DYREGROV CONSULTANTS
CONSULTING GEOTECHNICAL ENGINEERS

GEOTECHNICAL REPORT

NORTH END WATER POLLUTION CONTROL CENTRE

DISINFECTION FACILITY

Prepared for

EARTH TECH (CANADA) INC.

on behalf of

THE CITY OF WINNIPEG

December, 2004

Project 242663

GEOTECHNICAL REPORT

NORTH END WATER POLLUTION CONTROL CENTRE

DISINFECTION FACILITY

Prepared for

EARTH TECH (CANADA) INC.

on behalf of

THE CITY OF WINNIPEG

December, 2004

Project 242663

1.0 INTRODUCTION

This report summarizes the results of a geotechnical investigation undertaken by Dyregrov Consultants for the proposed Disinfection Facility at the North End Water Pollution Control Centre. The work was undertaken at the request of Earth Tech (Canada) Inc., on behalf of The City of Winnipeg, and as authorized by the Earth Tech facsimile of October 18, 2004 from their Mr. Eric Hutchinson, P.Eng. The work was done in accordance with our proposal of September 24, 2004.

2.0 PROPOSED DEVELOPMENT

It is our understanding that the proposed facility will be located in the general area between the existing Main Building at the North End Water Pollution Control Centre and Main Street as shown on Figure 1. The proposed Disinfection Facility will parallel the existing 2.286 mm outfall pipe to the Red River and will extend from an existing gate chamber on the west some 80 to 90 metres where the new facility will tie back into the outfall pipe. From the existing gate chamber, a channel bypass will be constructed which will be founded at a depth of about 7.5 metres and will extend to a pump area. The bottom of the pump wells will be about 11 metres below existing grade. The effluent will be passed through a series of U/V channels before returning to the existing outfall pipe. The general founding elevation for the balance of the facility will be some 6.5 metres below grade.

3.0 DESCRIPTION OF THE FIELDWORK

A total of 4 test holes were put down on November 18, 2004 at the locations shown on Figure 1. Truck-mounted caisson drilling equipment (LDH 80) was supplied by Subterranean Ltd. A 400 mm diameter auger was used to advance the borings with three of the test holes being carried to auger refusal and a fourth to 4.88 metres. The soil profile was examined and classified on a continuous basis as the drilling progressed and sampled at regular intervals. Disturbed samples from the auger cuttings and relatively undisturbed samples (three inch diameter Shelby tube samples) were obtained for laboratory strength and moisture content testing.

Observations were made during drilling concerning groundwater, seepage and caving conditions within the borings and the effect these factors may have on foundation selection and design. A temporary steel casing was required to advance the borings through a silt deposit which was encountered in each of the test holes.

All test holes were backfilled with the auger cuttings on completion. Ground elevations at the test holes and their locations were determined by Earth Tech (Canada) Ltd.

A test pit was put down to examine the soil conditions around the existing outfall pipe at the location illustrated on Figure 1. A description of the conditions is included on Figure 6.

4.0 THE SOIL PROFILE

A thick deposit of highly plastic Lake Agassiz silty clay is the predominate component of the soil profile and extends from about the ground surface to depths varying from 19.66 to 20.73 metres or elevations between 210.06 to 211.42 metres. (Existing ground elevations are approximately 230.8 metres.) The clay is common to the Winnipeg area and can be described as firm to stiff in terms of its relative consistency. Moisture contents are generally within the 45 to 55 percent range and are relatively uniform with depth. Plastic and Liquid limits for the clays were determined to be in the order of 65 and 100 percent respectively which would indicate Liquidity Indices in the order of 65 percent.

Undrained shear strengths were determined from unconfined compression, pocket penetrometer and Torvane tests in the laboratory. The results are shown on Figure 7 and indicate that the undrained shear strengths, based on the unconfined compression tests, are basically in the range between 43 and 55 kPa.

Near the upper part of the clay profile, a water-bearing tan silt was noted in each of the test holes at depths ranging from 2.44 to 3.05 metres. The thickness of the silt ranged from 1.06 to 1.37 metres with the bottom being at depths ranging from 3.81 to 4.27 metres. The silt was wet and sloughing which required the use of temporary steel sleeves to cut off the silt which enabled the advancement of the test holes.

The clays are underlain by a glacial silt till at depths between 19.66 and 20.73 metres (elevations between 210.06 and 211.42 metres). The glacial till is known to be a mixture of sand, gravel, cobbles and boulder materials within a predominately silt matrix. At the locations of the test holes, auger refusal was reached between elevations 209.60 and 210.10 metres. The thickness of the glacial till varied from 0.46 to 1.82 metres. The action of the drill suggested that the auger refusal could be on bedrock. The consistency of the glacial till was visually classified as soft and was confirmed by moisture contents in excess of 10 percent.

At the location of the test pit, which was carried to the spring line on one side, a hand-augered hole was drilled beside and beneath the existing outfall pipe. The top of the pipe was at a depth of 2.29 metres (elevation 228.29 metres). Immediately above the pipe was a silt and clay fill which appeared to be well compacted. Its lateral limits were vertical which would suggest that it was placed within a wide trench or within a shored excavation. The hand-augered test hole indicated the presence of the silt below the spring line which in turn was underlain by the silty clay. Some seepage was noted from the silt.

5.0 GROUNDWATER CONDITIONS

A perched groundwater table is evident in the tan silt which is within the upper four metres of the soil profile. When auger refusal was reached in the glacial till/bedrock, the water level rose to a depth of 6.78 metres (elevation 224.01 metres) in Test Hole 1, a trace of water noted in both Test Holes 2 and 3. The water level of 224.01 metres is consistent with the piezometric conditions in the underlying bedrock.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

It is our understanding that the proposed development will include a connection to an existing gate chamber on the existing outfall pipe to the Red River. The connection will connect to a bypass channel that will parallel the existing outfall pipe which will connect to a pump well for transfer into the Disinfection Facility structure. The treated effluent will then be connected back to the existing outfall pipe by a new tie-in chamber. The bypass channels and treatment building structure will be structurally supported on a pile foundation system. Deep excavations are required throughout the facility.

6.2 Foundations

The two principal foundation options for the support of the structural aspects of the project are driven precast prestressed end-bearing concrete piles and cast-in-place concrete friction piles. The preferred foundation alternative is the driven precast concrete piles which would be end bearing in the underlying glacial till. However, actions will have to be taken to minimize the impacts of vibrations induced by the pile driving operations.

Driven precast concrete piles have been used extensively at the NEWPCC and are considered appropriate for this project if the loads can be distributed to take full advantage of the relatively high capacities of these piles. These piles, if driven to practical refusal, may be assigned conventional supporting capacities of 445, 625 and 800 kN for nominal 300, 350 and 400 mm sizes respectively. The piles should be driven with a diesel hammer with a rated energy of not less than 40 kilojoules. Practical refusal may be defined as final penetration resistance sets of 5, 8 and 12 blows per less than 25 mm for the 300, 350 and 400 mm sizes respectively. At least three sets should be obtained. If followers are used, the final penetration resistance criteria should be increased by 50 percent. No reduction in individual pile capacity is necessary for reasons related to group action provided that pile heave is monitored, measures undertaken to minimize it (by preboring) and redriving is done as necessary in pile groups. Pile spacing should not be less than 2.5 pile diameters centre to centre. Pile concrete should be at least 7 days old.

Inspections of the driven pile installation should be undertaken by technologists experienced with their installation. The lack of large thicknesses of the glacial till and the presence of cobbles and boulders may result in pile installation problems which should be monitored.

Preboring should be done at the driven pile locations with diameters that are 50 mm larger than the pile size. The preboring is effective in reducing ground vibrations and pile heave and contributes positively to pile verticality. When driving within 3 metres of existing underground facilities, deeper prebore to within 1.5 metres of the glacial till (approximately elevation 213.0

metres) should be considered. If followers are required for driving the piles, the size of the prebore should be 50 mm larger than the follower and for a depth equal to the length of the follower.

It is understood that pile loads may be suitable for the use of the cast-in-place concrete friction piles. These piles should have a minimum diameter of 400 mm and may be sized on the basis of an allowable shaft adhesion of 16.7 kPa. The upper 5 feet of shaft support should be discounted and the piles should not penetrate the glacial silt till to avoid problems with the groundwater conditions which exist in the underlying bedrock, such as was encountered in Test Hole 1. In this regard, it is recommended that the pile tips should not extend closer than 1.5 metres to the glacial till surface or approximately 213.0 metres. Pile spacing should not be closer than 3 pile diameters centre to centre. If pile groups are required, group action should be considered. Temporary steel sleeves should be on hand and used on an as-required basis to prevent seepage and caving into the borings, particularly from the water-bearing silt.

The friction piles potentially subject to frost heave and uplift should contain full-length reinforcement and should be a minimum length of 7.6 metres. Alternatively, the piles could be protected by the use of flat-lying, rigid, high-density insulation around the pile at least 300 mm below the finished grade.

It is understood that a number of piles may be installed in areas where significant amounts of fill may be placed. Conventional down-drag forces on these piles are not of any consequence as fill will only be carried up to near the original grade such that the stresses in the underlying clay will not be significantly different than the original stresses with the result that consolidation of the clay will not occur. The self-consolidation of the fill around the piles is not expected to transmit any consequential loads to the pile due to the relatively loose condition that the fill will be placed around the piles.

6.3 Slabs

It is understood that structurally supported floor slabs will be used throughout. The floors (and grade beams) should be separated from the underlying soil subgrade by a 300 mm void. It is presumed that these slabs will have no underdrainage and that water could collect below them. This is conducive to swelling and generous allowance for this is recommended.

6.4 Excavations

Excavations are required throughout the project, some of which are quite deep, as well as adjacent to existing underground facilities such as the 2.286 metre diameter existing outfall to the Red River. The deep excavations will have to be shored or will require relatively flat excavation slopes. These slopes may require unloading of the overburden above the existing outfall to achieve satisfactory safety factors for the temporary slopes. Excavated materials should not be stockpiled immediately adjacent to the work as their presence may negatively impact the stability of the excavation slopes, shoring or the underground facilities.

The design of the excavation slopes should recognize the presence of the water-bearing silt which was noted in the test holes. The bottom of the silt was below the top of the existing outfall pipe. It may be necessary to control seepage from the silt during construction.

The excavated slopes should be protected from weathering by suitable temporary coverings.

Temporary shoring may be designed on the basis of the earth pressure distribution illustrated in Figure 8. Ground movements behind the shoring will occur and it is largely unavoidable. The amount that will occur cannot be predicted with much accuracy, mainly because the movement is as much a function of excavation procedures and workmanship as it is a function of theoretical considerations. The impact of these movements should be assessed.

It is recommended that toe support for soldier piles be provided by concrete plugs within the clay deposit immediately below the excavation surface. It is recommended that the toe support not be provided from driving the soldier piles and/or sheet piles into the underlying glacial till/bedrock. This will minimize the potential for a long-term groundwater connection between the bedrock aquifer and the proposed facility.

Where shoring is provided at the base of any excavated slopes, the effects of sloping ground above the shoring, on the shoring, must be considered.

6.5 Below-Grade Walls

The below-grade walls should be designed to resist lateral earth pressures that are derived on the basis of the following conventional relationship:

$$P = K \gamma D$$

where P = lateral earth pressure at depth below final grade D (kN/m^2)
K = earth pressure coefficient (0.5)
γ = soil backfill unit weight (17.5 kN/m^3)
D = depth from final grade to point of pressure calculation (m)

The base of the wall should be provided with a filter-protected positive drainage system to prevent the buildup of hydrostatic pressure against the wall. Where drainage is not provided, the lateral pressure should be increased by 9.81 kN/m^3. An allowance for surface live loads should be included if significant load is applied within a distance from the wall equal to the height of the wall. The lateral pressure due to the live load should be presumed equal to 50 percent of the vertical pressure due to the live load.

The selection of backfill materials should be reviewed during the design and their impact on the foregoing pressures assessed.

6.6 Backfill Over Structures

The backfill over structures can be undertaken with the clayey materials from the excavations. These materials should receive nominal compaction to about 90 percent of Standard Proctor Density. Due to the extensive areas of backfill, compaction equipment will have to be used. A unit weight of about 17.5 kN/m^3 can be used for the clayey materials for design of the roofs of the structures. Also, the loads induced by the compaction equipment on the roofs should be checked. If materials, other than the clayey materials are used, the design unit weights should be increased.

6.7 Pavements

It is recommended that for the relocation of the existing driveway and access to the facility, the pavement section should consist of 75 mm of asphaltic concrete placed on 380 mm of crushed granular base course or an equivalent section. Some consideration could be given to using 200 mm of reinforced concrete on 75 mm of a crushed granular base course service area adjacent to the facility.

The pavement sections should be placed on a prepared subgrade which should be compacted to a uniform density of at least 95 percent of Standard Proctor density at optimum moisture content. The subgrade should be "proof rolled" and any soft spots should be removed and replaced with suitable materials and compacted to this standard.

Although silt was encountered in all of the test holes, it is not expected that it will affect the subgrade preparation because it is relatively deep. It may, however, generate some frost heave in particularly cold winters.

6.8 Other

All concrete in contact with the soil should be manufactured with sulphate-resistant cement and should be of high quality.

Site drainage should be away from the facility site at a gradient of at least 2 percent.

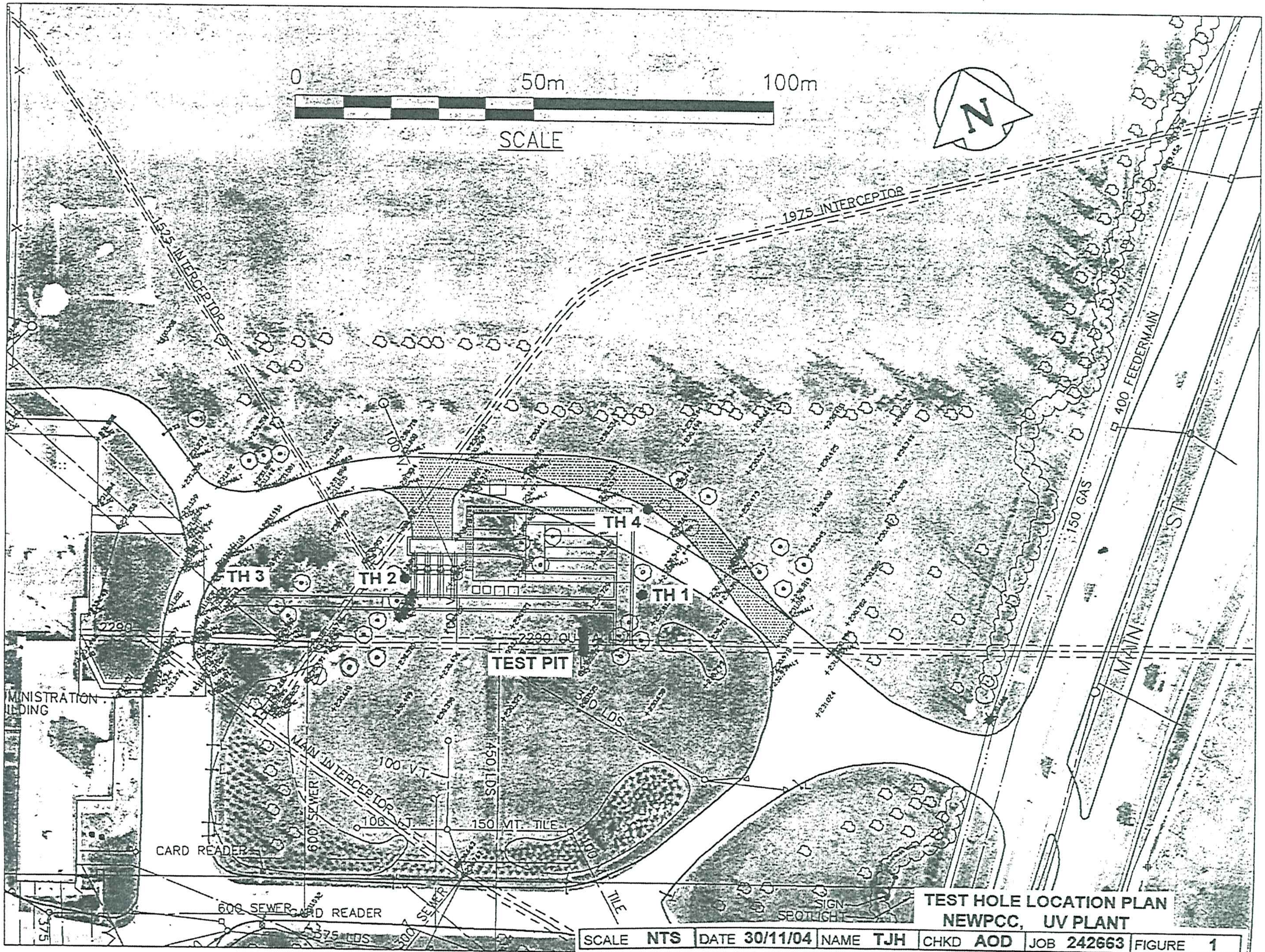
Respectfully submitted,

DYREGROV CONSULTANTS

PROVINCE OF MANITOBA
A. O.
DYREGROV
REGISTERED PROFESSIONAL ENGINEER
21/12/08

Per:

A.O. Dyregrov, P.Eng.



0
50m
100m
SCALE
N
1975 INTERCEPTOR
1525 INTERCEPTOR
400 FEEDERMAIN
150 GAS
MAIN ST
TH 1
TH 2
TH 3
TH 4
TEST PIT
2290 OU
2290
100 V.
300 LDS
450 LDS
100 V.T.
MAIN INTERCEPTOR
600 SEWER
100
150 VIT. TILE
TILE
CARD READER
600 SEWER
CARD READER
375 LDS
375
SEWER
SIGN
SPOTLIGHT
MINISTRATION
ILDING
TEST HOLE LOCATION PLAN
NEWPCC, UV PLANT
SCALE NTS
DATE 30/11/04
NAME TJH
CHKD AOD
JOB 242663
FIGURE 1

CONSULTANTS	Logged/Dwn.: TH	Test Hole No.	Project No.
	Checked: AOD	1	242663
PROJECT: NEWPCC		DATE OF INVEST.	NOVEMBER 18, 2004
CLIENT: EARTH TECH		DRILL :	SUBTERRANEAN, 16 INCH AUGER

SAMPLE NO.	DEPTH (m)	ELEV. (m)	SOIL DESCRIPTION
	0.00	230.79	0.00-0.15 SOD OVER TOPSOIL
1S1	0.75	230.04	0.15-0.90- CLAY, BLACK, MOIST, ORGANIC 0.90-2.59 CLAY BROWN, SILTY, STIFF, BLOCKY TO 1.5
1S2	1.50	229.29	
1S3	2.25	228.54	
1S4	3.00	227.79	2.59-3.81 SILT TAN, MOIST, MOIST TO WET AT 3.1, SLOUGHING, WATER
1S5	3.75	227.04	3.81-20.73 CLAY BROWN, SILTY, MOTTLED, STIFF, HIGH PLASTIC
	4.50	226.29	Qu-92.3 KPa Pp-95.8 KPa Tv-70.9 KPa W -16.67 KN/M
1S6 1T7	5.25	225.54	
1S8	6.00	224.79	AT 6.4 GREY, SILTY,
	6.75	224.04	
1T9	7.50	223.29	Qu-106.6 KPa Pp-86.1 KPa Tv-67.0 KPa W -16.69 KN/M
	8.25	222.54	
1S10	9.00	221.79	
	9.75	221.04	
	10.50	220.29	Qu-96.7 KPa Pp-71.8 KPa Tv-57.5 KPa W -17.07 KN/M
1T11	11.25	219.54	
1S12	12.00	218.79	
	12.75	218.04	
	13.50	217.29	VERY TILLY BETWEEN 13.71 AND 16.77 Qu-107.1 KPa Pp-76.6 KPa Tv-51.7 KPa W -17.14 KN/M
1T13	14.25	216.54	
	15.00	215.79	
1S14	15.75	215.04	Qu-90.1 KPa Pp-64.6 KPa Tv-53.6 KPa W -19.75 KN/M
	16.50	214.29	MORE HOMOGENEOUS BETWEEN 13.77 AND 19.50
	17.25	213.54	
1T15	18.00	212.79	
1S16	18.75	212.04	
	19.50	211.29	AT 19.50 MANY TILL INCLUSIONS
1S17 1S18	20.25	210.54	
	21.00	209.79	20.73- 21.19 GLACIAL SILT TILL
	21.75	209.04	
	22.50	208.29	END OF TEST HOLE AT 21.19 ON POSSIBLE BEDROCK
	23.25	207.54	NOTE: WATER ROSE QUICKLY IN FIVE MINUTES TO 6.78 BELOW GROUD SURFACE

MOISTURE CONTENT (%)

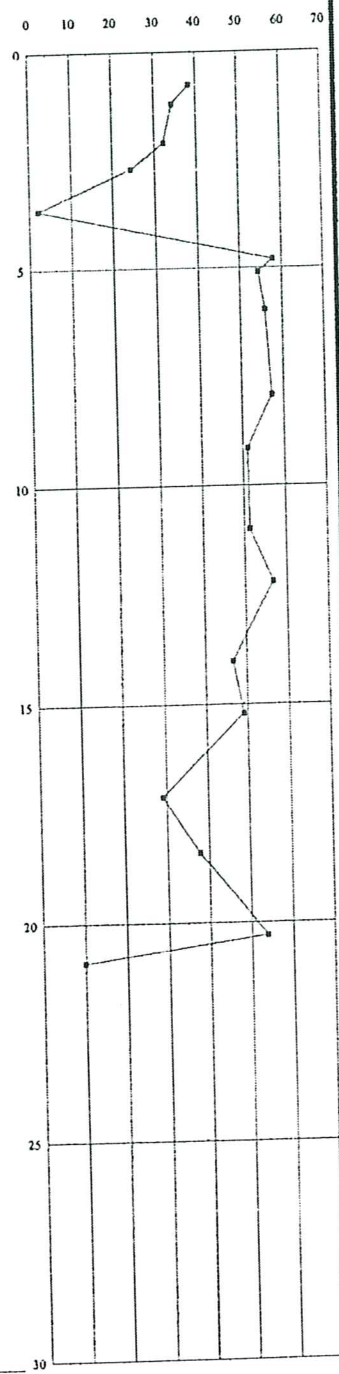


FIGURE 2

DYREGROV CONSULTANTS	Logged/Dwn.: TH Checked: AOD	Test Hole No. 2	Project No. 242663
PROJECT: NEWPCC		DATE OF INVEST.	NOVEMBER 18, 2004
CLIENT: EARTH TECH		DRILL :	SUBTERRANEAN, 16 INCH AUGER

SAMPLE NO.	DEPTH (m)	ELEV. (m)
	0.00	230.83
	0.75	230.08
2S1	1.50	229.33
2S2	2.25	228.58
2S3 2S4	3.00	227.83
	3.75	227.08
2T5	4.50	226.33
	5.25	225.58
	6.00	224.83
2T6	6.75	224.08
2S7	7.50	223.33
	8.25	222.58
	9.00	221.83
2T8	9.75	221.08
2S9	10.50	220.33
	11.25	219.58
	12.00	218.83
2T10	12.75	218.08
	13.50	217.33
1S11	14.25	216.58
	15.00	215.83
2T12	15.75	215.08
	16.50	214.33
	17.25	213.58
2S13	18.00	212.83
2T14	18.75	212.08
2S15	19.50	211.33
2S16	20.25	210.58
	21.00	209.83
	21.75	209.08

SYM

SOIL DESCRIPTION

0.00-0.15 SOD OVER TOPSOIL
0.15-0.76 CLAY, BLACK, ORGANIC, FIRM
0.76-2.44 CLAY
BROWN, SILTY, STIFF, WEATHERED

2.44-3.81 SILT
TAN TO LIGHT BROWN, MOIST, TRACE FINE SAND
MOIST TO WET AT 3.1, SLOUGHING, WATER

3.81-19.82 CLAY
BROWN GREY, SILTY, MOTTLED, TRACE SILT INCLUISIONS
FIRM, MEDIUM TO HIGH PLASTIC

Qu-80.0 KPa
Pp-95.8 KPa
Tv-77.6 KPa
W -16.74 KN/M

AT 6.4 GREY CLAY, TRACE SILT INCLUISIONS

Qu-95.7 KPa
Pp-71.8 KPa
Tv-68.9 KPa
W -16.83

Qu-10.1 KPa
PP-67.0 KPa
Tv-56.5 KPa
W -17.12

TRACE OF TILL POCKETS BELOW 12.2

Qu-99.1 KPa
Pp-87.0 KPa
Tv-47.9 KPA
W -16.42 KN/M

Qu-97.6 KPa
Pp-47.9 KPa
Tv-43.0 KPa
W -16.88 KN/M

19.82-20.73 GLACIAL SILT TILL
SILT MATRIX WITH SOME PEBBLES AND STONES,
TAN

END OF TEST HOLE AT 20.73 IN GLACIAL SILT TILL

NOTE: TRACE OF WATER AT COMPLETEION OF DRILLING

MOISTURE CONTENT (%)

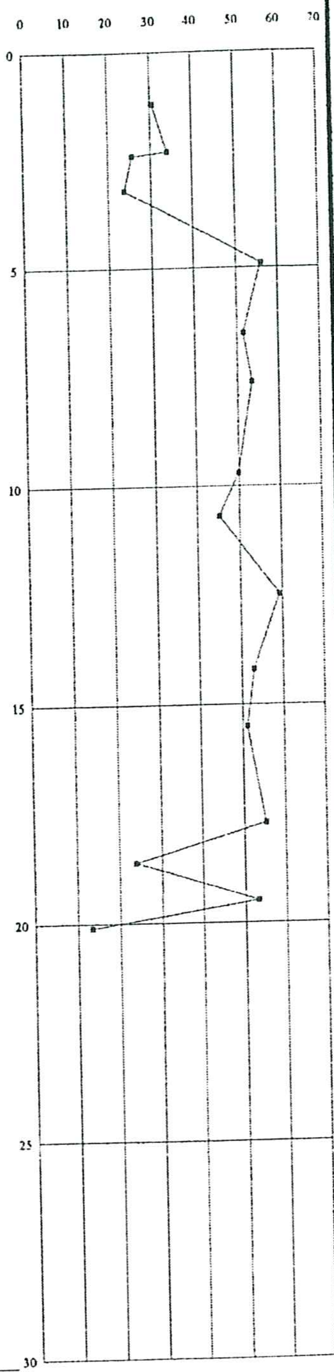


FIGURE 3

CONSULTANTS	Logged/Dwn.: TH Checked: AOD	Test Hole No. 3	Project No. 242663
PROJECT: NEWPCC CLIENT: EARTH TECH		DATE OF INVEST. NOVEMBER 18, 2004 DRILL : SUBTERRANEAN, 16 INCH AUGER	

SAMPLE NO.	DEPTH (m)	ELEV. (m)	SYM	SOIL DESCRIPTION	
	0.00	231.08		0.00-0.15 SOD OVER TOPSOIL	
	0.75	230.33		0.15-2.44 FILL, TO 0.61 CLAY MATRIX WITH POCKET OF PIT RUN	
3S1	1.50	229.58		AT 1.8 SAME WITH TRACE BLACK POCKET, TRACE STONES	
3S2	2.25	228.83		2.44-2.90 CLAY, BROWN, SILTY, STIFF	
3S3	3.00	228.08		2.90-3.96 SILT TAN, MOIST TO WET AT 3.1, SLOUGHING, WATER	
3S4 3S5 3S6 3T7	3.75	227.33			
	4.50	226.58		3.96-19.66 CLAY BROWN, SILTY/GREY, MOTTLED, TRACE TAN SILT INCLUSIONS, MEDIUM TO STIFF, MEDIUM TO HIGH PLASTI VERTICAL FISSURE FILLED WITH GREY SILT	Qu-79.8 KPa Pp-129 3 KPa Tv-119.7 KPa W-17.23 KN/M
	5.25	225.83			
3S8	6.00	225.08			
	6.75	224.33			
	7.50	223.58		GREY, STIFF, TRACE SILT AND FINE SAND INCLUSIONS	Qu-93.7 KPa Pp-95.7 KPa Tv-70.9 KPa W-16.88 KN/M
3T9	8.25	222.83			
3S10	9.00	222.08			
	9.75	221.33			
	10.50	220.58		SAME, MEDIUM TO STIFF, PLUS TRACE FINE GRAVEL INCLUSIONS	Qu-84.7 KPa Pp-95.7 KPa Tv-67.0 KPa W-18.1 KN/M
3T11	11.25	219.83			
	12.00	219.08			
3S12	12.75	218.33			
	13.50	217.58			
3T13	14.25	216.83		SAME, SOME FINE SILT AND FINE SAND LAYERING	Qu-91.7 KPa Pp-47.9 KPa Tv-47.9 KPa W-17.97 KN/M
	15.00	216.08			
3S14	15.75	215.33			
	16.50	214.58		SAME, MEDIUM, TRACE SILT, FINE SAND, AND FINE GRAVEL INCLUSIONS	Qu-77.9 KPa Pp-35.9 KPa Tv-38.3 KPa W-17.05 KN/M
3T15	17.25	213.83			
3S16	18.00	213.08			
	18.75	212.33			
3S17	19.50	211.58		19.66-21.48 GLACIAL SILT TILL SILT MATRIX WITH SAND, TRACE CLAY, SOME GRAVEL SOME COBBLES AND BOULDERS	
3S18	20.25	210.83			
	21.00	210.08			
	21.75	209.33		END OF TEST HOLE AT 21.48 AUGER REFUSAL ON ASSUMED BEDROCK NOTE: APPROXIMATELY 25mm OF WATER IN HOLE AT OMPLETION OF DRILLING	

MOISTURE CONTENT (%)

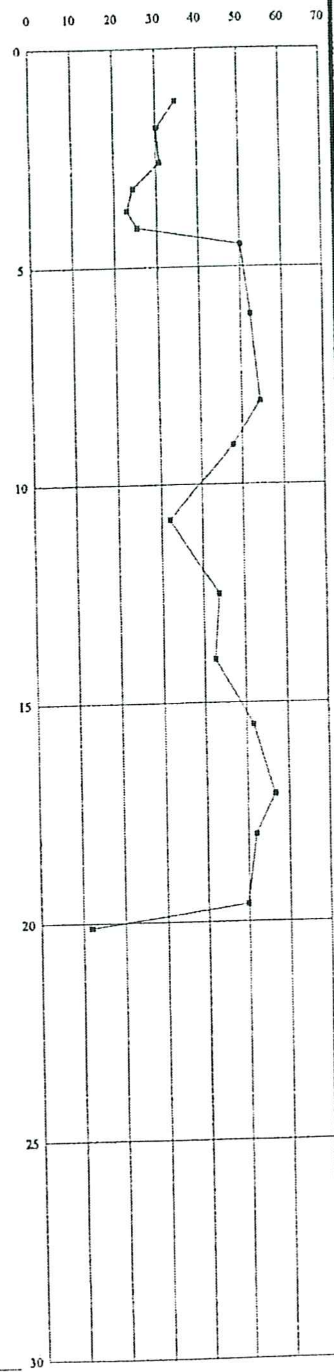


FIGURE 4

DYREGROV CONSULTANTS	Logged/Dwn.: TH Checked: AOD	Test Hole No. 4	Project No. 242663
PROJECT: NEWPCC		DATE OF INVEST.	NOVEMBER 18, 2004
CLIENT: EARTH TECH		DRILL :	SUBTERRANEAN, 16 INCH AUGER

SAMPLE NO.	DEPTH (m)	ELEV. (m)	SYM
	0.00	230.66	
	0.75	229.91	
	1.50	229.16	
	2.25	228.41	
	3.00	227.66	
	3.75	226.91	
	4.50	226.16	
	5.25	225.41	
	6.00	224.66	

SOIL DESCRIPTION

0.00-0.15 SOD OVER TOPSOIL
0.15-0.53 CLAY, BLACK, ORGANIC, MOIST, FIRM, LOW PLASTIC,
0.53-3.05 CLAY
BROWN, SILTY, TRACE SILT INCLUSIONS TO 1.5
SOME SILT VARVES TO 3.0, BLOCKY, WEATHERED,
FIRM, MEDIUM TO HIGH PLASTIC

3.05-4.27 SILT
TAN TO LIGHT BROWN, MOIST TO WET BELOW 3.3
SOFT, VERY LOW TO NON PLASTIC, SLOUGHING AND WATER BELOW 3.3
4.27-4.88 CLAY
BROWN, SILTY, MOTTLED, TRACE TINY SILT AND
FINE SAND INCLUSIOND

END OF TEST HOLE AT 4.88 IN BROWN SILTY CLAY

MOISTURE CONTENT (%)

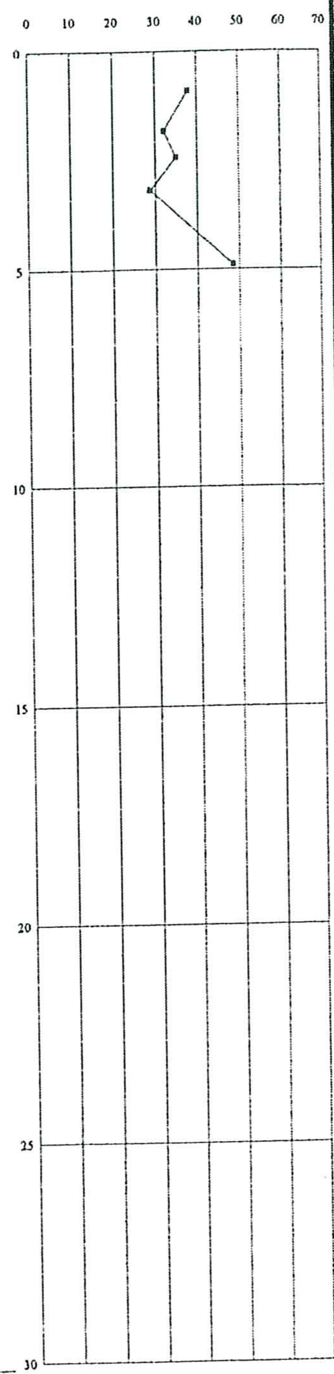
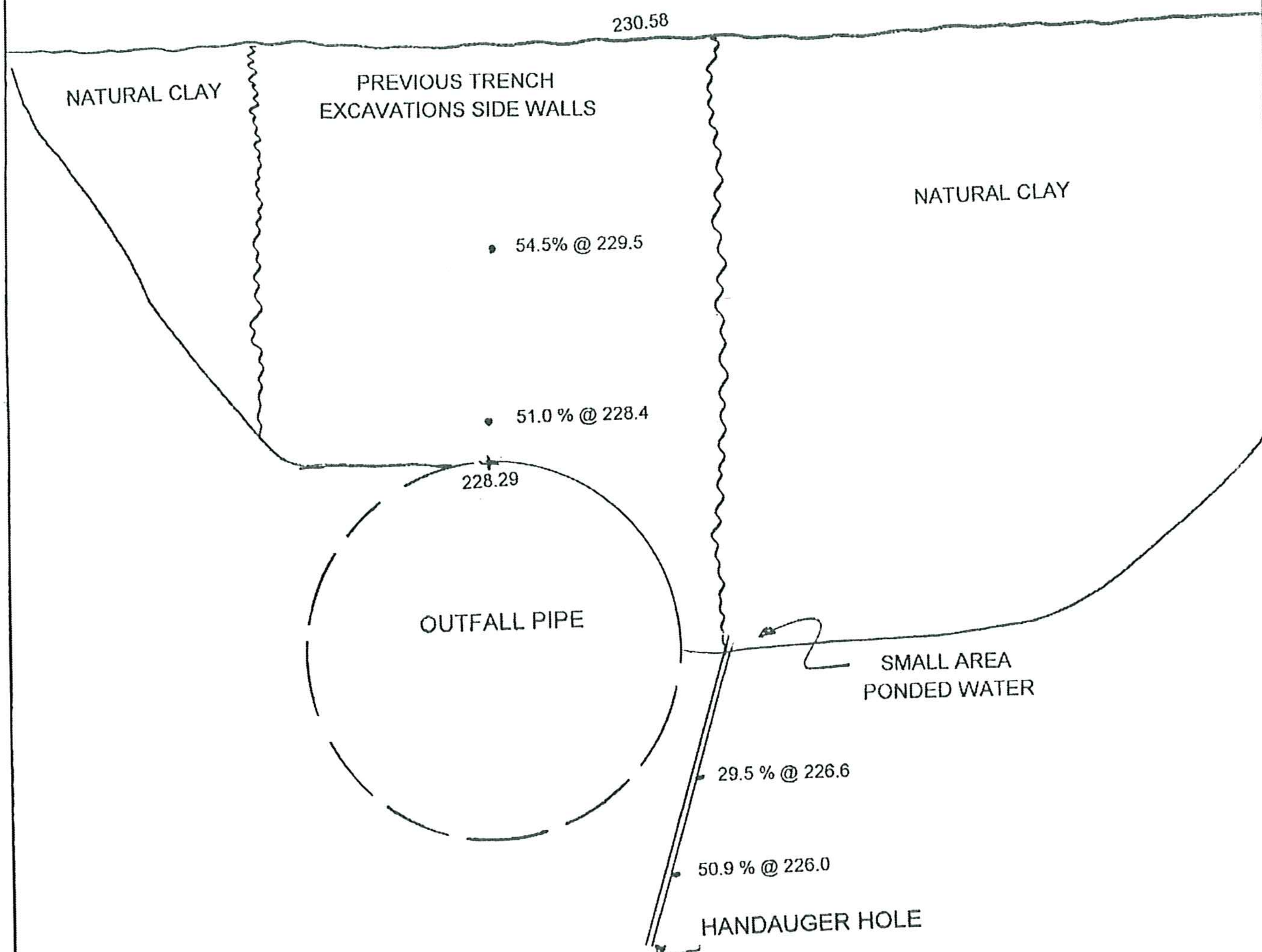


FIGURE 5



NOTE: The small area of poned water came from the silt and from around the pipe

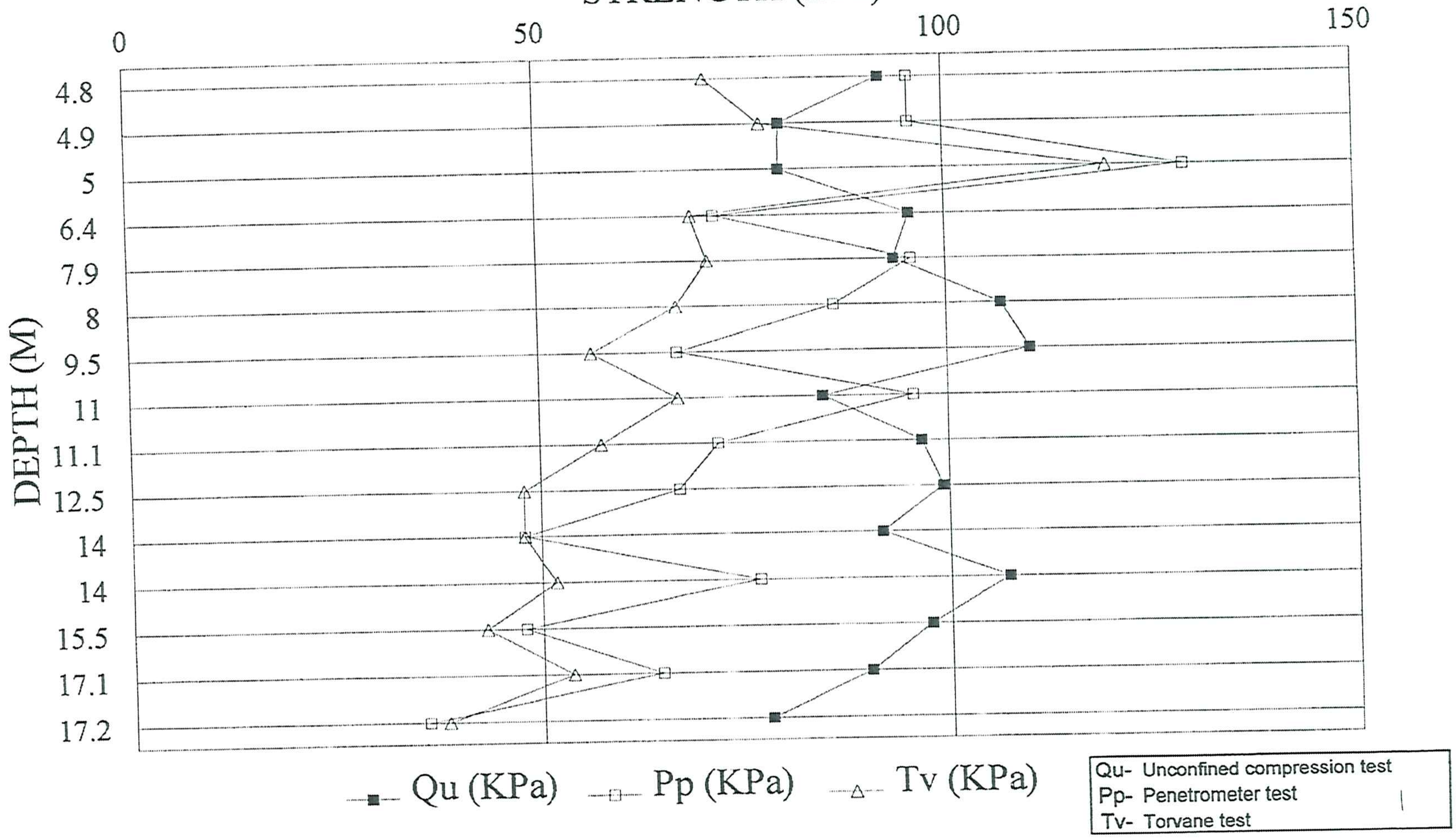
The outfall pipe was exposed with a large backhoe to approximately 300 above the pipe and the remainder of the fill was removed by shovell .

The outfall pipe was exposed only to the string line.

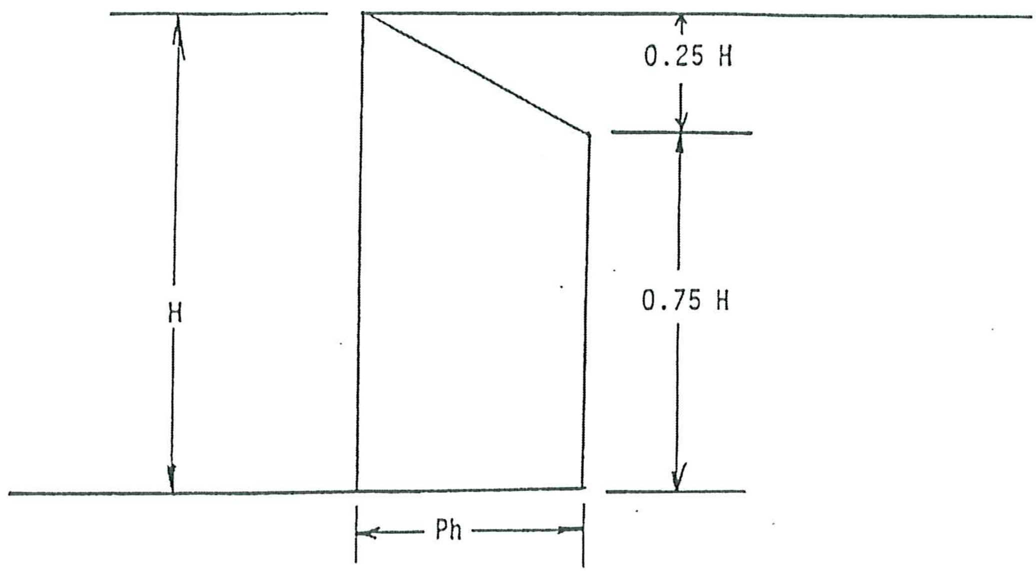
The previous excavated trench was +/- 3.0 m wide

DYREGROV CONSULTANTS CONSULTING GEOTECHNICAL ENGINEERS			TEST PIT EXPOSING OUTFALL PIPE NEWPCC, PROPOSED U V PLANT		
SCALE NTS	DATE 07/12/04	MADE TJH	CHKD AOD	JOB 242663	FIGURE 6

UNDRAINED SHEAR STRENGTH



DYREGROV CONSULTANTS CONSULTING GEOTECHNICAL ENGINEERS		**UNDRAINED SHEAR STRENGTH** **NEWPCC, PROPOSED U V PLANT**	
SCALE NTS	DATE 08/12/04	MADE TJH	CHKD AOD
JOB 242663	FIGURE 7		



$Ph = 0.4 \gamma H$

Where: Ph = Lateral earth pressure on shoring (kPa)

γ = Soil unit weight (17.28 KN/M^3)

H = Wall height (M)

Note: Add surface load surcharge where applicable

DYREGROV CONSULTANTS CONSULTING GEOTECHNICAL ENGINEERS	EARTH PRESSURE DISTRIBUTION TEMPORARY SHORING NEWPCC DISINFECTION FACILITY

SCALE NTS	DATE 07/12/04	MADE TJH	CHKD AOD	JOB 242663	FIGURE 8