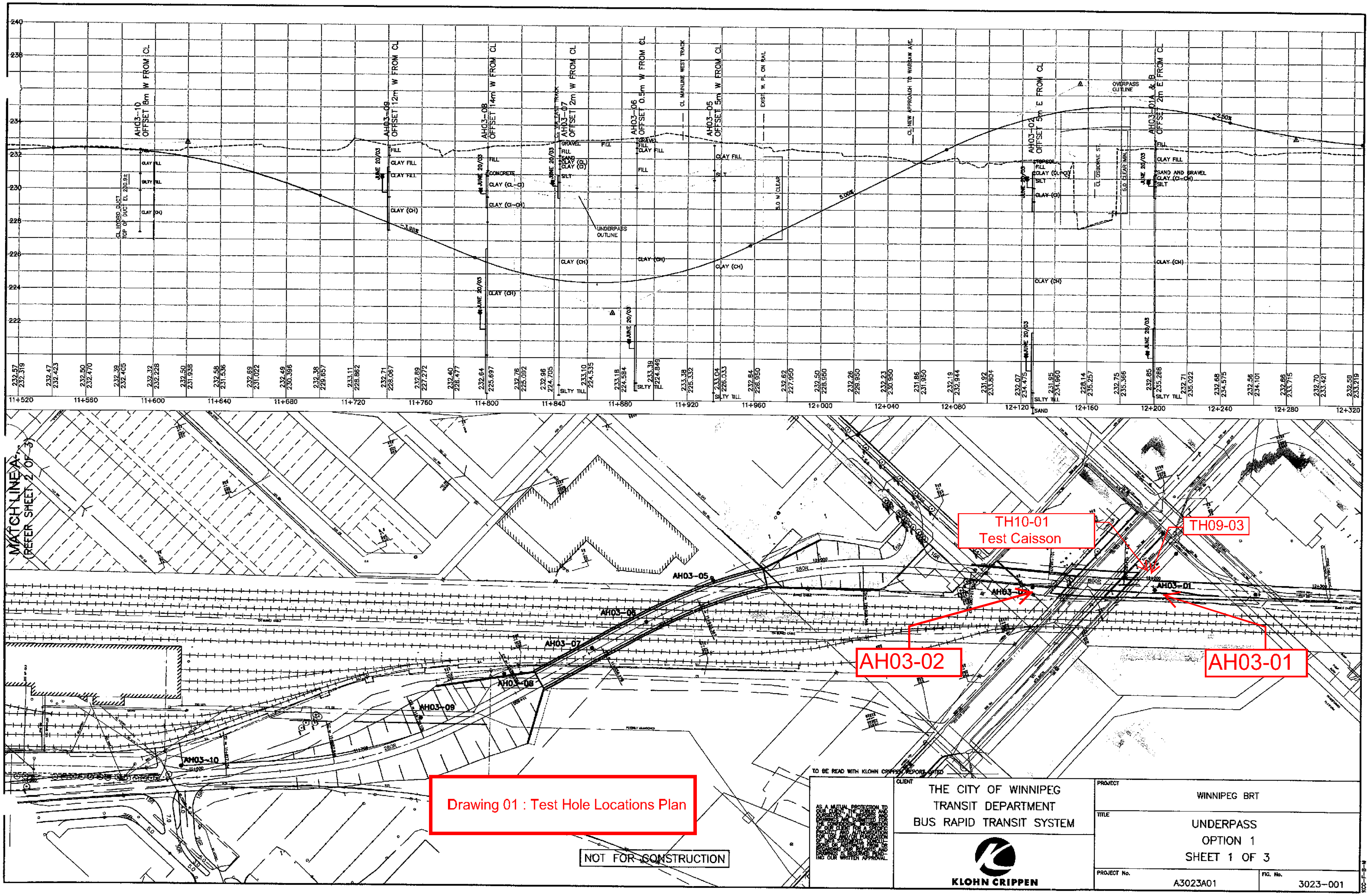
APPENDIX B

AECOM TEST HOLE LOGS AND GEOTECHNICAL REPORT

APRIL 2010



Drawing 01 : Test Hole Locations Plan

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

CLIENT

THE CITY OF WINNIPEG
TRANSIT DEPARTMENT
BUS RAPID TRANSIT SYSTEM

KLOHN CRIPPEN

PROJECT

WINNIPEG BRT

TITLE

UNDERPASS
OPTION 1
SHEET 1 OF 3

PROJECT No.
A3023A01

FIG. No.
3023-001

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.

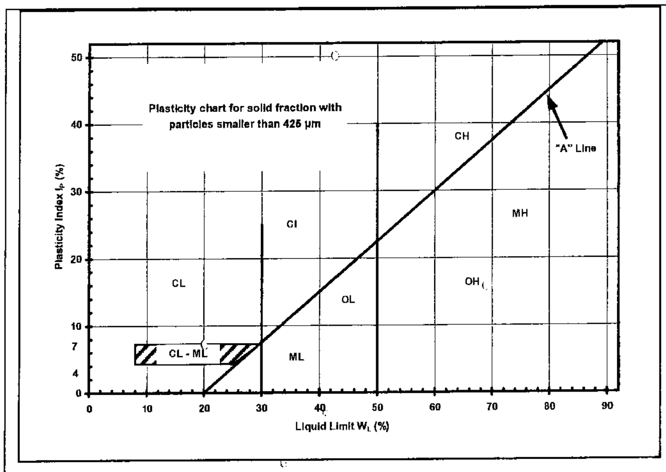
In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced 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 $4<W_P<7$ $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10}xD_{60}}$
			Poorly graded gravels, sandy gravels, with little or no fines	◀	GP	0-5	Not satisfying GW requirements		
		DIRTY GRAVELS (With some fines)	Silty gravels, silty sandy gravels	◀	GM	> 12		Atterberg limits below "A" line or $W_P<4$	
			Clayey gravels, clayey sandy gravels	▨	GC	> 12		Atterberg limits above "A" line or $W_P<7$	
	SANDS (More than 50% of coarse fraction of sand size)	CLEAN SANDS (Little or no fines)	Well graded sands, gravelly sands, with little or no fines	○	SW	0-5	$C_U > 6$ $1 < C_C < 3$		
			Poorly graded sands, gravelly sands, with little or no fines	○	SP	0-5	Not satisfying SW requirements		
		DIRTY SANDS (With some fines)	Silty sands, sand-silt mixtures	○	SM	> 12		Atterberg limits below "A" line or $W_P<4$	
			Clayey sands, sand-clay mixtures	▨	SC	> 12		Atterberg limits above "A" line or $W_P<7$	
FINE GRAINED SOILS	SILTS (Below 'A' line negligible organic content)	$W_L<50$	Inorganic silts, silty or clayey fine sands, with slight plasticity	▥	ML			Classification is Based upon Plasticity Chart	
		$W_L>50$	Inorganic silts of high plasticity	▥	MH				
	CLAYS (Above 'A' line negligible organic content)	$W_L<30$	Inorganic clays, silty clays, sandy clays of low plasticity, lean clays	▨	CL				
		$30<W_L<50$	Inorganic clays and silty clays of medium plasticity	▨	CI				
		$W_L>50$	Inorganic clays of high plasticity, fat clays	▨	CH				
	ORGANIC SILTS & CLAYS (Below 'A' line)	$W_L<50$	Organic silts and organic silty clays of low plasticity	▥	OL				
		$W_L>50$	Organic clays of high plasticity	▨	OH				
HIGHLY ORGAINIC SOILS			Peat and other highly organic soils	〰	Pt	Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

■	Asphalt	○	Till			AECOM
⬚	Concrete	▩	Bedrock (Undifferentiated)			
▩	Fill	▦	Bedrock (Limestone)			

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



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

* for example: gravelly, sandy clayey, silty

Definition of Oversize Material

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

LEGEND OF SYMBOLS

Laboratory and field tests are identified as follows:

q_u - undrained shear strength (kPa) derived from unconfined compression testing.

T_v - undrained shear strength (kPa) measured using a torvane

pp - undrained shear strength (kPa) measured using a pocket penetrometer.

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

F_v - undrained shear strength (kPa) measured using a field vane.

γ - bulk unit weight (kN/m^3).

SPT - Standard Penetration Test. Recorded as number of blows (N) from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 51 mm O.D. Raymond type sampler 0.30 m into the soil.

DPPT - Drive Point Pentrometer Test. Recorded as number of blows from a 63.5 kg hammer dropped 0.76 m (free fall) which is required to drive a 50 mm drive point 0.30 m into the soil.

w - moisture content (W_L, W_P)

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

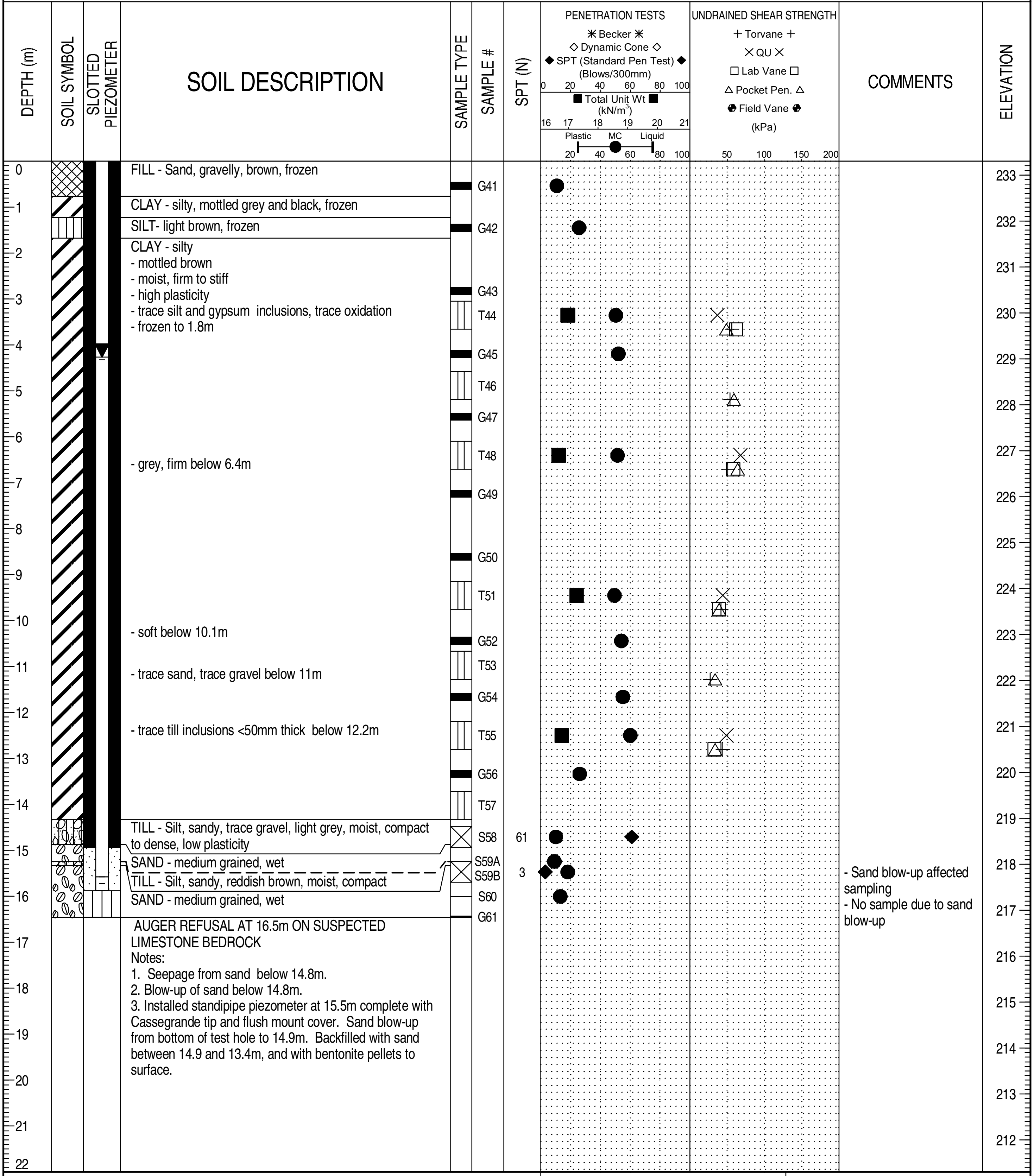
Su (kPa)	**CONSISTENCY**
<12	very soft
12 – 25	soft
25 – 50	medium or firm
50 – 100	stiff
100 – 200	very stiff
200	hard

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: Southwest Rapid Transit Corridor	CLIENT: COW/Dillon Consulting	TESTHOLE NO: **TH-09-03**
LOCATION: At East abutment of proposed bridge N 5526240 E 633577		PROJECT NO.: 60115506
CONTRACTOR: Paddock Drilling Ltd.	METHOD: 125mm Solid Stem Augers	ELEVATION (m): 233.30

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



FILL - Sand, gravelly, brown, frozen

CLAY - silty, mottled grey and black, frozen

SILT- light brown, frozen

CLAY - silty
- mottled brown
- moist, firm to stiff
- high plasticity
- trace silt and gypsum inclusions, trace oxidation
- frozen to 1.8m

- grey, firm below 6.4m

- soft below 10.1m

- trace sand, trace gravel below 11m

- trace till inclusions <50mm thick below 12.2m

TILL - Silt, sandy, trace gravel, light grey, moist, compact to dense, low plasticity

SAND - medium grained, wet

TILL - Silt, sandy, reddish brown, moist, compact

SAND - medium grained, wet

AUGER REFUSAL AT 16.5m ON SUSPECTED LIMESTONE BEDROCK

Notes:
1. Seepage from sand below 14.8m.
2. Blow-up of sand below 14.8m.
3. Installed standpipe piezometer at 15.5m complete with Cassegrande tip and flush mount cover. Sand blow-up from bottom of test hole to 14.9m. Backfilled with sand between 14.9 and 13.4m, and with bentonite pellets to surface.

- Sand blow-up affected sampling
- No sample due to sand blow-up

LOGGED BY: KT	COMPLETION DEPTH: 16.46 m
REVIEWED BY: FK	COMPLETION DATE: 26/3/09
PROJECT ENGINEER: Faris Khalil	Page 1 of 1

PROJECT: Southwest Rapid Transit Corridor	CLIENT: COW/Dillon Consulting	TESTHOLE NO: **TH-10-01**
LOCATION: Near the proposed east pier N5526241 E633579 (Test Caisson)		PROJECT NO.: 60115506
CONTRACTOR: Subterranean Ltd	METHOD: Large Diameter Auger/Coring Bucket	ELEVATION (m): 233.30

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ⧄ NO RECOVERY ◧ CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	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	COMMENTS	ELEVATION
0		FILL- SAND, gravelly, brown, frozen							233
1		CLAY-silty - grey, moist, firm -high plasticity							232
2									231
3									230
4									229
5									228
6									227
7									226
8									225
9									224
10									223
11									222
12									221
13									220
14									219
15		SILT (till) -wet, soft							218
16									217
17		-sand and gravel zone -water bearing BEDROCK-limestone (boulder and cobble size) -highly weathered broken pieces							216
18									215
19									214
20		CLAY-silty -grey, wet, soft -high plasticity							213
21		BEDROCK-limestone -massive, fine-grained -vertical and horizontal joints up to 40 mm thick							212
22									

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LOGGED BY: Faris Khalil	COMPLETION DEPTH: 27.74 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 3/4/10
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

PROJECT: Southwest Rapid Transit Corridor	CLIENT: COW/Dillon Consulting	TESTHOLE NO: **TH-10-01**
LOCATION: Near the proposed east pier N5526241 E633579 (Test Caisson)		PROJECT NO.: 60115506
CONTRACTOR: Subterranean Ltd	METHOD: Large Diameter Auger/Coring Bucket	ELEVATION (m): 233.30

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DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	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	COMMENTS	ELEVATION
22	▦	-infill with silt and clay (very stiff to hard)							211
23	▦								210
24	▦								209
25	▦								208
26	▦								207
27	▦								206
28		END HOLE AT 27.7m IN LIMESTONE BEDROCK Notes: 1. Seepage within sand and gravel zone at ~16.5m; 2. Hole backfilled with G 30 conc. to elv. 220 m, with stabilized fill to ground surface.							205
29									204
30									203
31									202
32									201
33									200
34									199
35									198
36									197
37									196
38									195
39									194
40									193
41									192
42									191
43									190
44									

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LOGGED BY: Faris Khalil	COMPLETION DEPTH: 27.74 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 3/4/10
PROJECT ENGINEER: Faris Khalil	

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Memorandum

APR 14 2010

Proj. No.: ______ Dist. ☐ ☐
Task No.: ______ ☐ ☐
File Type: ______ ☐ ☐

To	Dave Krahn	Page	1
CC			
Subject	City of Winnipeg – Bus Rapid Transit Project Osborne Overpass – Geotechnical Recommendations		
From	Faris Khalil		
Date	April 9, 2010	Project Number	60115506 (F504 444 01 – 4.6.1)

1. Introduction

The City of Winnipeg Transit Department is planning the construction of a new overpass structure over Osborne Street as part of the Bus Rapid Transit (BRT) project. Dillon Consulting, the project consultant, retained AECOM to provide geotechnical engineering services for the new structure. Klohn Crippen conducted geotechnical investigation at the site in 2003 and prepared a geotechnical report in 2004 documenting the investigation and providing design and construction recommendations. AECOM performed additional geotechnical investigations in 2009 and 2010 as part of the detailed design phase of the project.

This memorandum summarizes the geotechnical investigations undertaken at the site and provides updated geotechnical recommendations.

2. Available Information

The geotechnical investigation completed in 2003 by Klohn Crippen included drilling of thirteen test holes to determine the soil and groundwater conditions along the proposed alignment of the BRT. Two test holes (AH03-01 and AH03-02) were drilled at the site of the proposed overpass, one at each abutment at the location shown on Drawing 01. Disturbed and undisturbed samples were collected and laboratory testing included moisture content, consolidation, and unconfined compression. A standpipe piezometer was installed in each test hole to measure groundwater levels. The test hole logs are attached in Appendix A.

3. Geotechnical Investigation

AECOM drilled one test hole (TH 09-03) on March 26, 2009 to supplement the existing geotechnical information. The test hole was located in the vicinity of the proposed north abutment as shown on Drawing 01. Drilling was carried out by Paddock Drilling Ltd. using a drill rig equipped with 125 mm solid stem augers. The test hole was advanced to auger refusal at 16.5 m below existing ground surface. Disturbed and relatively undisturbed soil samples were collected at regular intervals. All soils observed during drilling were logged and visually classified on site by AECOM personnel. A standpipe piezometer was installed in the till to measure groundwater levels.

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Memorandum

APR 14 2010
Proj. No.: ☐ ☐
Task No.: ☐ ☐
File Type: ☐ ☐

To Dave Krahn

Page 1

CC

Subject City of Winnipeg – Bus Rapid Transit Project
Osborne Overpass – Geotechnical Recommendations

From Faris Khalil

Date April 9, 2010

Project Number 60115506
(F504 444 01 – 4.6.1)

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Soil samples recovered during drilling were transported to AECOM's Material Testing Laboratory in Winnipeg for further visual examination and testing. Laboratory testing consisted of determination of moisture content, unit weight, Atterberg limits and undrained shear strength. Test hole log is attached in Appendix A.

Between January 13 and 18, 2010, AECOM drilled one large diameter test caisson (TH 10-01) to evaluate bedrock conditions at site. The test caisson was located in the vicinity of the proposed north pier as shown on Drawing 01. Drilling was carried out by Subterranean (Manitoba) Ltd. using a track mounted Soilmec SR 30 piling rig equipped with a 1050 mm diameter flight auger and 910 mm core barrel. The test caisson was advanced through the overburden with the flight auger to practical refusal near the bedrock surface at elevation 216.5 m or 16.8 m below existing ground. Two temporary steel casings 1,060 and 960 mm diameter were used to sleeve the borehole and the test caisson was advanced into the bedrock using the core barrel. The rock coring was advanced 11 m below the rock/till interface or 27.8 m below ground surface. Large diameter rock cores were recovered for visual inspection at site by AECOM personnel, Figure 01.

Downhole inspection of the test caisson was not possible because of the high levels of gas detected inside the test caisson. Video camera inspection was completed. The test hole was backfilled with 30 MPa concrete to a level about 1 m above bedrock surface, the remainder of the test caisson was backfilled with stabilized fill. A log of the test caisson is attached in Appendix A.

Figure 01: Large diameter rock cores recovered from test caisson

3.1. Soil Profile

In descending order the soil profile consists of:

Fill

Fill about 0.5 m thick was encountered at ground surface in all test holes. The fill consists of gravely sand and contains trace organics.

Glacio-lacustrine Clay

Galciolacustrine silty clay about 14 m thick was encountered beneath the fill in all test holes. Generally, the clay is brown changing to grey with depth, firm to stiff becoming soft to firm with increasing depth, moist and of high plasticity. Below 10 m, trace sand and trace gravel inclusions were observed. Trace till inclusions were observed at depths close to the clay/ till contact.

A wet silt layer about 0.3 m thick was encountered in all test holes about 1.5 and 2.5 m below existing ground.

Moisture contents range from 40 to 65 percent. The average bulk unit weight of the clay is 17 kN/m^3. Undrained shear strength measured from unconfined compression tests range from 36 to 50kPa.

Glacial Till

The clay is underlain by glacial silt till that typically contains variable amounts of clay, sand and gravel. A wet and loose sand seam encountered within or at the base of the till. Blow-up of this layer into the test hole was encountered. The test holes were advanced 1.3 to 2.3 m into till and terminated at auger refusal. The till is light grey/brown, soft to compact, moist to wet, and non plastic to low plasticity. It should be recognized that although not encountered in the test holes, boulders and cobbles are not uncommon within the till unit in the Winnipeg area.

Limestone bedrock

The till is underlain by Paleozoic Carbonate (limestone) bedrock, which forms the Upper Carbonate Aquifer. The depth to the bedrock surface is about 16.8 m below ground surface or approximately at elevation 216.5 m. The top 2.9 m of the bedrock is highly weathered and at the test caisson location is underlain by a 1.2 m thick clay layer. The clay deposit is likely associated with an infilled (wide) crack or sink hole in the bedrock, occurrences which are not uncommon in local limestone formations.

Sound bedrock was encountered at about 20.8 m below existing ground or at elevation 212.5 m.

Further discussion is provided in Section 5.2.

3.2. Groundwater Conditions

Sloughing, seepage and blow up was observed in the till and the underlying sand in TH09-03. Standpipe piezometers were installed in the silt layer and in the till to measured groundwater level (GWL). The measurements are summarized in Table 01. It should be noted that these levels could vary seasonally or as a result of construction activities.

Table 01: Groundwater Measurements

Date	TH03-01A (Till)	TH03-01B (Silt)	TH03-02A (Till)	TH03-02B (Silt)	TH09-03 (Till)
End of Drilling	223.9	230.9	219.76	231.4	228.4
17-Jun-2009	220.9	230.6	NA	NA	NA
20-Jun-2009	220.5	230.7	220.1	230.4	NA
01-April- 2009	NA	NA	NA	NA	229.0
06-April-2010	225.8	230.4	224.5	229.2	Not accessible

4. Stability and Settlement

It is our understanding that the approach fills at each side of the structure will be of limited heights and not to exceed 2.0 m. Settlement of the foundation soil is not expected to be of concern under these fill heights. Side slopes constructed at no steeper than 4H:1V are expected to perform satisfactorily.

5. Foundations

Steel H piles and rock-socketted caissons were selected by Dillon as the preferred foundation systems to support the proposed structure and cast-in-place concrete friction piles were selected to support a short retaining wall. This report discusses only these foundations types.

5.1. Driven Steel H Piles

Driven steel H piles are considered suitable to support the proposed bridge, and are recommended at the bridge abutments as pile splicing can be undertaken economically where the depth to refusal is large. Steel H piles can be designed on the basis of the structural capacity of the pile section provided the piles are driven to practical refusal. The structural capacity of the pile can be determined from the steel sectional area and the maximum allowable stresses of 0.3fy. Practical refusal can be defined as 15 blows/25 mm penetration using a well maintained hammer with rated energy of not less than 50 kJ.

The actual refusal criteria and load capacity for specific steel section and pile driving system should be established based on Pile Driving Analyzer (PDA) testing so that the geotechnical and structural capacity can be confirmed and to protect against pile damage during installation.

Steel piles driven to practical refusal will develop the majority of their capacity from toe resistance, and therefore, no reduction in pile capacity is necessary for reasons related to group action. The design capacity of a pile group can be taken as the number of piles in the group multiplied by the allowable capacity per pile.

The following additional recommendations regarding steel piles are provided.

1. The allowable capacities noted pertain to geotechnical resistance only. The pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation.
2. All piles driven within 5 pile diameters of one another should be monitored for heave and where observed, the piles should be re-driven to the specified refusal criteria.
3. The driving of all piles should be documented by competent and experienced geotechnical personnel.
4. Piles should be fitted with an appropriate toe or shoe to help protect the pile tip during installation.
5. Pile spacing should be a minimum of 3 pile diameters measured centre to centre.
6. The driving of all piles should be documented by experienced geotechnical personnel to confirm and record acceptable piling installation.

5.2. Cast-in-Place Rock-Socketted Caissons

Rock-socketted caissons are considered to be viable foundation system to support the proposed structure. Local practice is to design the caissons based on values of allowable end bearing and shaft adhesion of 3.0 and 1.0 MPa, respectively, provided that down hole inspection and assessment of the rock competency are undertaken. The assessment of the rock competency consists of small diameter proof drilling to 2 m below the socket to detect the presence of voids or clay layers of any significance and determine if deeper socket boring is required. High levels of gas inside the test caisson however, prevented down-hole inspections. It is very possible that these conditions will be encountered for production piles and in this regard, caissons founded in sound bedrock should be designed on the basis of a reduced allowable shaft adhesion of 0.69 MPa with no contribution from end bearing.

Inspection of the recovered rock cores by qualified and experienced geotechnical personnel and down hole video inspection will be required to aid in assessing the competency of the bedrock and determining if longer socket lengths are required. The depth to sound bedrock should be expected to vary across the site and it should be recognized that the presence of the heavily fractured rock and infill material above the socket length may require that a permanent steel casing be left in the ground so that the integrity of the shaft is maintained. In this regard, the basis for measurement and payment for the rock-socket installation should be established in the contract preparation stage to recognize that the bedrock conditions at some rock-socket locations may require unanticipated extra effort and materials for their completion.

The socket length should be a minimum of one socket diameter within sound competent bedrock. The minimum shaft diameter of the rock-socket should not be less than 710 mm and the maximum diameter should be selected to suit the locally available coring equipments. The rock sockets should not be spaced closer than 2.5 socket diameters, centre to centre. Based on the seepage rate encountered during the test caisson investigation, tremie placement of concrete is likely to be required.

5.3. Cast-in-Place Friction Piles

Cast-in-place concrete friction piles can be used to support lightly loaded structures such as retaining walls using an allowable unit skin friction of 12 kPa and no contribution from end bearing. The frictional resistance for the top 2 m along the pile shaft should be excluded from the design calculations. The piles should not extend into the soft clay above the till layer to protect against seepage and collapse of the open hole. In this regard, friction piles should not extend deeper than elevation 223.0.

Additional design and construction recommendations are provided below:

1. Pile diameter should not be less than 0.4 m,
2. Piles should be adequately reinforced to resist possible tension from clay swelling or frost heave,
3. Pile spacing should be a minimum of 3 pile diameters measured centre to centre,
4. Temporary casing to facilitate cleaning, inspection and prevent seepage and sloughing during construction should be available on site,
5. All piles must be taken to completion once they have been initiated,

6. Lateral Earth Pressure

It is understood that the design incorporates three types of retaining structures:

- Abutments and retaining walls – the abutments are designed to retain backfill soil at the bridge approaches, the walls are designed to retain backfill soils underneath the last south span.
- Permanent sheet pile wall – the wall is designed to retain and support the existing railway grade north of Osborne Street,
- Contiguous bored pile (secant pile) wall – the wall is designed to retain and support the existing Hydro cable ducts near the proposed south abutment.

<u>Abutments and Retaining Walls</u>

The lateral earth pressures transferred to the bridge abutments or to the retaining wall will be a function of the backfill material, the method of placing and compacting the backfill, and the amount of horizontal deflection allowed by the abutment or the wall after the backfill is placed. It is recommended that the abutments and the walls be backfilled with a free draining granular material containing a maximum of 5 percent fines (maximum of 5 percent finer than #200 sieve). Cohesive soils are not recommended for backfill behind retaining structures. For free draining coarse granular soils, an active earth pressure coefficient (Ka) of 0.30 should be used to calculate lateral loads on retaining structures which are allowed to translate or deflect horizontally by at least 0.2 percent of the retained. For retaining structures, which are not free to translate, an at-rest earth pressure coefficient (K_o) of 0.5 should be used. Compaction of granular fill within about 1.5 m of the abutment should be conducted with a light hand operated vibrating plate compactor. Over-compaction of the backfill may result in earth pressures that are considerably higher than those predicted in design. Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

Permanent Sheet Pile Wall

The wall should be designed to resist the earth pressure shown on Figure 02. The passive resistance in front of the wall can be taken as shown on Figure 03. Passive resistance should only be accounted for from soils 0.5 m below the final grade in front of the wall. The wall should be designed to resist lateral pressure from live load surcharges including railway loading as shown on Figure 04. Lateral pressures from railway loading should be determined as per CN Guidelines and AREMA using Cooper E90 loading. The lateral pressure distributions should be extended to the base of the wall system (i.e., the bottom of the piles). The wall must be embedded deeply enough to provide kick out resistance for the portion of the wall below the excavation. A minimum factor of safety of 1.5 should be applied to the available passive resistance.

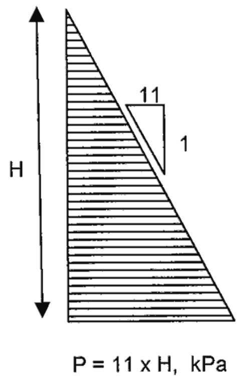


Figure 02: Earth pressure behind sheet pile and secant pile walls

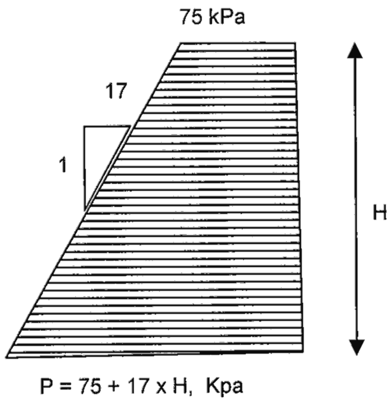


Figure 03: passive earth pressure in front of for sheet pile wall

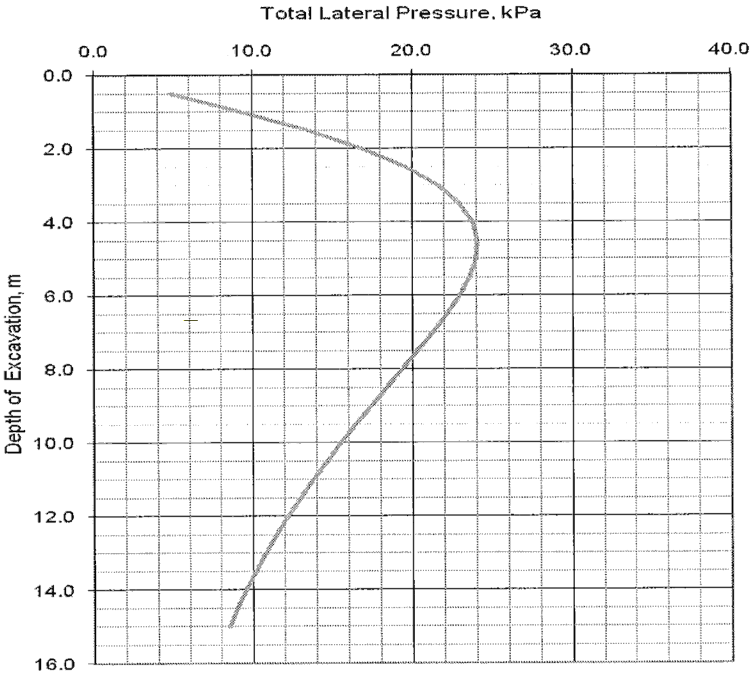


Figure 04: Lateral Pressure from Cooper E90 Railway Loading
(2 tracks, CL at 7.0 and 11.5 m setback from wall face)

Secant Pile Wall

The wall should be designed to resist the earth pressure shown on Figure 02 and any lateral loads from other applicable surcharges including railway and construction loading. Passive resistance should only be accounted for from soils 0.5 m below the final grade in front of the wall. In this regard, the coefficient of horizontal subgrade reaction (k) can be assumed to increase linearly with increasing depth using the following equation (NAVFAC, 1982):

$k = f\ (z/D)$

Where:

k = coefficient of subgrade reaction, $kN/m^2/m$
f = coefficient of variation of subgrade reaction = 350, $kN/m^2/m$
z = depth, m
D = pile diameter, m

To prevent the build-up of hydrostatic pressures behind the retaining structures, a sub-drainage system consisting of filter-wrapped drainage pipe placed in washed gravel should be used at the base of the abutments and wall backfill. A provision for drainage (e.g., weep holes) should be provided to protect against the development of hydrostatic water pressure behind sheet pile and secant pile walls.

7. Deformation Analysis

Deformation analysis was completed to estimate the lateral displacement of the permanent sheet pile wall and the corresponding vertical displacement of the ground under the CN tracks. The analysis was performed using PLAXIS finite element software. The model is based on the design section (i.e., sheet pile section PZ 27), Cooper E90 railway loading and staged excavations. The railway loading was modeled as a 96 kPa uniform pressure acting over two 2.8 m wide strips parallel to the wall alignment at a distance of 7 and 11 m. The following conditions and construction sequence were assumed in the model:

1. In situ condition,
2. Drive sheet pile wall,
3. Apply railway loading Cooper E90 , 2 tracks at 7.0 and 11.0 m setback from the wall face to centreline,
4. Excavate to 1 m below existing grade,
5. Excavate to 2 m below existing grade,
6. Excavate to 3 m (design grade) below existing grade.

The analysis inputs and results including the geometry, sheet pile section properties, soil parameters and construction sequencing are provided in tabular and graphical format in Appendix B. The results indicate that ground subsidence on the retained soil (track) side of the wall in the order of 10 mm can be expected for a 3 m deep excavation in front of the wall. The maximum lateral displacement of the wall is estimated to be 21 mm. It should be recognized that the estimated settlement and lateral displacement are considered to be conservative estimates since the analysis assumes the railway live loads are permanently applied and there is no contribution from the concrete capping beam and the concrete skin wall in front of the wall. However the estimated value of 10 mm is not expected to significantly impact the operation of the tracks.

8. Temporary Excavation and Shoring

All excavations must comply with the Manitoba Workplace Safety and Health Regulations. Temporary excavations in the order of 3 m deep will be required to facilitate the construction of the proposed structure. In some cases (e.g. east side of Osborne Street), the permanent retaining walls (discussed in Section 6), will be installed prior to the excavations and can be designed to provide adequate excavation support. However on the south side of Osborne Street, temporary shoring will be required to provide lateral support to the railway grade. It is understood that the shoring will consist of 3 m high cantilevered sheet pile wall and it will be installed at 12 m offset form the centreline of the nearest track. The shoring should be designed for the earth pressures discussed in Section 6. The lateral railway loads acting on the shoring will be less than the loads shown on Figure 07 because of the greater offset between the shoring and the tracks.

It is understood that the temporary shoring will be designed using a sheet pile section similar to the section used in the permanent wall at the north approach (i.e., PZ 27). Lateral displacement of the shoring wall and the anticipated ground displacement on the retained side are expected to be less than the corresponding displacements calculated for the permanent sheet pile wall in Section 7 because of the greater offset from the railway track (i.e., 12 m offset for the temporary shoring versus 7 m offset for the permanent wall). All necessary measures should be undertaken to protect against undermining the foundation or stability of existing infrastructure.

9. Closure

The findings and recommendations of this Memorandum were based on the results of field and laboratory investigations, combined with an interpolation of soil and ground water conditions between the test hole locations. If conditions are encountered that appear to be different from those shown by the test holes drilled at this site and described in this Memorandum, or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and adjusted, if necessary.

Soil conditions, by their nature, can be highly variable across a site. The placement of fill and prior construction activities on a site can contribute to the variability especially near surface soil conditions. A contingency should be included in the construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.

AECOM requests the opportunity to review drawings and specifications related to this work or other designs based on the recommendations provided in this report to confirm that said recommendations have been correctly interpreted.

AECOM Canada Ltd.

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

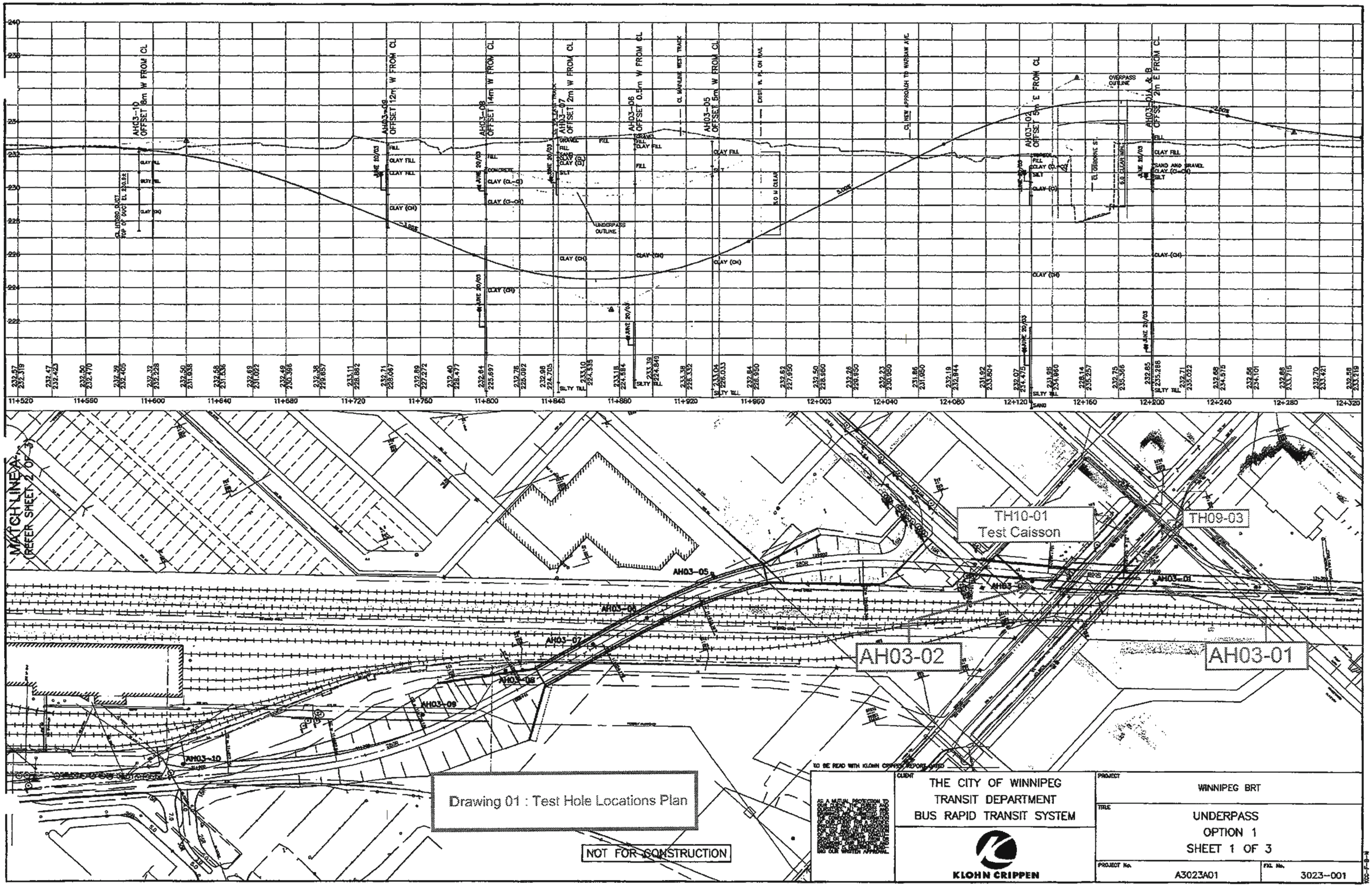
Reviewed By:

Ken Skaftfeld, P.Eng.
Lead Geotechnical Engineer
Environment

PROVINCE OF MANITOBA
F. S.
KHALIL
Member
31366
09.04.10
REGISTERED PROFESSIONAL ENGINEER

APPENDIX A

Test Hole Location Plan
Test Hole Logs
Test Caisson Log



Drawing 01 : Test Hole Locations Plan

TO BE READ WITH KLOHN CRIPPEN REPORT DATED

CLIENT
THE CITY OF WINNIPEG
TRANSIT DEPARTMENT
BUS RAPID TRANSIT SYSTEM

KLOHN CRIPPEN

PROJECT
WINNIPEG BRT

TITLE
UNDERPASS
OPTION 1
SHEET 1 OF 3

PROJECT No.
A3023A01

FIG. No.
3023-001

TEST HOLE LOG

STARTED: June 16, 2003 **FINISHED:** June 16, 2003
DRILL METHOD: Solid Stem Auger
GROUND ELEV. (m): 233.32
COORDINATES (m): N 5526235.4 E 633589.4

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.
1		Grab	1
	4 3 2	SPT	2
2		Grab	3
		SY	3b
3	3 4 4	SPT	4
4		SY	5
5	2 2 2	SPT	6
6		SY	7
7			
8	3 2 3	SPT Grab	8
9		SY	9
10			

DESCRIPTION OF MATERIALS

FILL
Cinders, trace gravel, trace sand, black, dry, upper 10cm is silty with rootlets, sparse grass coverage

0.50 / 232.82 CLAY (CI)
Silty, medium plasticity, trace fine gravel, mottled light brown to dark grey, some rusting, moist (FILL)

1.80 / 231.52 SAND AND GRAVEL (SW-GW)
Fine to coarse sand, fine to medium gravel, moist (FILL)

2.05 / 231.27 CLAY (CI-CH)
Medium to high plasticity, firm to stiff, light greyish brown, moist

2.45 / 230.87 SILT (ML)
Trace clay, non-plastic to low plasticity, soft, light brown, wet
2.45 m: first appearance of water in hole

2.75 / 230.57 CLAY (CH)
Trace to some silt, high plasticity, firm to stiff, moist, olive grey mottled with brown, oxidized in spots
3.0 - 3.8 m: occasional pebbles
3.8 m: Sample 5 = 24% silt, 76% clay

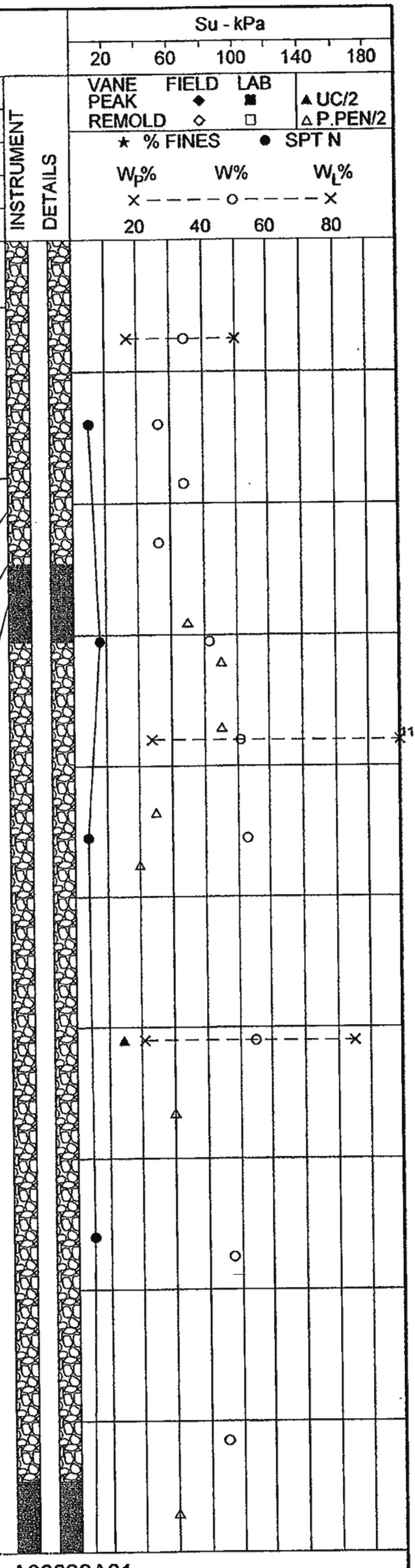
4.5m: occasional gypsum inclusions (white, powdery)

Unconfined Compressive Strength = 57.4 kPa

7.6 m: no recovery in split-spoon, grab sample taken

9.1 m: occasional inclusions of sandy silt, trace gravel, becoming grey to olive grey

Continued Next Page



KLOHN CRIPPEN

PROJECT NO.: A03023A01
PROJECT: Winnipeg BRT
LOCATION: Southwest Corridor - Overpass
LOGGED BY: JNH **CHECKED BY:** DWR
SHEET 1 OF 2 **HOLE NO.:** AH03-01A

TEST HOLE LOG

STARTED: June 16, 2003 **FINISHED:** June 16, 2003
DRILL METHOD: Solid Stem Auger
GROUND ELEV. (m): 233.32
COORDINATES (m): N 5526235.4 E 633589.4

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.
11	2 2 2	SPT	10
12		SY	10b
14	2 2 3	SPT	11
		SPT	12
15		SY	13
16		Grab	14
17			
18			
19			
20			

DESCRIPTION OF MATERIALS

10.7 m: trace gravel

13.7 m: frequent inclusions / lenses of TILL
- Samples 11 & 12 from same SPT test but split based on material type

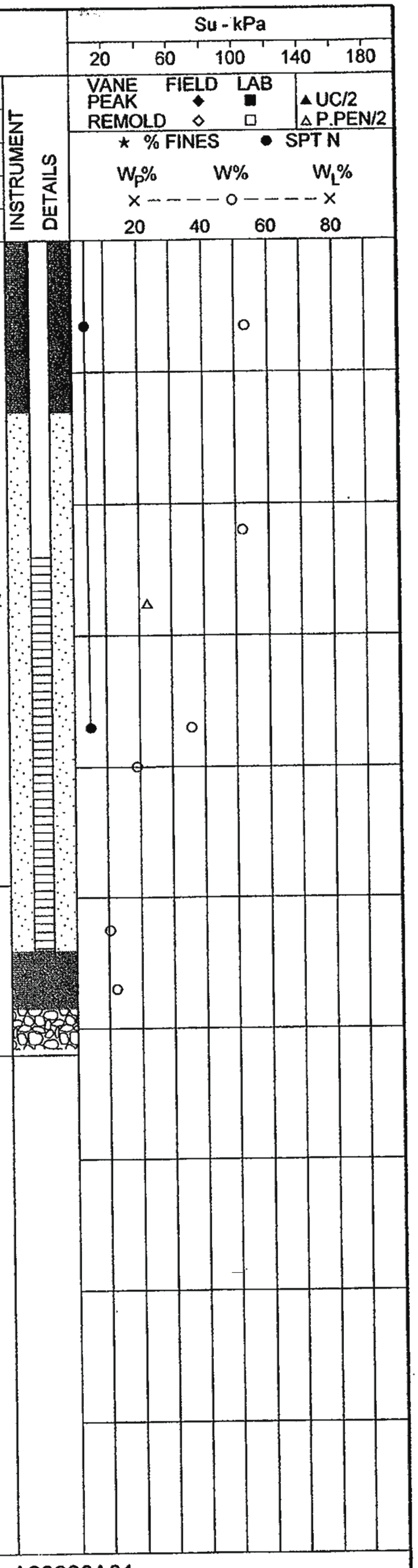
14.0 - 14.15: TILL inclusion (silt, trace sand, trace gravel, light greyish brown)

14.90
218.42
SILT (ML)
Gravely (fine to medium), some sand (fine to coarse), light brown to reddish brown, wet (TILL-LIKE)

16.20
217.12
16.20 m: Auger refusal

End of Hole at: 16.2 m

25 mm standpipe installed in lockable steel casing
Stick-up = 0.94 m
Initial water level = 9.4 m bgl
June 17/03 water level = 12.41 m bgl
June 20/03 water level = 12.76 m bgl



KLOHN CRIPPEN

PROJECT NO.: A03023A01
PROJECT: Winnipeg BRT
LOCATION: Southwest Corridor - Overpass
LOGGED BY: JNH **CHECKED BY:** DWR
SHEET 2 **OF** 2 **HOLE NO.:** AH03-01A

KC_TEST_HOLE-SI SW_ U ROCK_006.GDT 24/10/03

TEST HOLE LOG

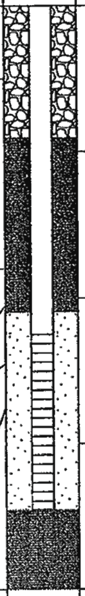
STARTED: June 16, 2003 **FINISHED:** June 16, 2003

DRILL METHOD: Solid Stem Auger

GROUND ELEV. (m): 233.32

COORDINATES (m): N 5526234 E 633589

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS
					FILL Cinders, trace gravel, trace sand, black, dry, upper 10cm is silty with rootlets, sparse grass coverage
0.50 232.82					CLAY (CL-CI) Silty, low to medium plasticity, trace fine gravel, mottled light brown to dark grey, some rusting, moist (FILL)
1.80 231.52					SAND AND GRAVEL (SW-GW) Fine to coarse sand, fine to medium gravel, moist (FILL)
2.05 231.27					CLAY (CI) Medium plasticity, firm to stiff, light greyish brown, moist
2.45 230.87					SILT (ML) Trace clay, non-plastic to low plasticity, soft, light brown, wet 2.45 m: first appearance of water in hole
2.75 230.57					CLAY (CI-CH) Medium to high plasticity, firm to stiff, moist, olive grey mottled with brown, oxidized in spots
4.00 229.32					

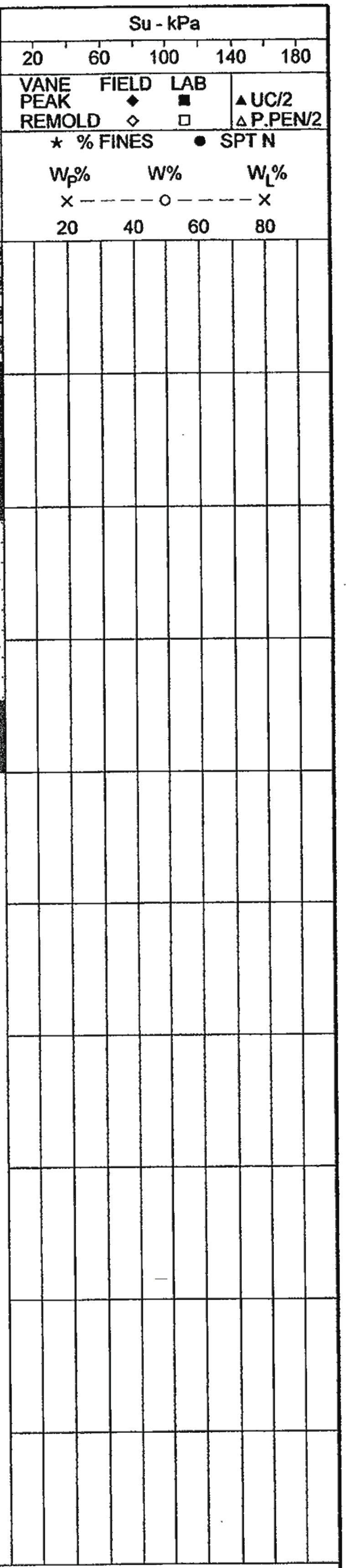


End of Hole at: 4.0 m

New hole drilled for AH03-01B due to bridging in AH03-1A (unable to install shallow standpipe)

Located ~1 m South of AH03-1A. No logging or sampling performed on hole.

25 mm standpipe installed in lockable steel casing
Stick-up = 0.94 m
Initial water level estimated from drilling = 2.44 m bgl
June 17/03 water level = 2.77 m bgl
June 20/03 water level = 2.63 m bgl



KLOHN CRIPPEN

PROJECT NO.: A03023A01

PROJECT: Winnipeg BRT

LOCATION: Southwest Corridor - Overpass

LOGGED BY: JNH **CHECKED BY:** DWR

SHEET 1 OF 1 **HOLE NO.:** AH03-01B

TEST HOLE LOG

STARTED: June 18, 2003 **FINISHED:** June 18, 2003

DRILL METHOD: Solid Stem Auger

GROUND ELEV. (m): 232.10

COORDINATES (m): N 5526165.9 E 633565.1

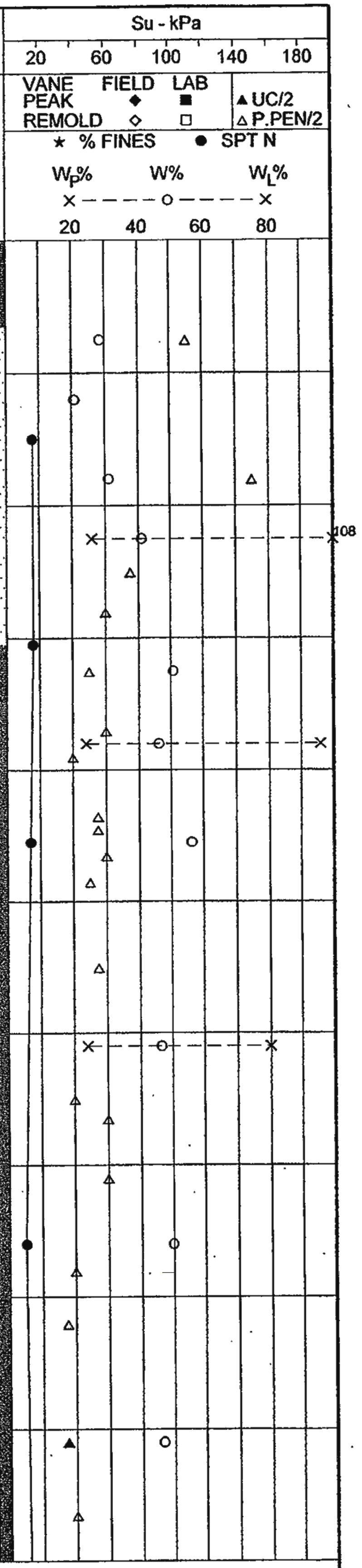
DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.
		Grab	1
1		Grab	2
	3 3 5	SPT	
2		Grab	3
		SY	4
3	4 4 4	SPT Grab	5
4		SY	6
5	3 3 4	SPT	7
6		SY	8
7			
8	2 2 3	SPT	9
9		SY	10
10			

DESCRIPTION OF MATERIALS

Depth / Elev.	Description
0.15 231.95	TOPSOIL - Silty, black, dry, rootlets, dense vegetation cover
0.70 231.40	FILL Silty, some cinders, some gravel (fine to coarse), black, dry
0.85 231.25	CLAY (CL-CI) Low to medium plasticity, stiff, dark grey
1.70 230.40	SILT (ML) Non-plastic to low plasticity, soft, light brown to light brownish yellow, moist PP/2 < 25 kPa 1.5m: SPT picked up slough only, no sample recovery
2.50 229.60	CLAY (CI-CH) Trace silt, medium to high plasticity, stiff to very stiff, brown to olive grey mottled with some dark grey, dry to moist 2.25 m: Sample 4 = 12% silt, 88% clay
6.10 226.00	CLAY (CH) Trace silt, high plasticity, firm to stiff, brown to olive grey, moist, gypsum deposits 3.05 m: SPT numbers are too high - pushing rock, no sample 3.25 m: some rusty brown spots Sample 6: 0.01% sulphate by weight Consolidation test performed
	CLAY (CH) High plasticity, firm to stiff, olive grey, moist, gypsum deposits 6.1m: some small flecks of silt beginning to appear 7.6m: becoming olive grey to grey, some rusty seams, frequent flecks of silt 9.1m: beginning of occasional pebbles (subangular to angular, 2-15mm dia.) Unconfined Compressive Strength = 69.4 kPa

Continued Next Page

INSTRUMENT DETAILS



KLOHN CRIPPEN

PROJECT NO.: A03023A01

PROJECT: Winnipeg BRT

LOCATION: Southwest Corridor - Overpass

LOGGED BY: JNH **CHECKED BY:** DWR

SHEET 1 **OF** 2 **HOLE NO.:** AH03-02

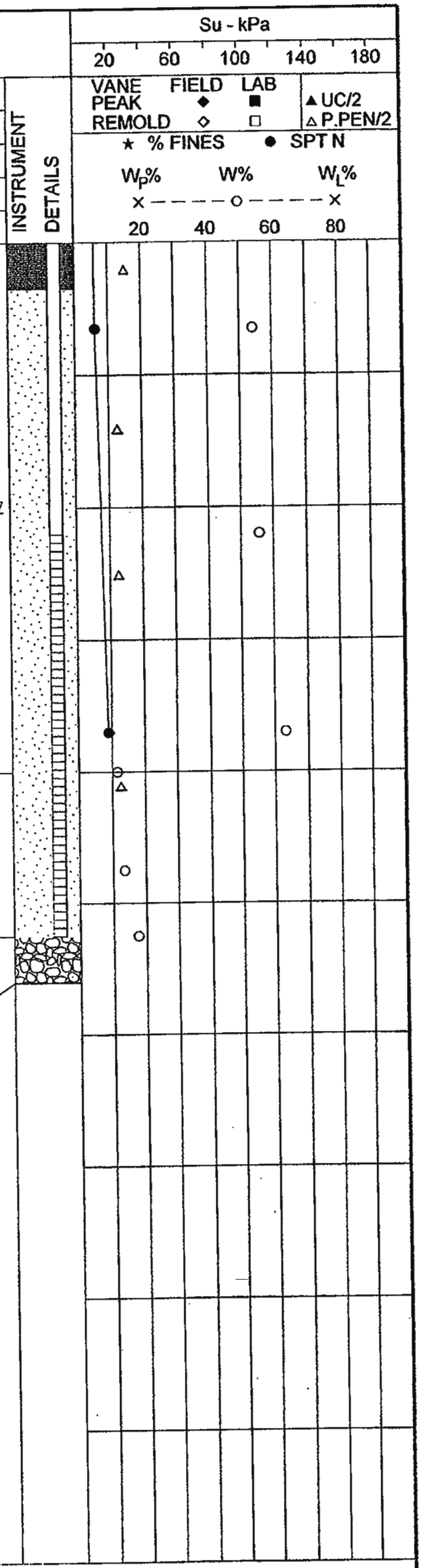
TEST HOLE LOG

STARTED: June 18, 2003 **FINISHED:** June 18, 2003
DRILL METHOD: Solid Stem Auger
GROUND ELEV. (m): 232.10
COORDINATES (m): N 5526165.9 E 633565.1

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	DESCRIPTION OF MATERIALS
11	2 3 3	SPT	11	10.65 - 14.0m: becoming soft to firm PP/2 < 25 kPa
12		SY	12	
13	6 5 4	SPT	13a 13b	13.7m: SPT sample split
14				14.00 218.10 SILT (ML) Some gravel (fine to coarse, subangular to angular), trace coarse sand, trace clay, non-plastic to low plasticity, soft to firm, pinkish brown becoming light brown, wet (TILL-LIKE) 14.75 m: becoming gravely, firm
15		Grab	14	15.25 216.85 SAND (SM) Fine, silty, light brown, wet
		Grab	15	15.60 216.50 15.60 m: Auger refusal
16				
17				
18				
19				
20				

End of Hole at: 15.6 m

2 x 25 mm standpipes installed in lockable steel casing
Stick-up = 0.90 m for both
AH03-02A: Initial water level = 12.34 m bgl
June 20/03 water level = 12.03 m bgl
AH03-02B: Initial water level = 0.73 m bgl
June 20/03 water level = 1.71 m bgl



KLOHN CRIPPEN

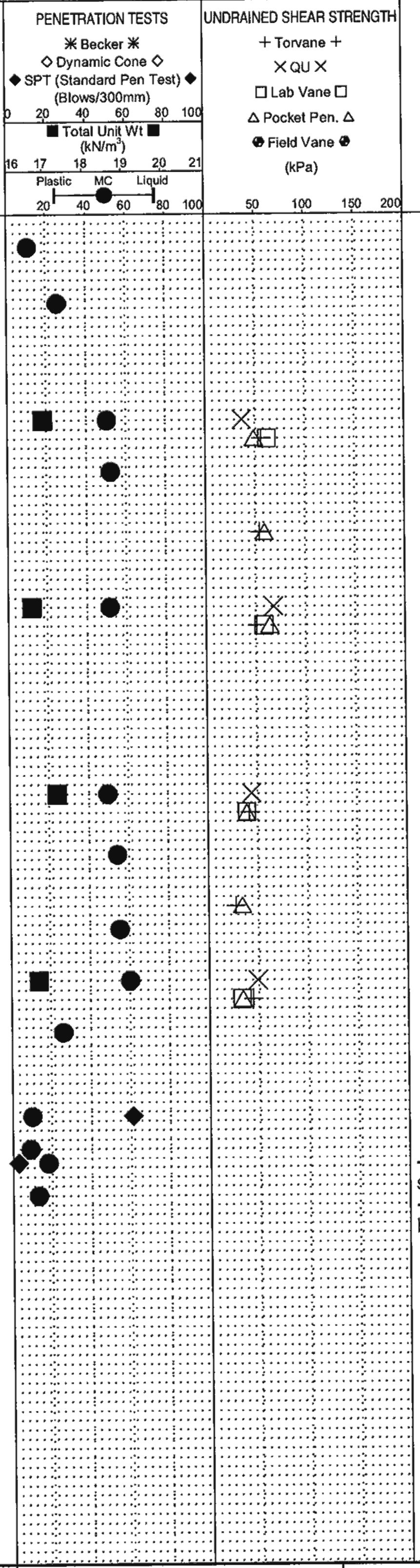
PROJECT NO.: A03023A01
PROJECT: Winnipeg BRT
LOCATION: Southwest Corridor - Overpass
LOGGED BY: JNH **CHECKED BY:** DWR
SHEET 2 **OF** 2 **HOLE NO.:** AH03-02

PROJECT: Southwest Rapid Transit Corridor
CLIENT: COW/Dillon Consulting
TESTHOLE NO: **TH-09-03**
LOCATION: At East abutment of proposed bridge N 5526240 E 633577
PROJECT NO.: 60115506
CONTRACTOR: Paddock Drilling Ltd.
METHOD: 125mm Solid Stem Augers
ELEVATION (m): 233.30

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
0	FILL - Sand, gravelly, brown, frozen	■	G41			233
1	CLAY - silty, mottled grey and black, frozen					232
	SILT- light brown, frozen	■	G42			
2	CLAY - silty - mottled brown - moist, firm to stiff - high plasticity - trace silt and gypsum inclusions, trace oxidation - frozen to 1.8m	■	G43			231
3		▥	T44			230
4		■	G45			229
5		▥	T46			228
		■	G47			
6	- grey, firm below 6.4m	▥	T48			227
7		■	G49			226
8		■	G50			225
9		▥	T51			224
10	- soft below 10.1m	■	G52			223
		▥	T53			
11	- trace sand, trace gravel below 11m	■	G54			222
12	- trace till inclusions <50mm thick below 12.2m	▥	T55			221
13		■	G56			220
14		▥	T57			219
	TILL - Silt, sandy, trace gravel, light grey, moist, compact to dense, low plasticity	☒	S58	61		
15	SAND - medium grained, wet	☒	S59A		- Sand blow-up affected sampling	218
	TILL - Silt, sandy, reddish brown, moist, compact		S59B	3		
16	SAND - medium grained, wet		S60		- No sample due to sand blow-up	217
			G61			
17	AUGER REFUSAL AT 16.5m ON SUSPECTED LIMESTONE BEDROCK					216
18						215
19						214
20						213
21						212
22						

Notes:
1. Seepage from sand below 14.8m.
2. Blow-up of sand below 14.8m.
3. Installed standipipe piezometer at 15.5m complete with Cassegrande tip and flush mount cover. Sand blow-up from bottom of test hole to 14.9m. Backfilled with sand between 14.9 and 13.4m, and with bentonite pellets to surface.



LOGGED BY: KT
REVIEWED BY: FK
PROJECT ENGINEER: Faris Khalil
COMPLETION DEPTH: 16.46 m
COMPLETION DATE: 26/3/09

AECOM

PROJECT: Southwest Rapid Transit Corridor	CLIENT: COW/Dillon Consulting	TESTHOLE NO: **TH-10-01**
LOCATION: Near the proposed east pier N5526241 E633579 (Test Caisson)		PROJECT NO.: 60115506
CONTRACTOR: Subterranean Ltd	METHOD: Large Diameter Auger/Coring Bucket	ELEVATION (m): 233.30

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

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	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	COMMENTS	ELEVATION
0		FILL- SAND, gravelly, brown, frozen							233
1		CLAY-silty - grey, moist, firm -high plasticity							232
2									231
3									230
4									229
5									228
6									227
7									226
8									225
9									224
10									223
11									222
12									221
13									220
14									219
15		SILT (till) -wet, soft							218
16									217
17		-sand and gravel zone -water bearing BEDROCK-limestone (boulder and cobble size) -highly weathered broken pieces							216
18									215
19									214
20		CLAY-silty -grey, wet, soft -high plasticity							213
21		BEDROCK-limestone -massive, fine-grained -vertical and horizontal joints up to 40 mm thick							212
22									

AECOM

LOGGED BY: Faris Khalil	COMPLETION DEPTH: 27.74 m
REVIEWED BY: Faris Khalil	COMPLETION DATE: 3/4/10
PROJECT ENGINEER: Faris Khalil	Page 1 of 2

PROJECT: Southwest Rapid Transit Corridor | CLIENT: COW/Dillon Consulting | TESTHOLE NO: **TH-10-01**

LOCATION: Near the proposed east pier N5526241 E633579 (Test Caisson) | PROJECT NO.: 60115506

CONTRACTOR: Subterranean Ltd | METHOD: Large Diameter Auger/Coring Bucket | ELEVATION (m): 233.30

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

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	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	COMMENTS	ELEVATION
22	▦	-infill with silt and clay (very stiff to hard)							211
23	▦								210
24	▦								209
25	▦								208
26	▦								207
27	▦								206
28		END HOLE AT 27.7m IN LIMESTONE BEDROCK Notes: 1. Seepage within sand and gravel zone at ~16.5m; 2. Hole backfilled with G 30 conc. to elv. 220 m, with stabilized fill to ground surface.							205
29									204
30									203
31									202
32									201
33									200
34									199
35									198
36									197
37									196
38									195
39									194
40									193
41									192
42									191
43									190
44									

LOGGED BY: Faris Khalil | COMPLETION DEPTH: 27.74 m

REVIEWED BY: Faris Khalil | COMPLETION DATE: 3/4/10

PROJECT ENGINEER: Faris Khalil

APPENDIX B

Results of Deformation Analysis

REPORT

04/06/2010

User: UMA Engineering Ltd.

Title: BRT - Osborne Overpass
Permanent Sheet Pile wall
DEFORMATION ANALYSIS

Table of Contents

1. General Information

Table [1] units

Type	Unit
Length	m
Force	kN
Time	day

Table [2] Model dimensions

	min.	max.
X	0.000	55.000
Y	0.000	20.000

Table [3] Model

Model	Plane Strain
Element	15-Noded

2. Geometry

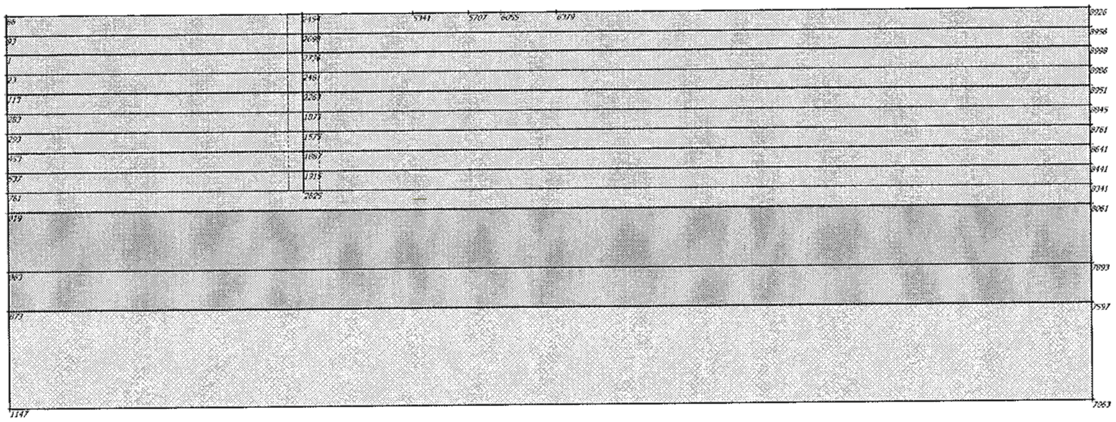


Fig. 1 Plot of geometry model with significant nodes

3. Structures

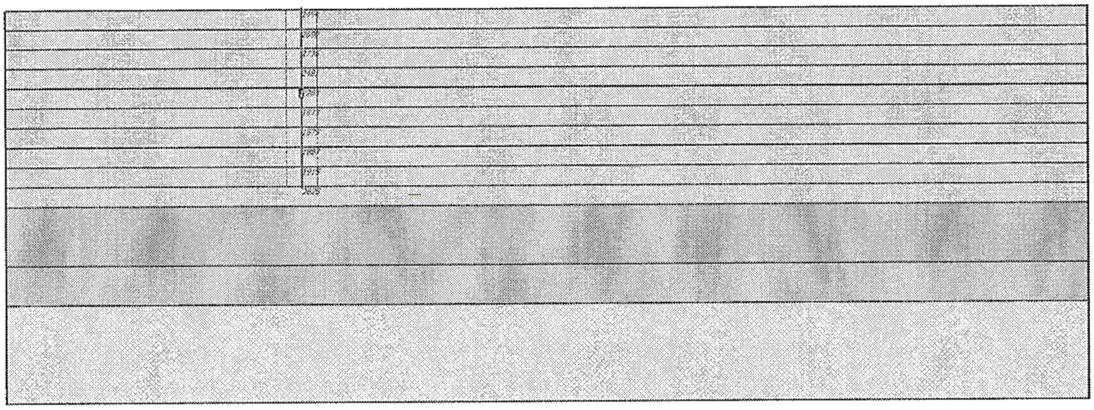


Fig. 2 Plot of geometry model with structures

Table [4] Beams

Plate no.	Data set	Length [m]	Nodes
1	SHEETPILE	9.000	2454, 2698, 2736, 2481, 2263, 1873, 1575, 1887, 1915, 2625.

4. Loads & boundary conditions

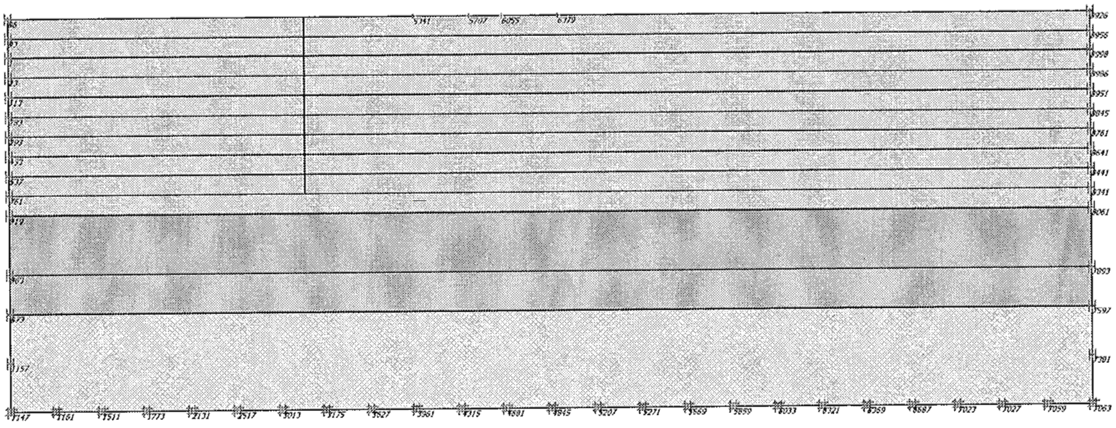


Fig. 3 Plot of geometry with loads & boundary conditions

Table [5] Node fixities

Node no.	Sign	Horizontal	Vertical	Node no.	Sign	Horizontal	Vertical
1147	#	Fixed	Fixed	453	||	Fixed	Free
7063	#	Fixed	Fixed	8641	||	Fixed	Free
1161	#	Fixed	Fixed	819	||	Fixed	Free
1511	#	Fixed	Fixed	8061	||	Fixed	Free
1773	#	Fixed	Fixed	863	||	Fixed	Free

Node no.	Sign	Horizontal	Vertical	Node no.	Sign	Horizontal	Vertical
2131	#	Fixed	Fixed	7893	||	Fixed	Free
2517	#	Fixed	Fixed	93	||	Fixed	Free
3013	#	Fixed	Fixed	8956	||	Fixed	Free
3175	#	Fixed	Fixed	1	||	Fixed	Free
3627	#	Fixed	Fixed	8998	||	Fixed	Free
3961	#	Fixed	Fixed	23	||	Fixed	Free
4315	#	Fixed	Fixed	8986	||	Fixed	Free
4681	#	Fixed	Fixed	113	||	Fixed	Free
4845	#	Fixed	Fixed	8951	||	Fixed	Free
5207	#	Fixed	Fixed	283	||	Fixed	Free
5271	#	Fixed	Fixed	8845	||	Fixed	Free
5669	#	Fixed	Fixed	293	||	Fixed	Free
5859	#	Fixed	Fixed	8761	||	Fixed	Free
6033	#	Fixed	Fixed	537	||	Fixed	Free
6321	#	Fixed	Fixed	8441	||	Fixed	Free
6369	#	Fixed	Fixed	761	||	Fixed	Free
6687	#	Fixed	Fixed	8341	||	Fixed	Free
7023	#	Fixed	Fixed	873	||	Fixed	Free
7027	#	Fixed	Fixed	7597	||	Fixed	Free
7059	#	Fixed	Fixed	7381	||	Fixed	Free
8926	||	Fixed	Free	1157	||	Fixed	Free
86	||	Fixed	Free				

4.1. Load system A

Table [6] Distributed loads A

Loads no.	First node	qx [kN/m/m]	qy [kN/m/m]	Last node	qx [kN/m/m]	qy [kN/m/m]
1	5707	0.000	0.000	5341	0.000	0.000

4.2. Load system B

Table [7] Distributed loads B

Loads no.	First node	qx [kN/m/m]	qy [kN/m/m]	Last node	qx [kN/m/m]	qy [kN/m/m]
1	6379	0.000	0.000	6055	0.000	0.000

5. Material data

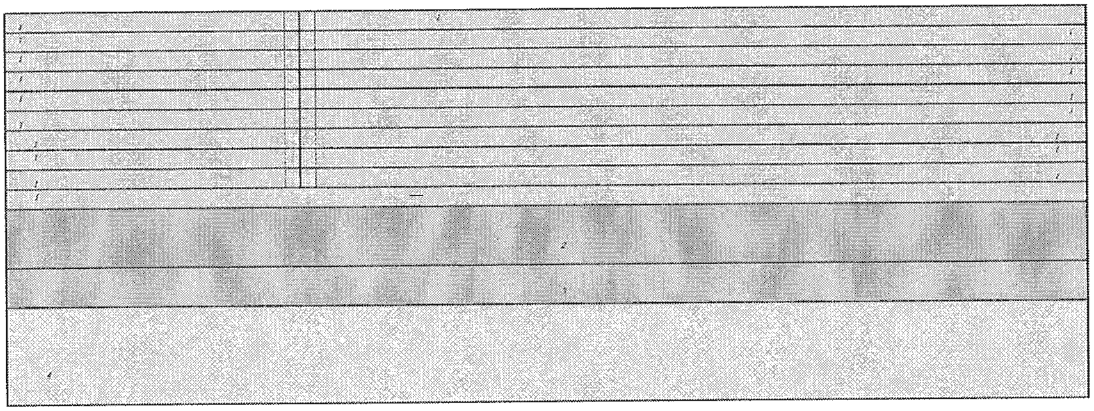


Fig. 4 Plot of geometry with material data sets

Table [8] Soil data sets parameters

Mohr-Coulomb		1 CLAY	2 CLAY1	3 CLAY TILL	4 LIMESTONE
Type		UnDrained	UnDrained	Drained	Drained
γ_{unsat}	[kN/m³]	15.00	15.00	17.00	22.00
γ_{sat}	[kN/m³]	17.00	17.00	20.00	24.00
k_x	[m/day]	0.000	0.000	0.000	1.000
k_y	[m/day]	0.000	0.000	0.000	1.000

Mohr-Coulomb		1 CLAY	2 CLAY1	3 CLAY TILL	4 LIMESTONE
e_{init}	[-]	0.500	0.500	0.500	0.500
c_k	[-]	1E15	1E15	1E15	1E15
E_{ref}	[kN/m^2]	16000.000	12000.000	100000.000	1000000.000
ν	[-]	0.350	0.350	0.350	0.200
G_{ref}	[kN/m^2]	5925.926	4444.444	37037.037	416666.667
E_{oed}	[kN/m^2]	25679.012	19259.259	160493.827	1111111.111
c_{ref}	[kN/m^2]	40.00	30.00	0.50	0.25
φ	[°]	0.50	0.50	27.00	45.00
ψ	[°]	0.00	0.00	0.00	0.00
E_{inc}	[kN/m^2/m]	0.00	0.00	0.00	0.00
y_{ref}	[m]	0.000	0.000	0.000	0.000
$c_{increment}$	[kN/m^2/m]	0.00	0.00	0.00	0.00
$T_{str.}$	[kN/m^2]	0.00	0.00	0.00	0.00
$R_{inter.}$	[-]	0.70	1.00	1.00	1.00
Interface permeability		Neutral	Neutral	Neutral	Neutral

Table [9] Beam data sets parameters

no.	Identification	EA [kN/m]	EI [kNm^2/m]	w [kN/m/m]	ν [-]	Mp [kNm/m]	Np [kN/m]
1	SHEETPILE	3.44E6	51200.00	1.34	0.00	1E15	1E15

6. Calculation phases

Table [10] List of phases

Phase	Ph-No.	Start phase	Calculation type	Load input	First step	Last step
Initial phase	0	0		-	0	0
<Phase 6>	6	0	Plastic analysis	Total multipliers	17	19
<Phase 7>	7	6	Plastic analysis	Staged construction	20	21
<Phase 12>	12	7	Plastic analysis	Staged construction	22	23
<Phase 8>	8	7	Plastic analysis	Staged construction	24	26
<Phase 9>	9	8	Plastic analysis	Staged construction	27	28
<Phase 10>	10	9	Plastic analysis	Staged construction	13	14
<Phase 11>	11	10	Plastic analysis	Staged construction	15	16

Table [11] Staged construction info

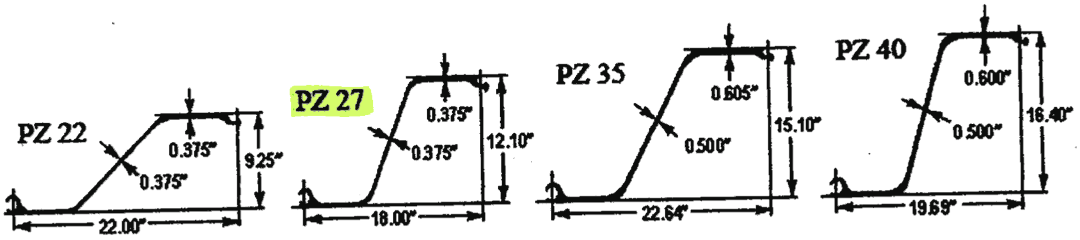
Ph-No.	Active clusters	Inactive clusters	Active beams	Active geotextiles	Active anchors
0	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22.				
7	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22.				
12	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22.		1.		
8	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22.		1.		

Ph-No.	Active clusters	Inactive clusters	Active beams	Active geotextiles	Active anchors
9	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 14, 15, 16, 17, 18, 19, 20, 21, 22.	13.	1.		
10	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 14, 15, 16, 17, 18, 19, 20, 21, 22.	12, 13.	1.		
11	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 14, 15, 16, 17, 18, 19, 20, 21, 22.	11, 12, 13.	1.		

6.3. Total multipliers

Table [12] Total multipliers - input values

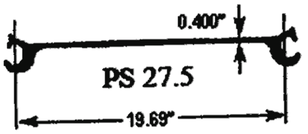
Ph-No.	Displ.	Load A	Load B	Weight	Accel	Time	s-f
0	1.0000	1.0000	1.0000	0.0000	0.0000	0.0000	1.0000
6	1.0000	1.0000	1.0000	1.0000	0.0000	0.0000	1.0000
7	1.0000	1.0000	1.0000	1.0000	0.0000	0.0000	1.0000
12	1.0000	1.0000	1.0000	1.0000	0.0000	0.0000	1.0000
8	1.0000	96.0000	96.0000	1.0000	0.0000	0.0000	1.0000
9	1.0000	1.0000	1.0000	1.0000	0.0000	0.0000	1.0000
10	1.0000	1.0000	1.0000	1.0000	0.0000	0.0000	1.0000
11	1.0000	1.0000	1.0000	1.0000	0.0000	0.0000	1.0000

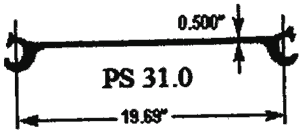


					Per Single Section						Per Foot Of Wall			
Section Designation	Nominal Width	Wall Depth (Height)	Web Thickness	Flange Thickness	Area	Weight Per Foot	Moment of Inertia	Section Modulus	Total Surface Area	Nominal Coating Area	Area	Weight Per Foot	Moment Of Inertia	Section Modulus
	in.	in.	in.	in.	$in.^2$	lbs/ft	$in.^4$	$in.^3$	ft^2/ft	ft^2/ft	$in.^2/ft$	lbs/ft^2	$in.^4/ft$	$in.^3/ft$
PZ 22	22	9.25	0.375	0.375	12.2	41.5	156	33.7	4.96	4.46	6.65	22.6	85.1	18.4
PZ 27	18	12.1	0.375	0.375	12.2	41.5	281	46.4	4.96	4.46	8.13	27.7	187.3	31
PZ 35	22.64	15.1	0.5	0.605	19.4	66	697	92.3	5.83	5.33	10.28	35	369.4	48.9
PZ 40	19.69	16.4	0.5	0.6	19.28	65.6	824.8	100.6	5.83	5.33	11.75	40	502.7	61.3
All Dimensions Are Nominal														

Flat Sections

Typically used for circular cell design applications, flat sheet pile is a proven technology that is still used on many projects. While these sheet piles offer much less beam strength than Z piles, the sections use a Thumb and Finger interlock with a three-point contact connection to generate significant pull strength to counter the hoop stress applied to them.





			Per Single Section				Per Foot Of Wall		
Section Designation	Nominal Width	Web Thickness	Weight Per Foot	Moment of Inertia	Section Modulus	Nominal Coating Area	Weight Per Foot	Moment Of Inertia	Section Modulus
	in.	in.	lbs/ft	$in.^4$	$in.^3$	ft^2/ft	lbs/ft^2	$in.^4/ft$	$in.^3/ft$
PS 27.5	19.69	0.4	45.2	5	3.2	3.68	27.5	3	1.9
PS 31.0	19.69	0.5	50.9	5	3.2	3.68	31	3	1.9
All Dimensions Are Nominal									

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Kansas City, Kansas
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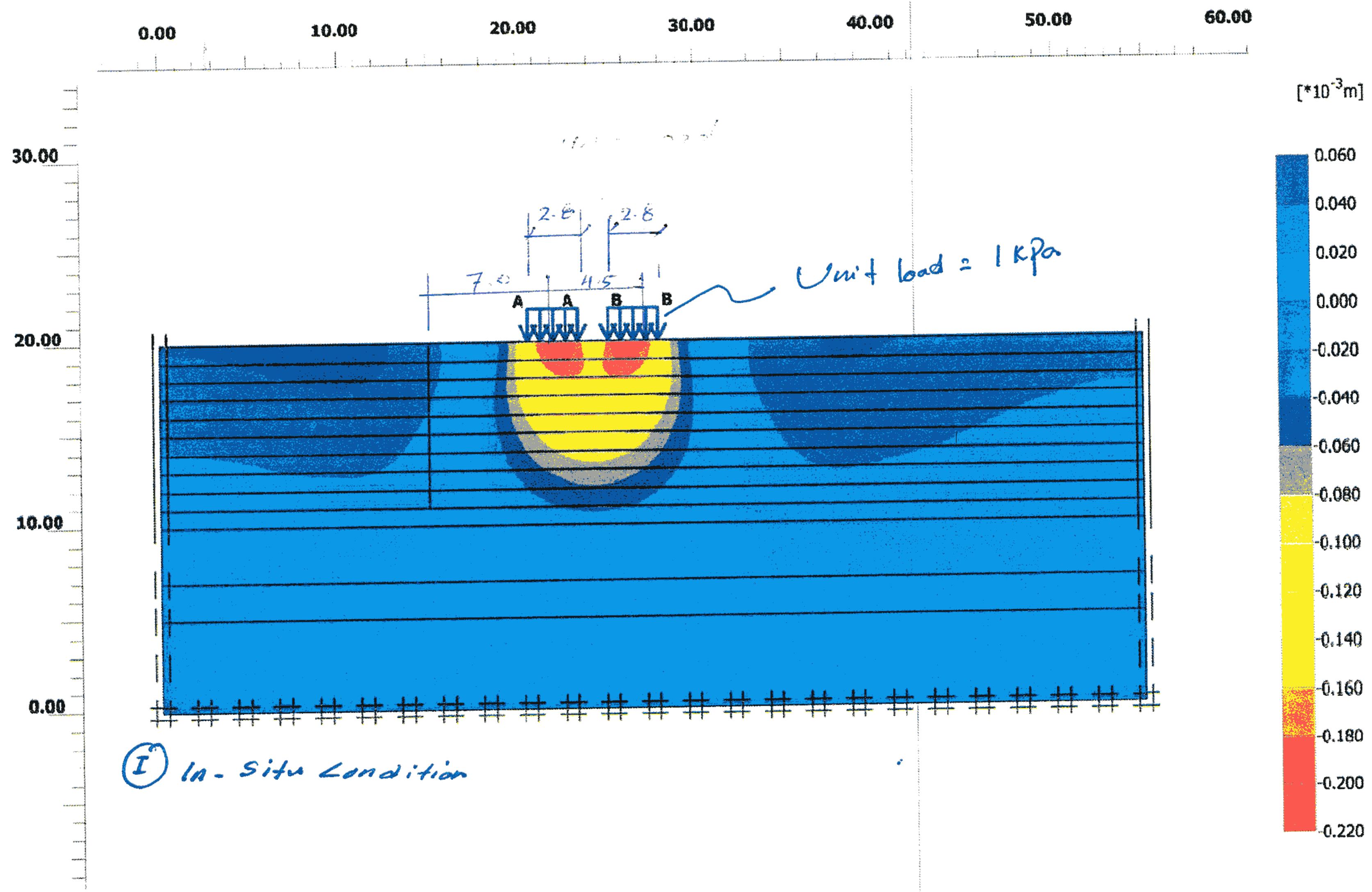
Minneapolis, Minnesota
Ph: 952.469.6060



① In-Situ Condition

Vertical displacements (Uy)

Extreme Uy -201.66*10^{-6} m ≈ 0.2 mm

PLAXIS

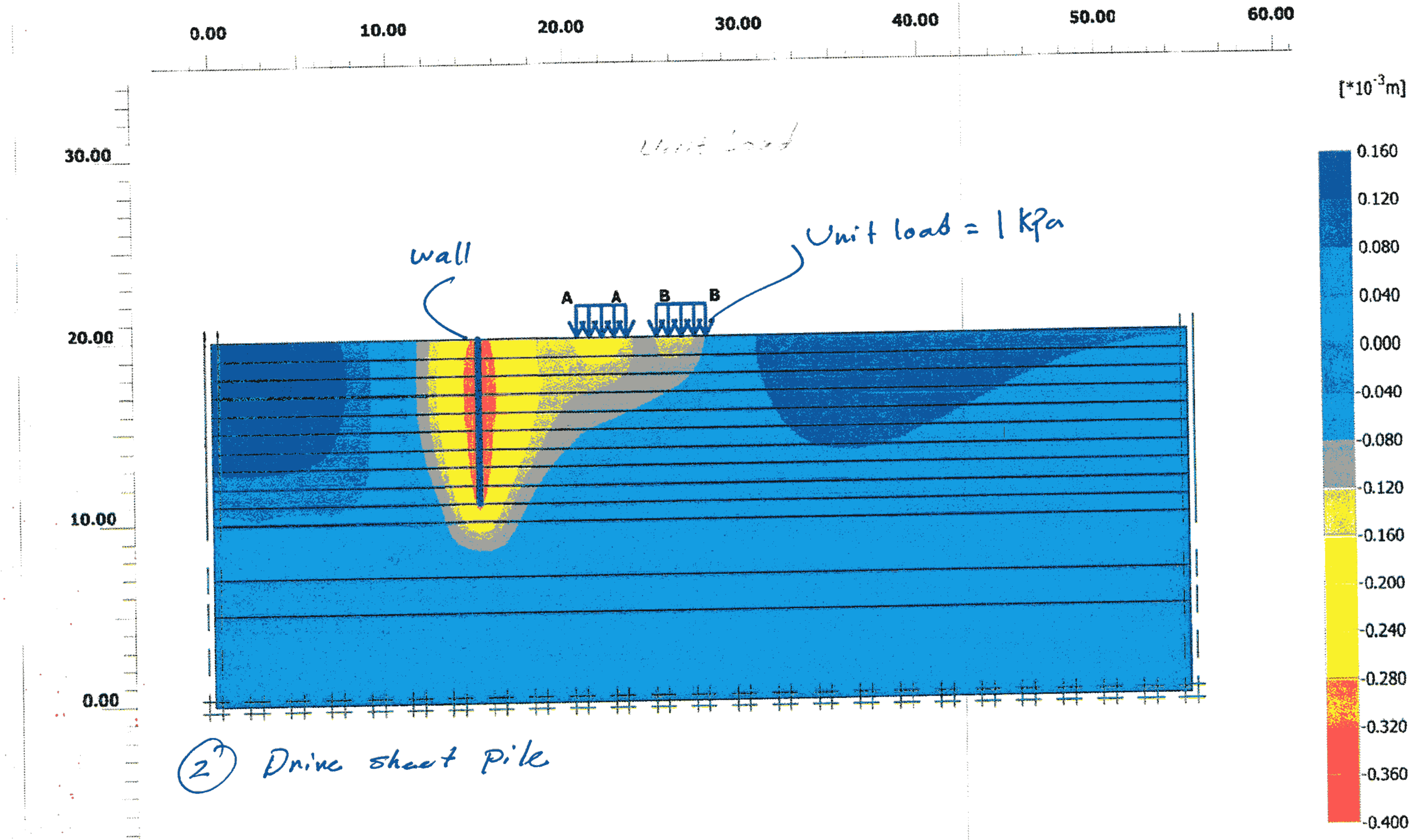
Finite Element Code for Soil and Rock Analyses

Version 8.5.0.1133

Project description
Sheetwall 28R

Project name	*Step*	*Date*	*User name*
Sheetwall 28R	5	04/01/10	UMA Engineering Ltd.

Plaxis 8.5



Unit load

wall

Unit load = 1 kPa

② Drive sheet pile

Vertical displacements (Uy)

Extreme Uy -396.47×10^{-6} m = 0.4 mm

PLAXIS

Finite Element Code for Soil and Rock Analyses

Project description
Sheetwall 28R

Project name
Sheetwall 28R

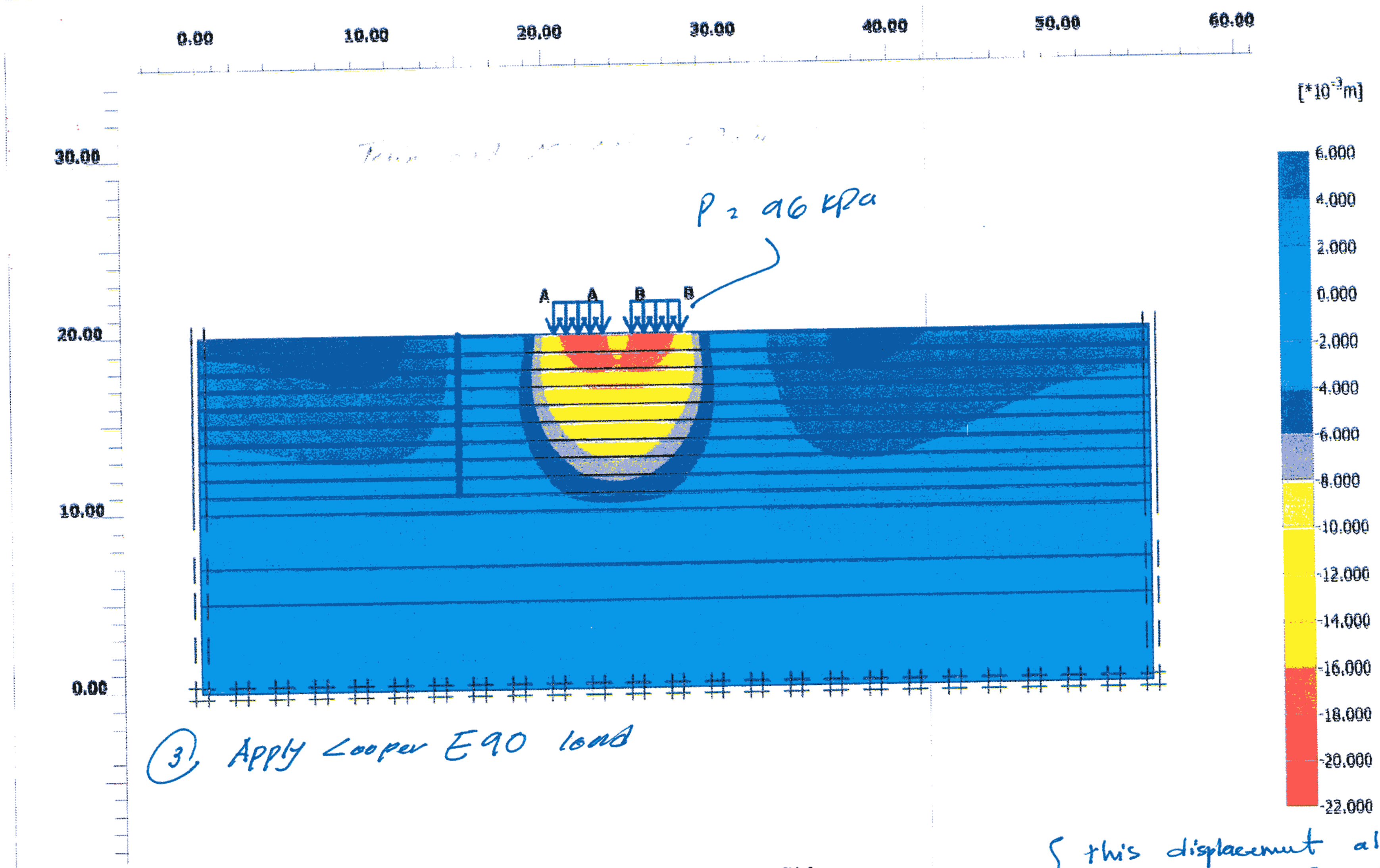
Step
7

Date
04/01/10

User name
UMA Engineering Ltd.

Version 8.5.0.1133

Plaxis 8.5



P = 96 kPa

③ Apply Cooper E90 load

Vertical displacements (Uy)

Extreme Uy -20.74*10^{-3} m = 20.7 mm

{ this displacement already Occurred under Current railway traffic regardless of the excavation

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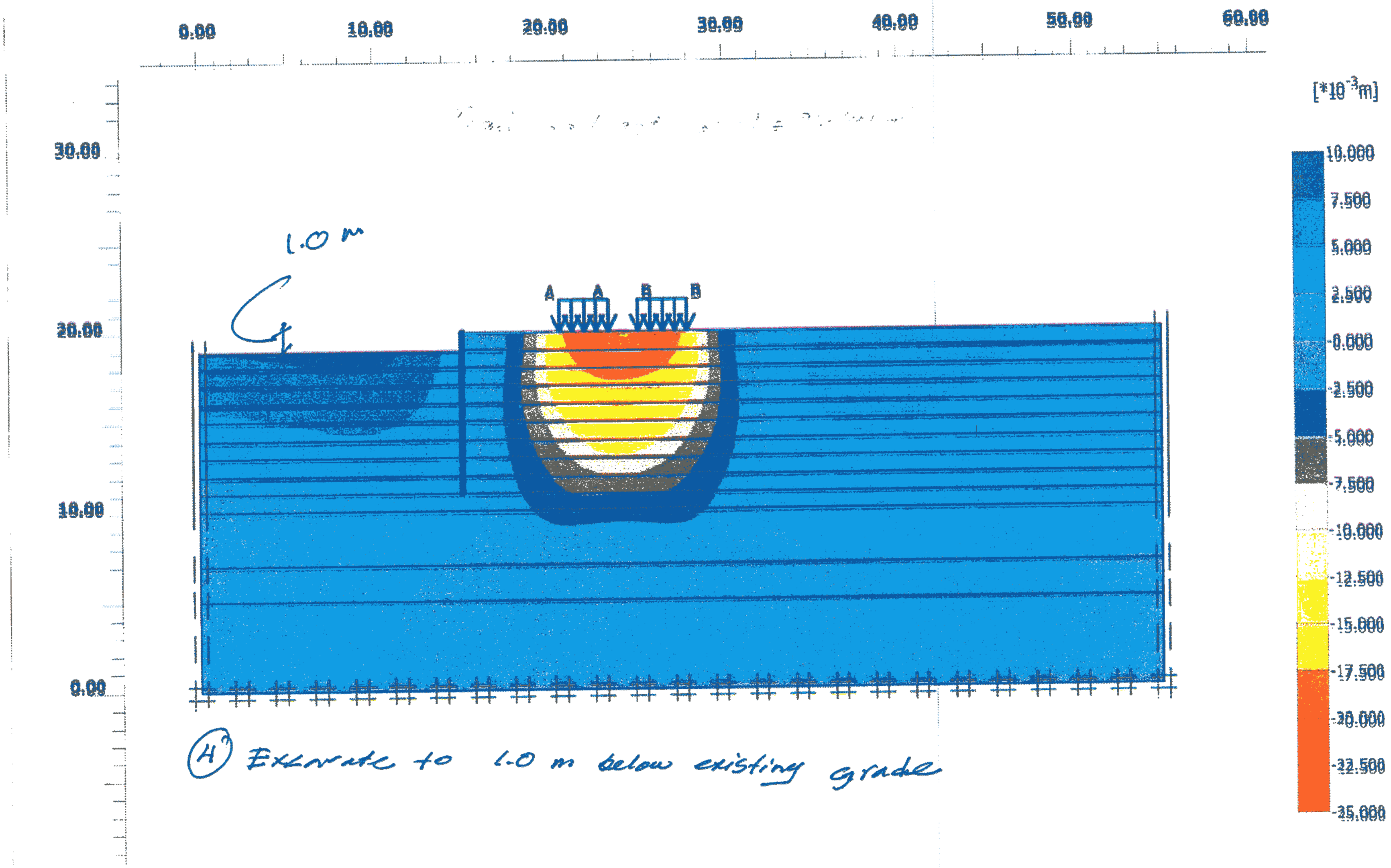
Finite Element Code for Soil and Rock Analyses

Version 8.5.0.1133

Project description			
Sheetwall28R			
Project name	Step	Date	User name
Sheetwall 28R	10	04/01/10	UMA Engineering Ltd.

Plaxis 8.5



④ Excavate to 1.0 m below existing grade

Vertical displacements (Uy)

Extreme Uy -23.33×10^{-3} m = 23.3 mm

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.5.0.1133

Project description
Sheetwall28R

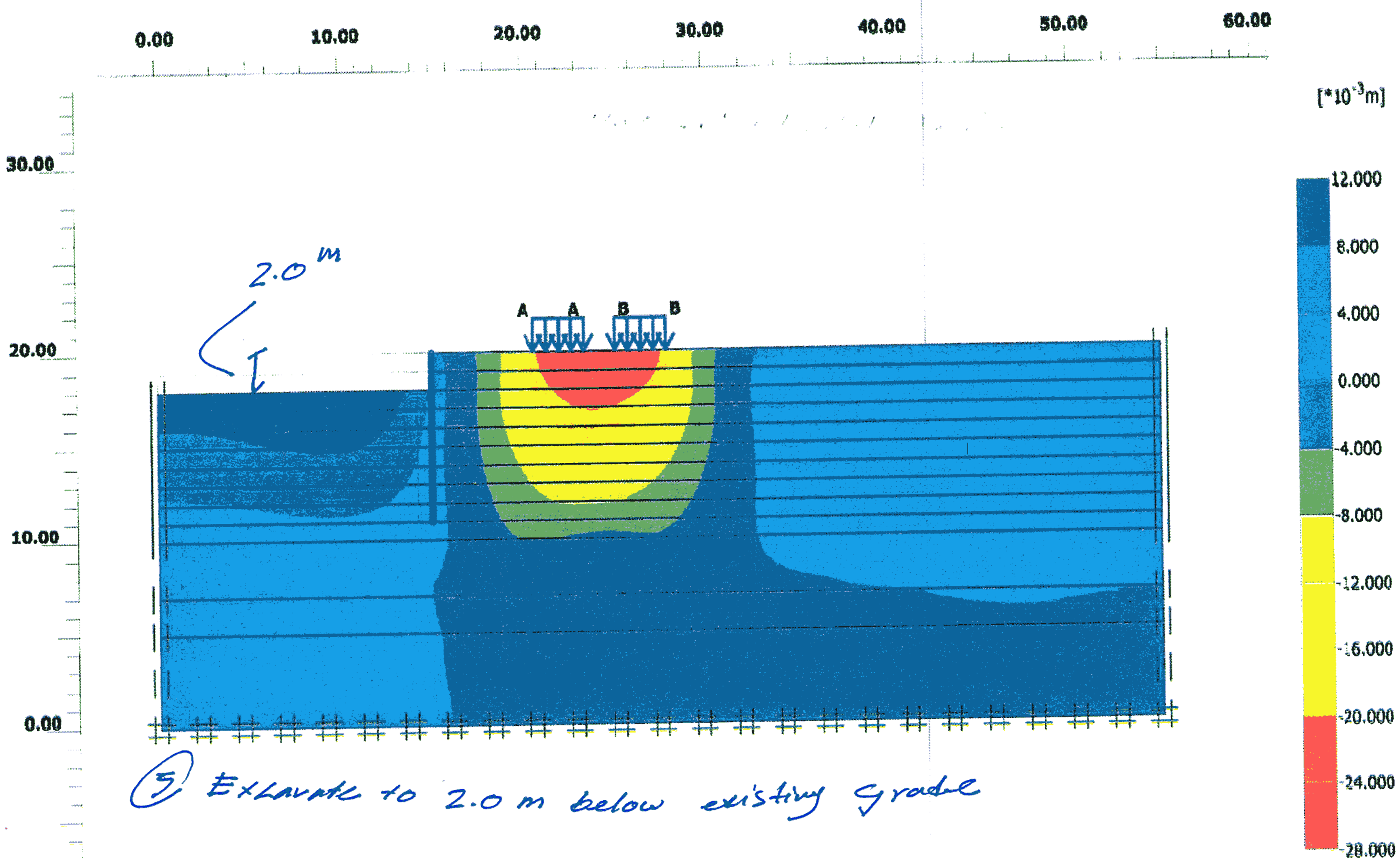
Project name
Sheetwall 28R

Step
12

Date
04/01/10

User name
UMA Engineering Ltd.

Plaxis 8.5



2.0 m

⑤ Excavate to 2.0 m below existing grade

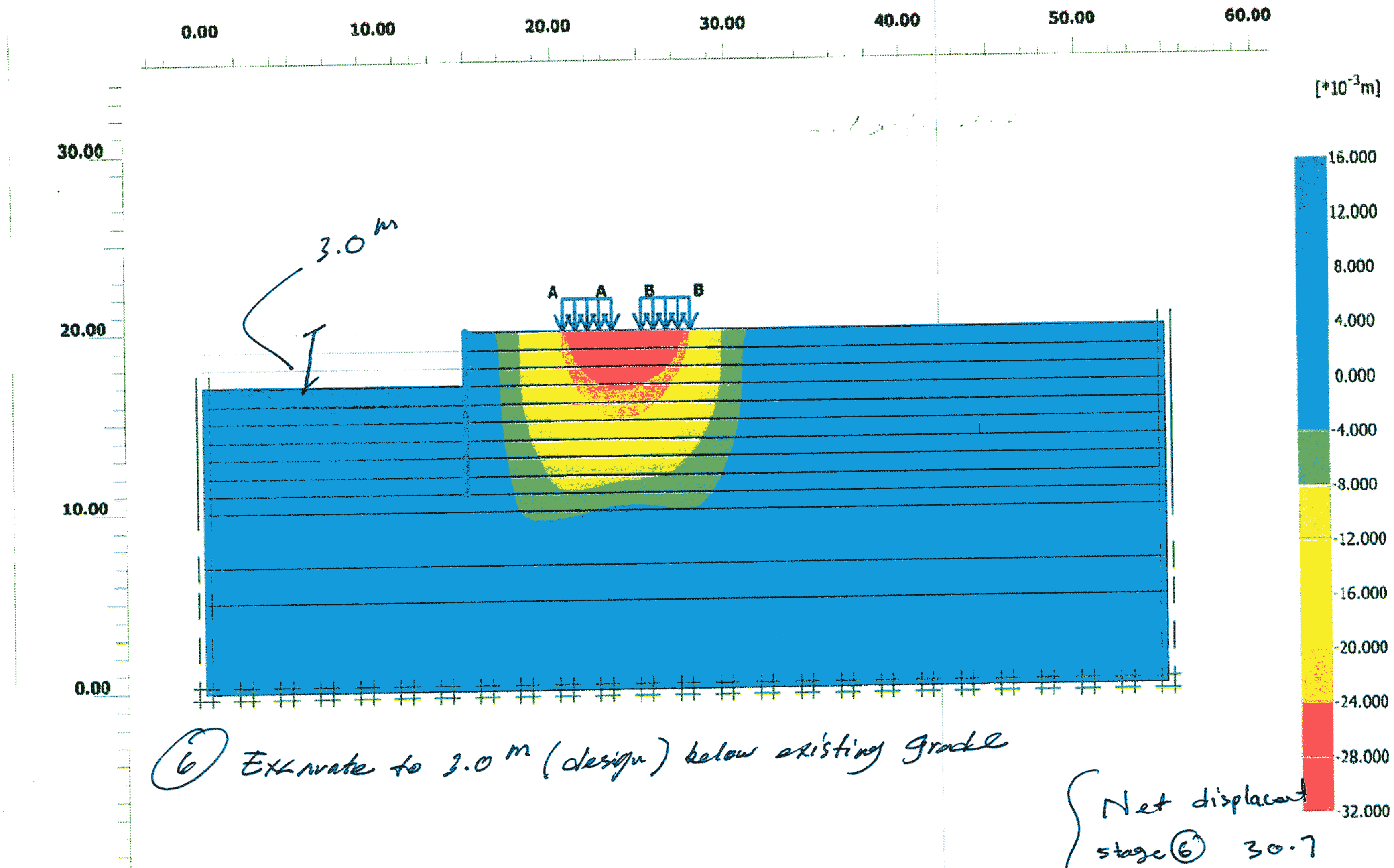
Vertical displacements (Uy)

Extreme Uy $-26.49*10^{-3}$ m = 26.5 mm

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description	Sheetwall 28R	
	Project name Sheetwall 28R	Step 14	Date 04/01/10	User name UMA Engineering Ltd.

Version 8.5.0.1133

Plaxis 8.5



3.0 m

⑥ Excavate to 3.0 m (design) below existing grade

Vertical displacements (Uy)

Extreme Uy -30.72*10^{-3} m = 30.7 mm

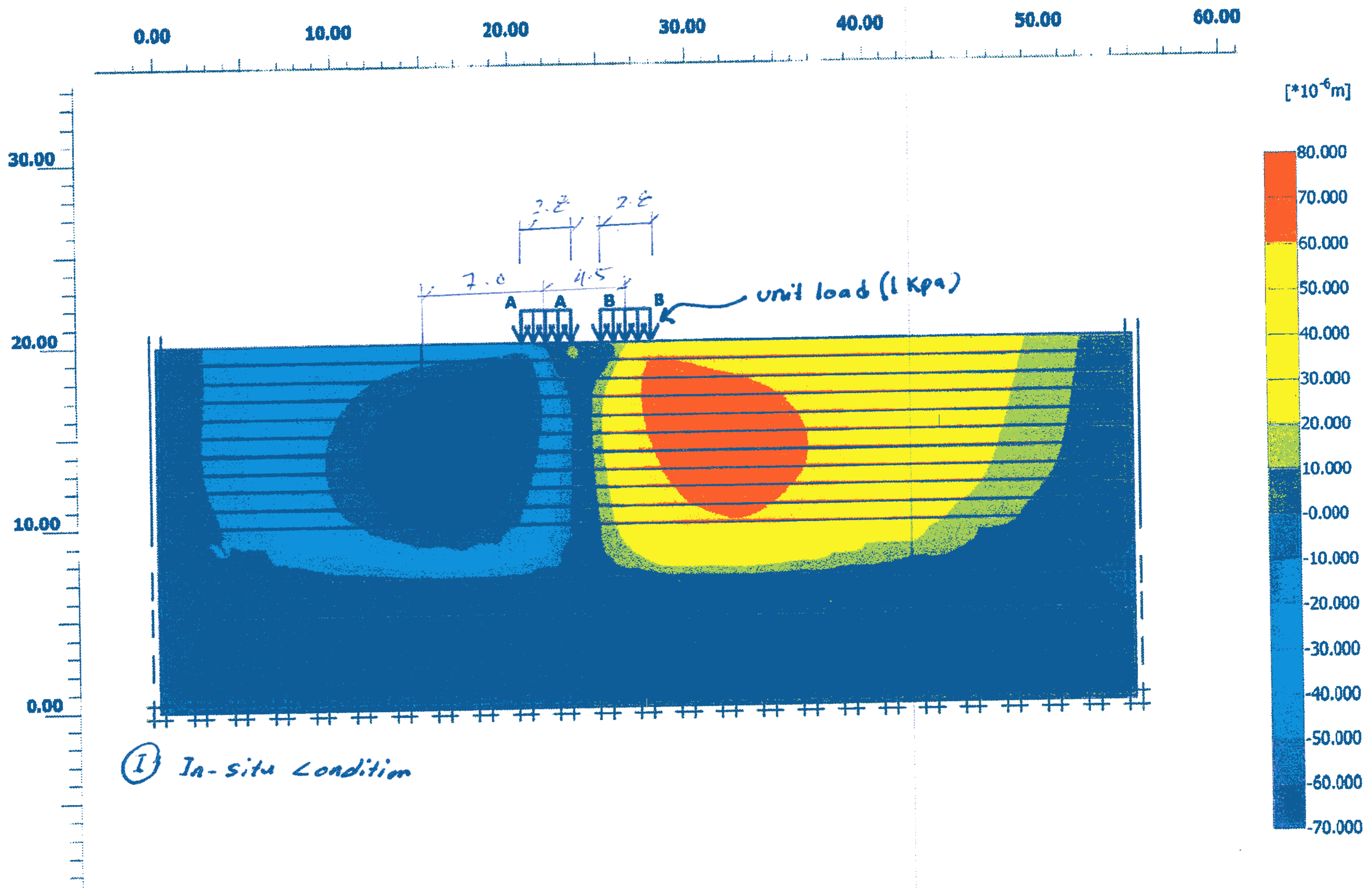
Net displacement
stage ⑥ 30.7
Stage ③ 20.7 −
10.0 mm

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.5.0.1133

Project description
Sheetwall 28R

Project name	*Step*	*Date*	*User name*
Sheetwall 28R	16	04/01/10	UMA Engineering Ltd.

Plaxis 8.5



unit load (1 Kpa)

① In-situ Condition

Horizontal displacements (Ux)

Extreme Ux 78.52*10^{-6} m ≈ 0.0 mm

PLAXIS

Finite Element Code for Soil and Rock Analyses

Version 8.5.0.1133

Project description

Sheetwall 28R

Project name

Sheetwall 28R

Step

21

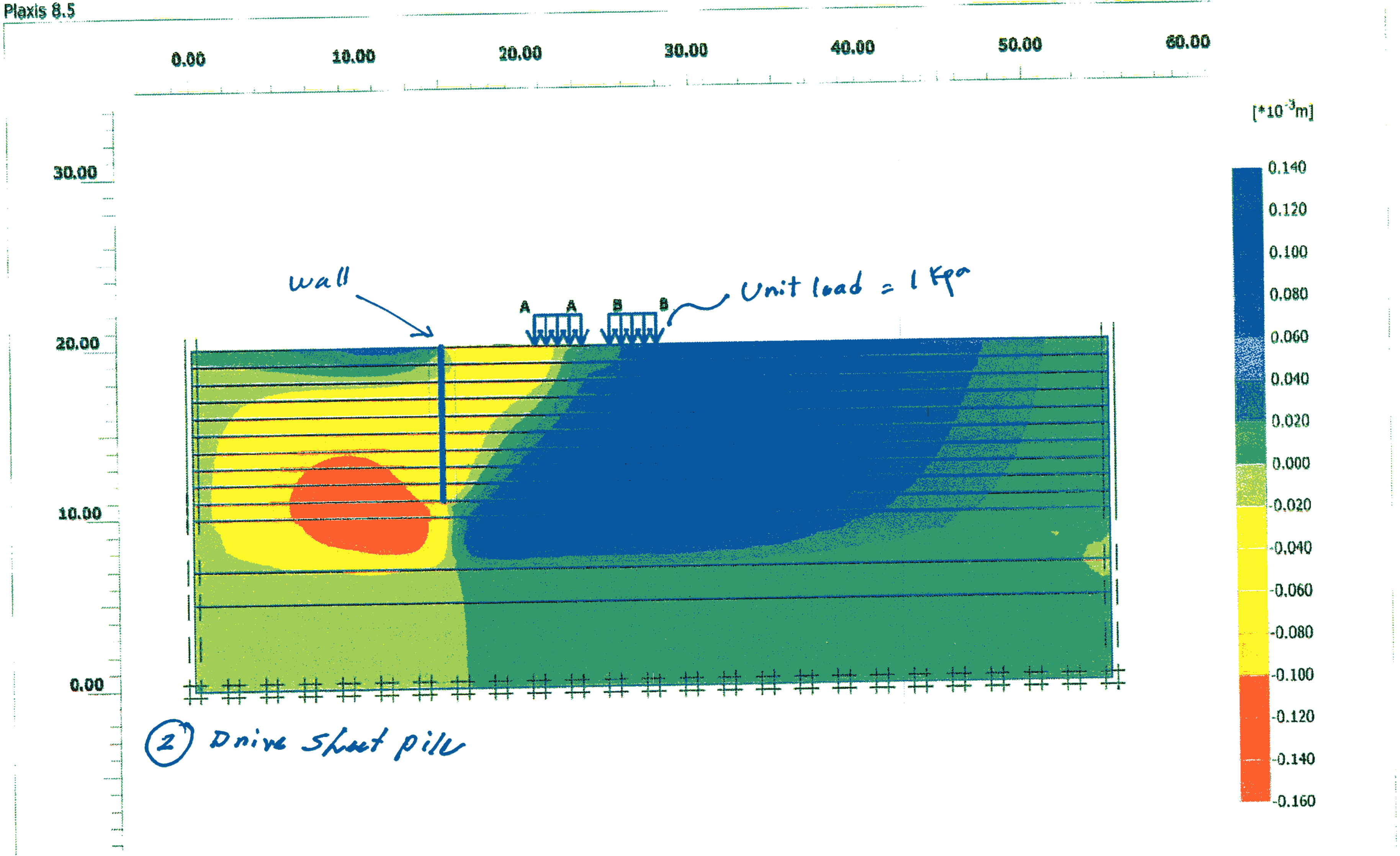
Date

04/01/10

User name

UMA Engineering Ltd.

Plaxis 8.5



wall

Unit load = 1 Kpa

② Drive Sheet pile

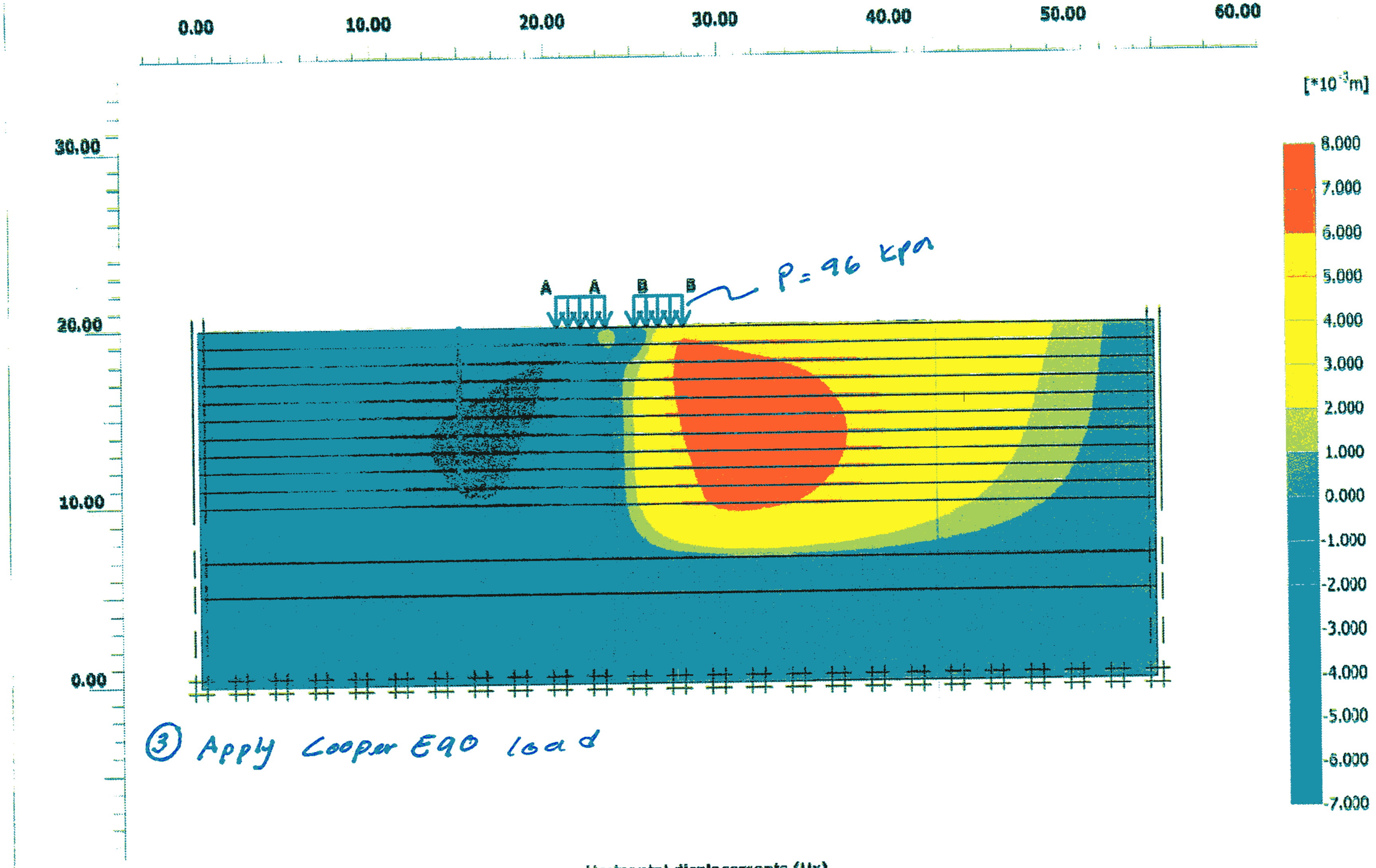
Horizontal displacements (Ux)

Extreme Ux -154.84*10^{-6} m ≈ 0.1 mm

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.5.0.1133

Project description	Sheetwall 28R		
Project name Sheetwall 28R	*Step* 23	*Date* 04/01/10	*User name* UMA Engineering Ltd.

Plaxis 8.5



P = 96 kpa

③ Apply Cooper E90 load

Horizontal displacements (Ux)

Extreme Ux 7.92*10^{-3} m = 7.9 mm

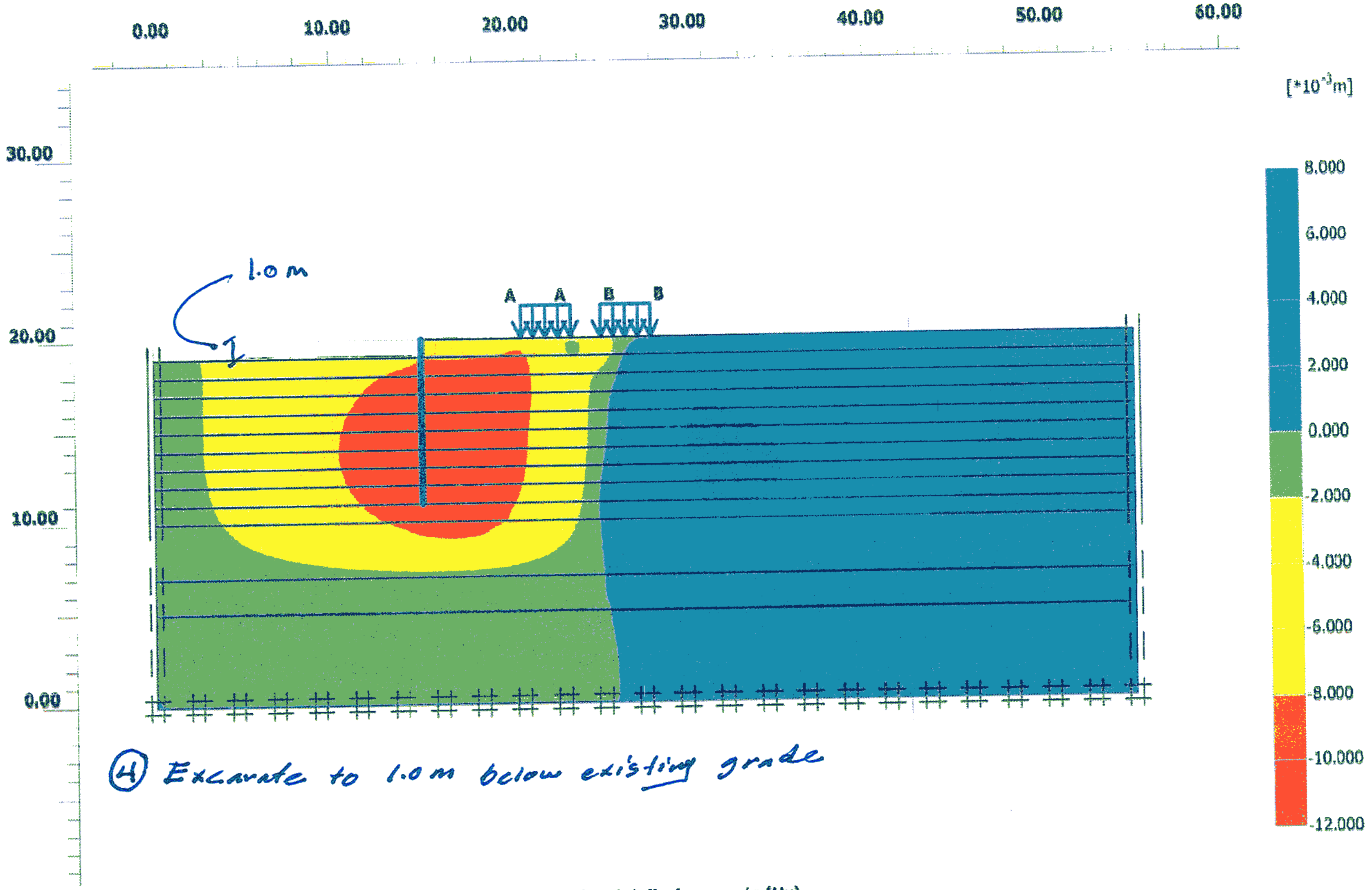
PLAXIS

Finite Element Code for Soil and Rock Analyses

Version 8.5.0.1133

Project description	Sheetwall 28R		
Project name Sheetwall 28R	*Step* 26	*Date* 04/01/10	*User name* UMA Engineering Ltd.

Plaxis 8.5



④ Excavate to 1.0 m below existing grade

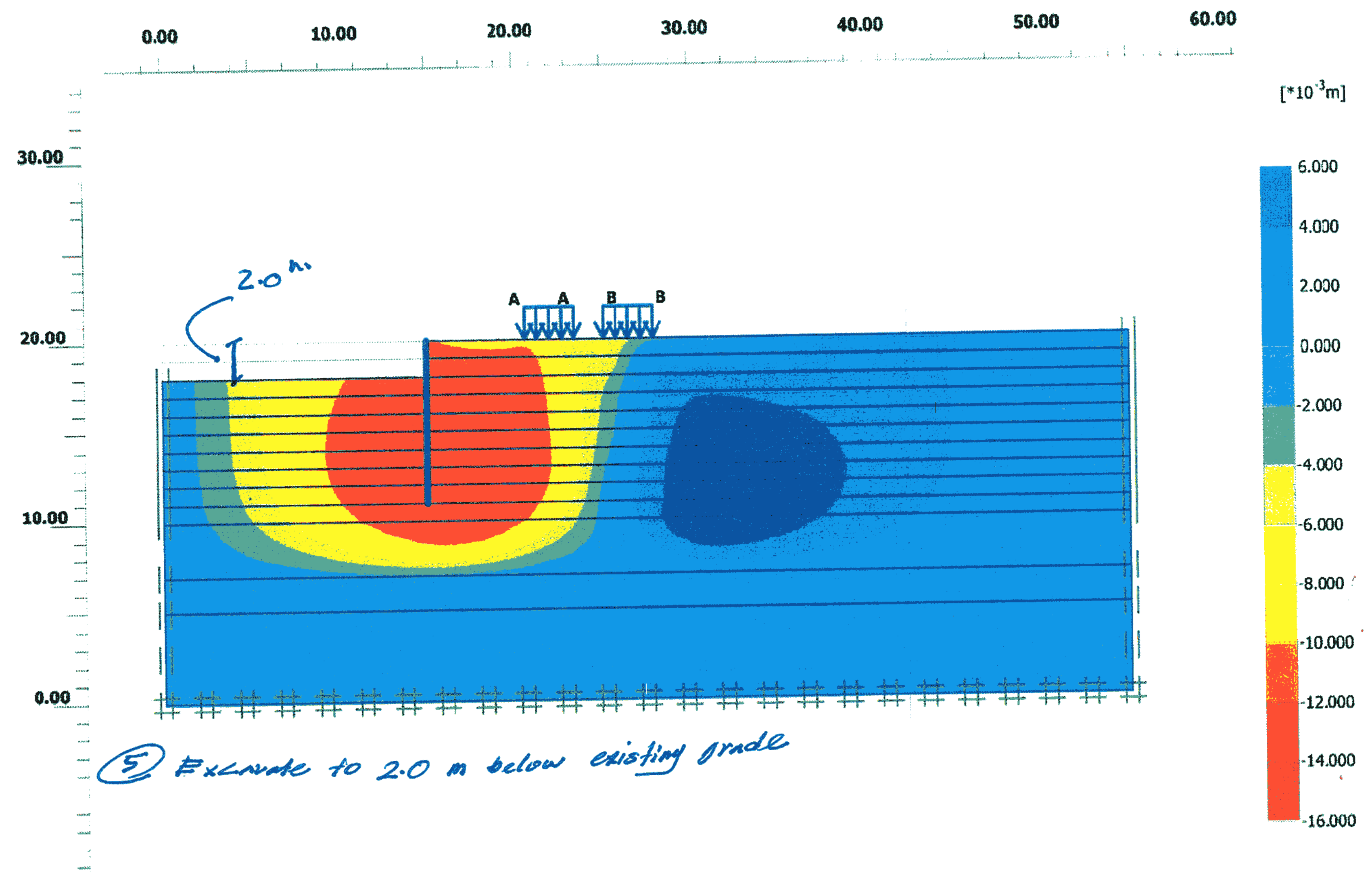
Horizontal displacements (Ux)

Extreme Ux $-10.75*10^{-3}$ m = 10.7 mm

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.5.0.1133

Project description			
Sheetwall 28R			
Project name	Step	Date	User name
Sheetwall 28R	28	04/01/10	UMA Engineering Ltd.

Plaxis 8.5



⑤ Excavate to 2.0 m below existing grade

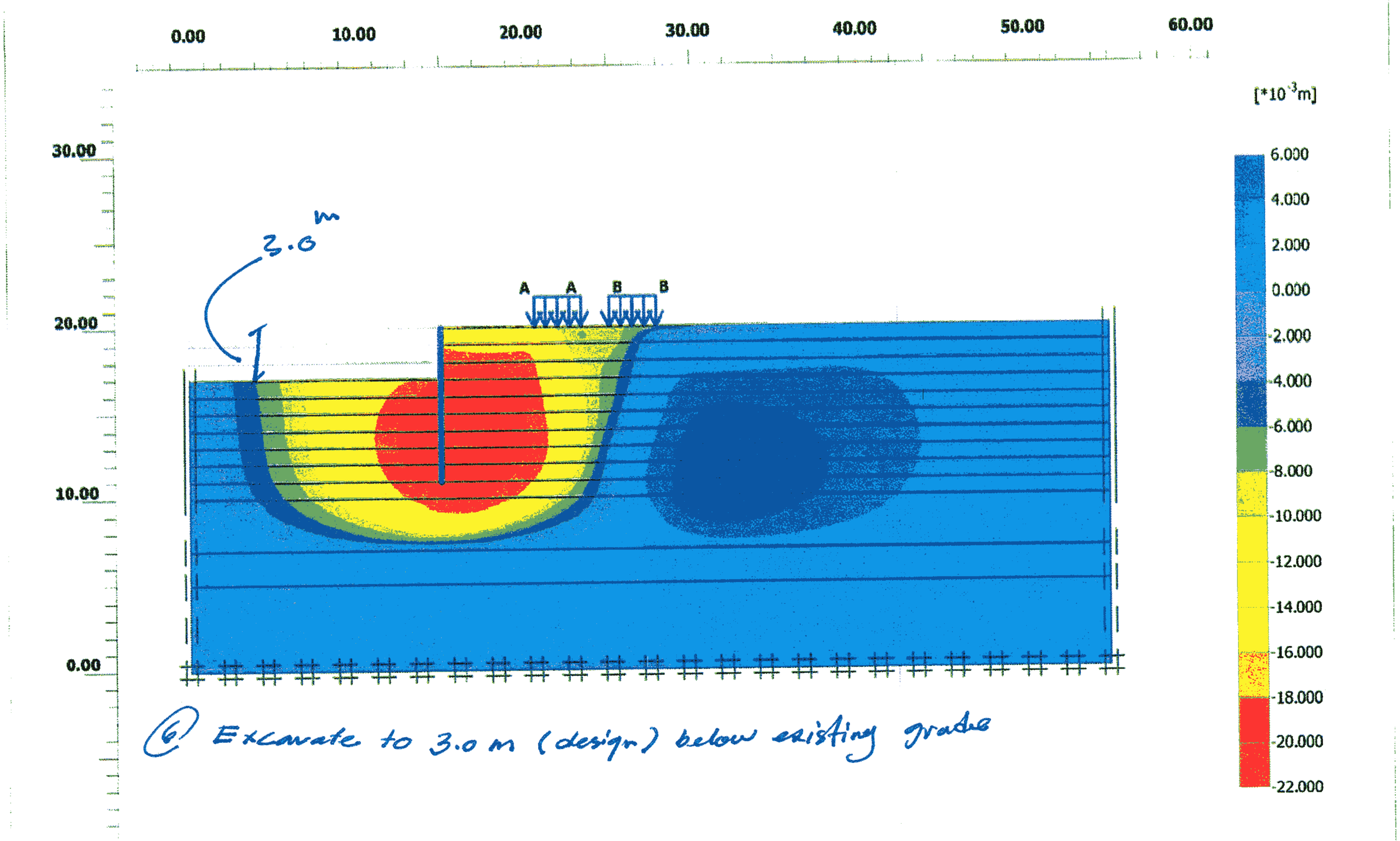
Horizontal displacements (Ux)

Extreme Ux $-15.37*10^{-3}$ m = 15.4 mm

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.5.0.1133

Project description	Sheetwall 28R		
Project name Sheetwall 28R	*Step* 14	*Date* 04/01/10	*User name* UMA Engineering Ltd.

Plaxis 8.5



⑥ Excavate to 3.0 m (design) below existing grade

Horizontal displacements (Ux)

Extreme Ux -20.52*10^{-3} m = 20.5 mm

PLAXIS
Finite Element Code for Soil and Rock Analyses
Version 8.5.0.1133

Project description	Sheetwall 28R		
Project name Sheetwall 28R	*Step* 16	*Date* 04/01/10	*User name* UMA Engineering Ltd.