
	PROJECT ENGINEER: Faris Khalil	Page 2 of 2

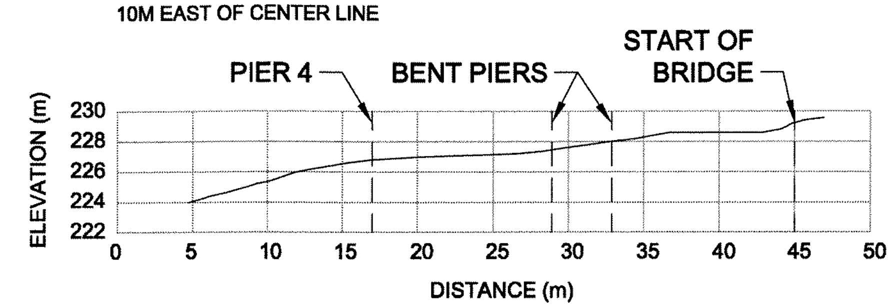




SECTION A

m SCALE 1:500
0 5 10

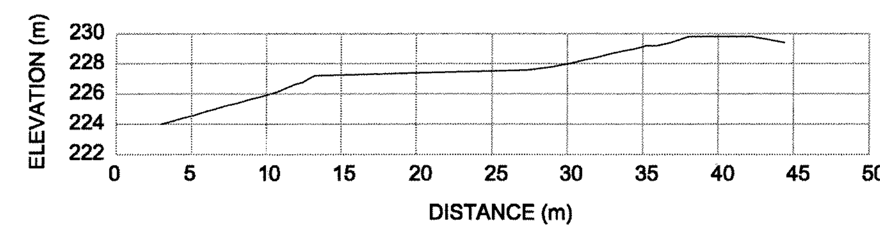
10M EAST OF CENTER LINE



SECTION B

m SCALE 1:500
0 5 10

CENTER LINE OF BRIDGE



SECTION C

m SCALE 1:500
0 5 10

10M WEST OF CENTER LINE

AECOM

AECOM Canada Ltd.
Elm Park Pedestrian Bridge
Substructure Repair Works

Topography, Cross Sections & Test Hole Locations

Figure - 01

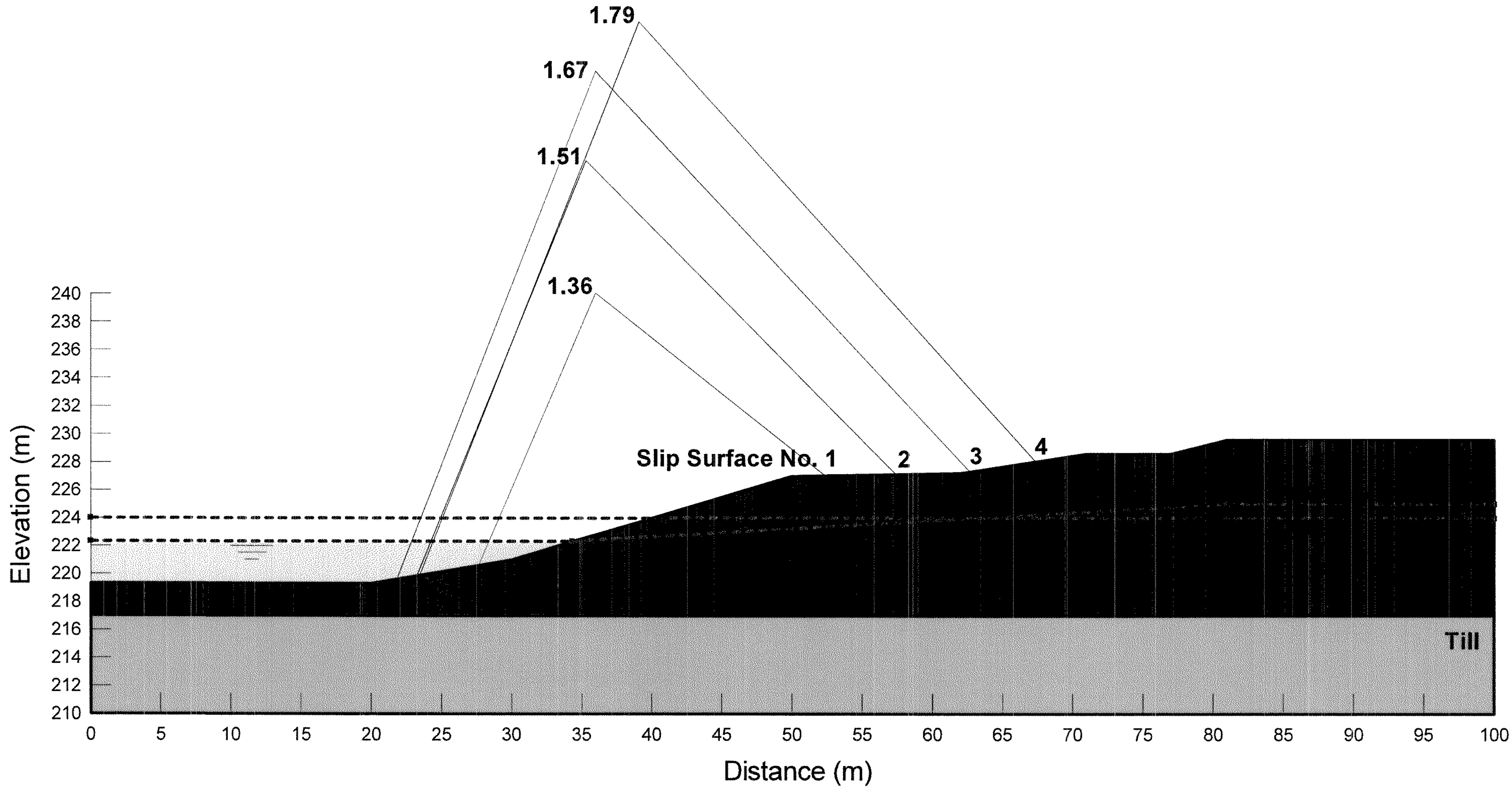


Figure 02: Stability Results For The Existing Geometry

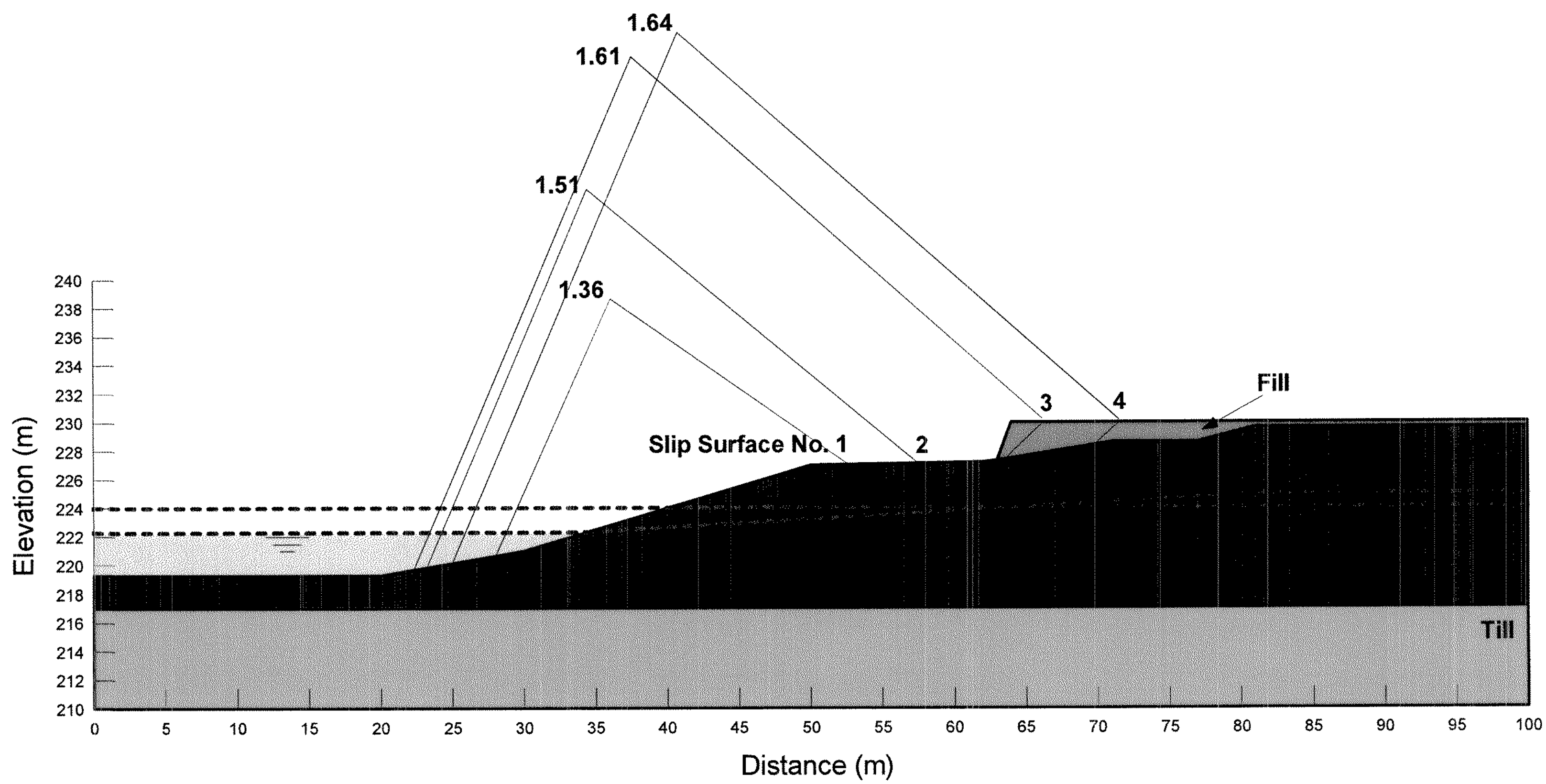


Figure 03: Stability Results For The Proposed Geometry