

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 30M12-220

DIST. 6 REGION

W.P. No. 159-81-01

CONT. No.

W. O. No.

STR. SITE No. 24-124

HWY. No. 401

LOCATION DERRY ROAD UNDERPASS

STRUCTURE REPLACEMENT

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



Ministry
of
Transportation

PATROL YARD -
HWY 10 & HWY 401 NE CORNER
JOHN DEVRIS
(416) 670-3080

FILE No. _____ DATE RON MARTIN

REMARKS _____

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 159-81-01 DIST 6
HWY 401 STR SITE 24-124 Bridge
24-252 Culvert

Culvert Extension Retaining Wall & HML
Hwy 401 and Derry Rd.

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GEOCRES 30M12-220

DATE **SEP 28 1994**

FOUNDATION INVESTIGATION REPORT
For
Hwy 401 Structure Replacement over Derry Road
Culvert Extension, Retaining Wall, and HML
W.P. 159-81-01,
Str Site: 24-124 (Bridge), Str Site: 24-252 (Culvert)
Hwy. 401, District 6, Toronto

INTRODUCTION

This report summarizes the results of a supplementary foundation investigation at the above mentioned sites. This report should be read in conjunction with the Geotechnical report prepared by Geocon, Lavalin, dated September, 1990. The initial investigation by Geocon was carried out for the replacement of Hwy 401 and Derry Road bridge structure. Since then the scheme of the structure has been revised. The new bridge structure will be longer and wider than originally proposed, but the Geocon report is adequate. This investigation was carried out to determine soil condition at the inlet and outlet of Mullet creek culvert and five new highmast light pole foundations at Hwy 401 on the east and the west sides of Derry road. The investigation was carried out at the request of Central Region Structural Section.

SITE DESCRIPTION

Please refer to Geocon's report dated September, 1990

INVESTIGATION PROCEDURES

The supplementary fieldwork for soil information at the inlet and outlet of the Mullet Creek culvert and near proposed highmast light foundations was carried out on 94 06 14 and 94 06 15 and consisted of 4 sampled boreholes (BH 1 through BH 4) advanced to depths ranging from 6.1m to 7.7m below ground surface.

The boreholes were advanced using a CME-55 track-mounted auger machine equipped with solid augers.

Sampling was carried out at each borehole location by means of a 50mm O.D. split spoon sampler driven into the soil according to the specifications of the Standard Penetration Test (ASTM D 1586).

Samples were obtained at 0.7m intervals within the first 6.1m and then at 1.5m intervals thereafter.

Groundwater levels were obtained by monitoring the levels in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled at the completion of the fieldwork.

SUBSURFACE CONDITIONS

The stratigraphy at the site can be described as fill (generally associated with Hwy 401 embankment) overlying about 1m to 4m thick glacial till which in turn overlies shale bedrock of Queenston formation. Near the inlet and outlet of the Mullet creek culvert the soil mainly consists of about 3m to 4m thick clayey silt to silty clay glacial till underlain by weathered shale at elevation 174.3m

For detail soil information please refer to attached borehole logs (BH 1 through BH 4 for supplementary investigation) and Geocon's report dated September, 1990

The locations of new boreholes (BH1 through BH4) are shown on the attached drawing Dwg. No. 1598101-A.

Groundwater Conditions

In supplementary investigation groundwater was encountered in three boreholes (BH1,2 and 3). The water level ranged from 177.1m (BH 3) to 180.6m (BH 1) at depths 1.2m (BH3) to 2.3m (BH1) below ground surface.

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$, $\phi = 35^\circ$, $K_o = 0.43$, $K_a = 0.27$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$, $\phi = 30^\circ$, $K_o = 0.50$, $K_a = 0.33$
Native Soil	$\gamma = 20.0 \text{ kN/m}^3$, $\phi = 26^\circ$, $K_o = 0.56$, $K_a = 0.39$

If the structure is to be designed as a rigid frame then the coefficient of earth pressure at rest (K_o) should be used. For structural elements rigidly connected to the concrete box culvert, at rest condition (K_o) should be used to calculate lateral pressure.

Sliding Resistance

Sliding resistance calculations will not be required for culvert foundation. Sliding resistance for wing walls footings should be calculated in accordance with the O.H.B.D.C. assuming unfactored angle of friction, $\phi = 28^\circ$

Stability and Settlement

No deep seated stability problems are anticipated for the proposed construction. Total and differential settlement will be less than 25mm if the foundation recommendations are followed.

CONSTRUCTION CONSIDERATION FOR CULVERT

Temporary Diversion

To facilitate the construction of the culvert, the flow of the creek would have to be diverted. This could be achieved by constructing impervious earth dikes composed of suitable clay material (CH, see OPSS 1205) and pumping water from the up stream side to the down stream side possibly through pipes running inside the existing culvert.

Dewatering

The excavation for culvert construction would extend below prevailing groundwater table. It would be required to lower the groundwater table prior to the excavation.

A special provision should be in the contract requiring the contractor to lower the groundwater table. The contractor should be advised that cohesionless material may be encountered at the proposed culvert foundation elevations and it would be susceptible to disturbance under conditions of unbalanced hydrostatic head. The contractor should also be advised to construct without disturbance to the underlying foundations. Although the dewatering method is the responsibility of the contractor, it is anticipated that the groundwater can be lowered by a system of oversize excavation and sumps. The contractor should submit his dewatering proposal for review a minimum of 15 working days prior to construction.

Excavation

It is expected that the depth of excavation for culvert, wing walls and retaining walls will be up to 3m. Temporary excavation up to 4m deep would be stable at 1H:1V above water table and 2H:1V below water table.

Cambering

Due to competent soil condition, no significant settlements are anticipated. Therefore, cambering is not required.

Construction Joints

Proper joints will be required between the existing culvert and the culvert extension. Such joints should be able to accommodate differential settlement and provide proper seal.

Erosion Protection

The existing culvert appears to be performing well. A seal of cohesive material (CI-CH clay) with a minimum thickness of 0.6m should be constructed at the culvert inlet. The seal should extend a minimum of 2m on each side of the culvert inlet and from the high water level down the embankment to the creek bed. The material for clay seal should be as per OPSS 1205. If suitable clay is not available then clay mixture should be prepared as per OPSS 1205.05.03. Rock protection will be only required at the inlet since no changes will take place at the outlet. At the inlet a minimum blanket thickness of 0.6m should be placed to protect the embankment. It should extend from the high water level to the toe of the slope and at least 2m along the creek bed. In transverse direction, the erosion protection should extend a minimum of 5m.

Weep Holes

In order to relieve excess hydrostatic pressure behind the retaining walls sufficient weep holes should be provided.

Highmast Light Foundations

It is proposed to install 5 high mast lighting poles (F1 through F5) along Hwy 401 median between stations 12+350 and 13+050 (Hwy 401 chainage). The details of high mast lighting poles locations and elevations are attached to this report in Appendix A.

The High Mast Lighting poles will be founded on single reinforced concrete caissons. The foundations for HML should be designed in accordance with the methods described by B.B. Broms in the following two papers:

Broms, B.B.; Lateral Resistance of Piles in Cohesive Soils,
Journal of the Soil Mechanics and Foundations Division,
ASCE, Vol.90, No.SM2, Paper 3825, March 1964.

Broms, B.B.; Lateral Resistance of Piles in Cohesionless Soils,
Journal of the Soil Mechanics and Foundations Division,
ASCE, Vol.90, No.SM3, Paper 3909, May 1964.

There will be no significant grade changes at any of the pole locations.

If the grade is to be changed at the pole locations then, the most critical lowest surface elevations should be assumed for design purposes. It should be assumed that soil in the zone of frost penetration does not provide any lateral resistance. The depth of frost penetration at this site is 1.2m.

The design values at each of the HML locations are as follows:

SOIL PARAMETERS AT EACH HIGH MAST LIGHT POLES

HML Poles	W.L. Elev (m)	Elev (m) From - To	Soil Type	ϕ (Deg)	Q_u kPa	γ kN/m ³
F1 (BH 4)	Dry	190.8-188.5	Cohesive	0	200	20.0
		188.5-186.8	Cohesive	0	500	21.2
		186.8-Below	Shale	0	700	22.5
F2 (BH 4)	Dry	190.8-188.5	Cohesive	0	200	20.0
		188.5-186.8	Cohesive	0	500	21.2
		186.8-Below	Shale	0	700	22.5
F3 Geocon (BH 3)	180.0	186.0-178.8	Cohesive	0	100	19.6
		178.8-177.0	Non-Cohesive	40	0	21.2
		177.0-Below	Shale	0	700	22.5
F4 (BH 2)	177.6	183.5-177.2	Non-Cohesive	30	0	19.6
		177.2-175.8	Cohesive	0	200	20.0
		175.8-174.1	Cohesive	0	500	21.2
		174.1-Below	Shale	0	700	22.5
F5 (BH 1)	182.8	182.9-180.1	Non-Cohesive	28	0	19.6
		180.1-Below	Shale	0	700	22.5

Where:

HML = High Mast Lighting

ϕ = Apparent angle of internal friction for non-cohesive Soils

Q_u = Unconfined Compressive Strength (kPa)

γ = Unit Weight (kN/m³)

APPENDIX A
DETAILS OF HIGH MAST POLES

DESIGN BRIEF HWY 401 / DERRY Rd. W.P. No. 159-81-01

PROJECT TITLE HWY 401

Project No. 0.06812 Act Sub. 1 Sub. 2 Sub. 3 Design No.

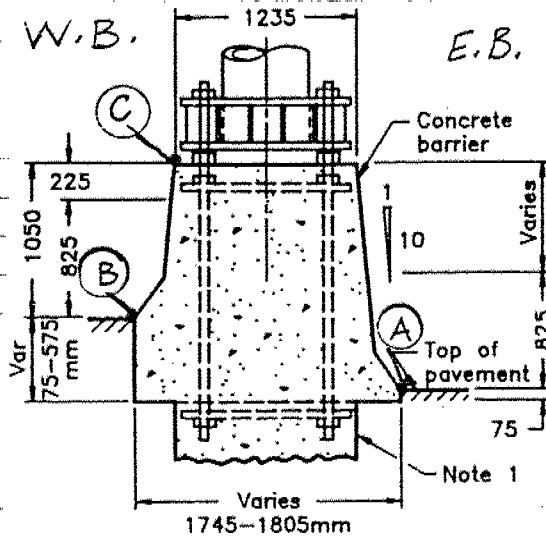
SUBJECT HIGH MAST LIGHTING POLES ELEVATIONS

Page 1 of

DESIGNED BY: A.N.D.

940218

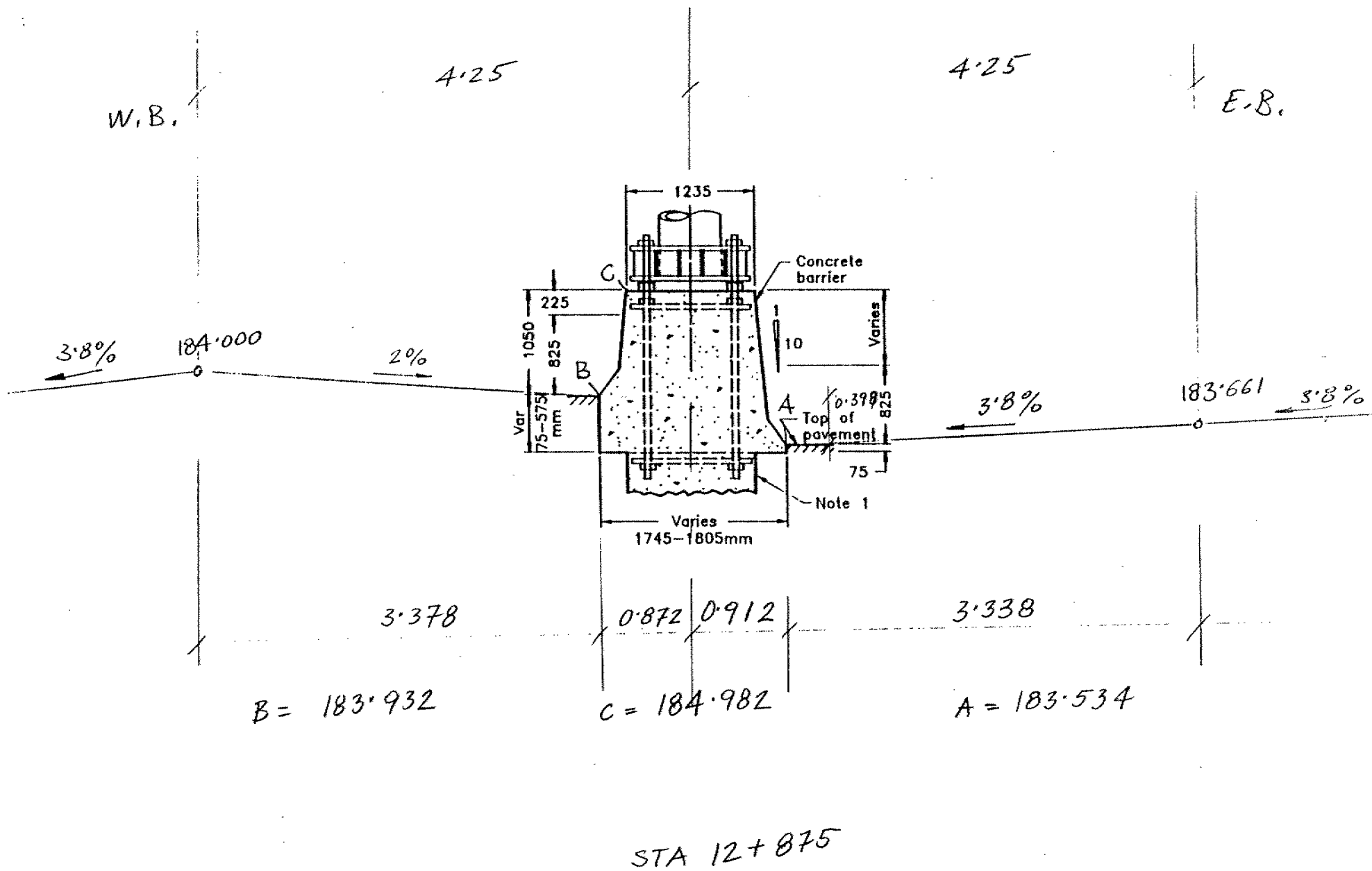
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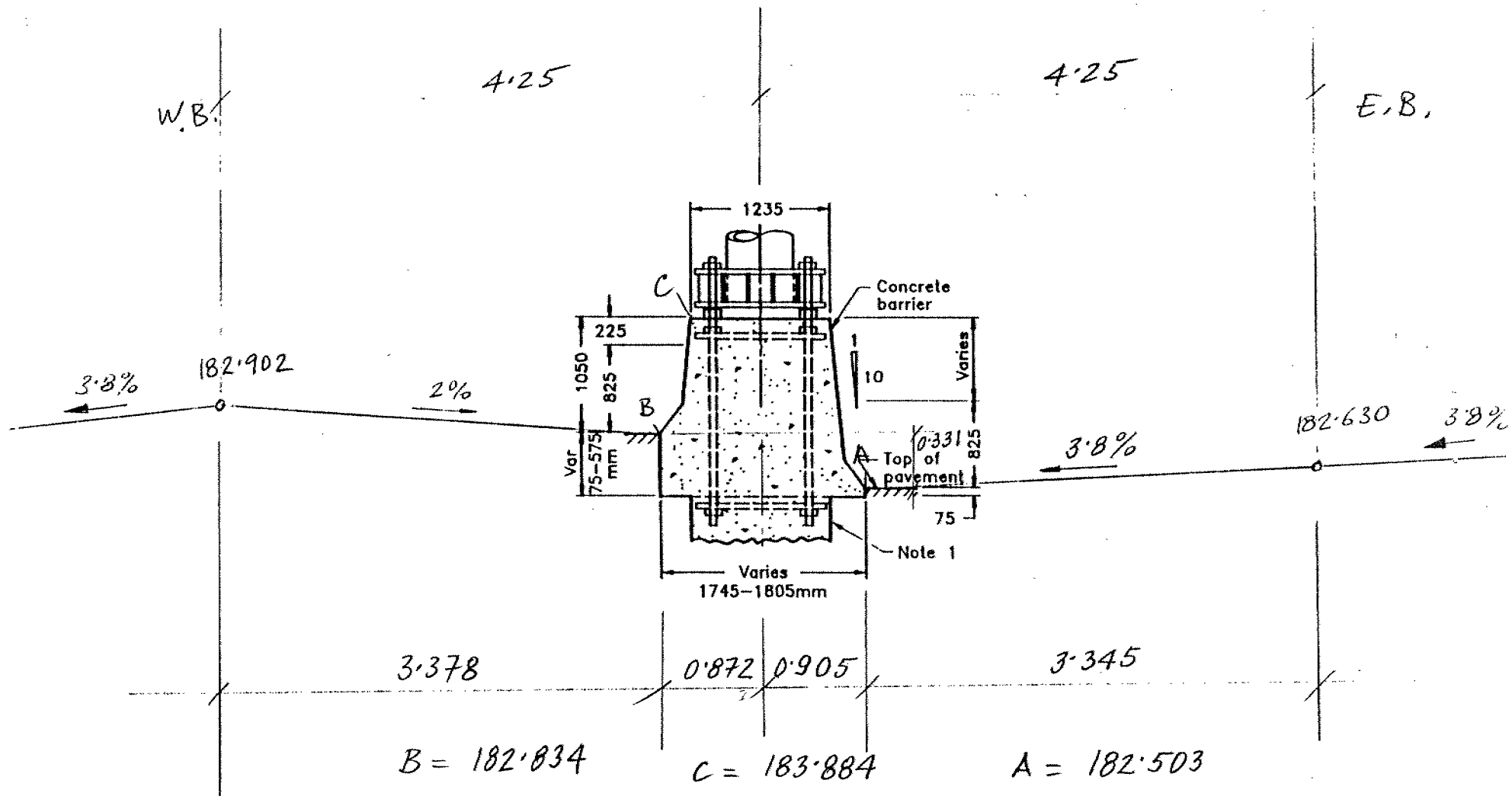


POLE LOCATION	ELEVATION		
STATION	(A)	(B)	(C)
12 + 350	190.675	190.715	191.765
12 + 525	188.406	188.810	189.860
12 + 700	185.973	186.352	187.402
12 + 875	183.534	183.932	184.982
13 + 050	182.503	182.834	183.884

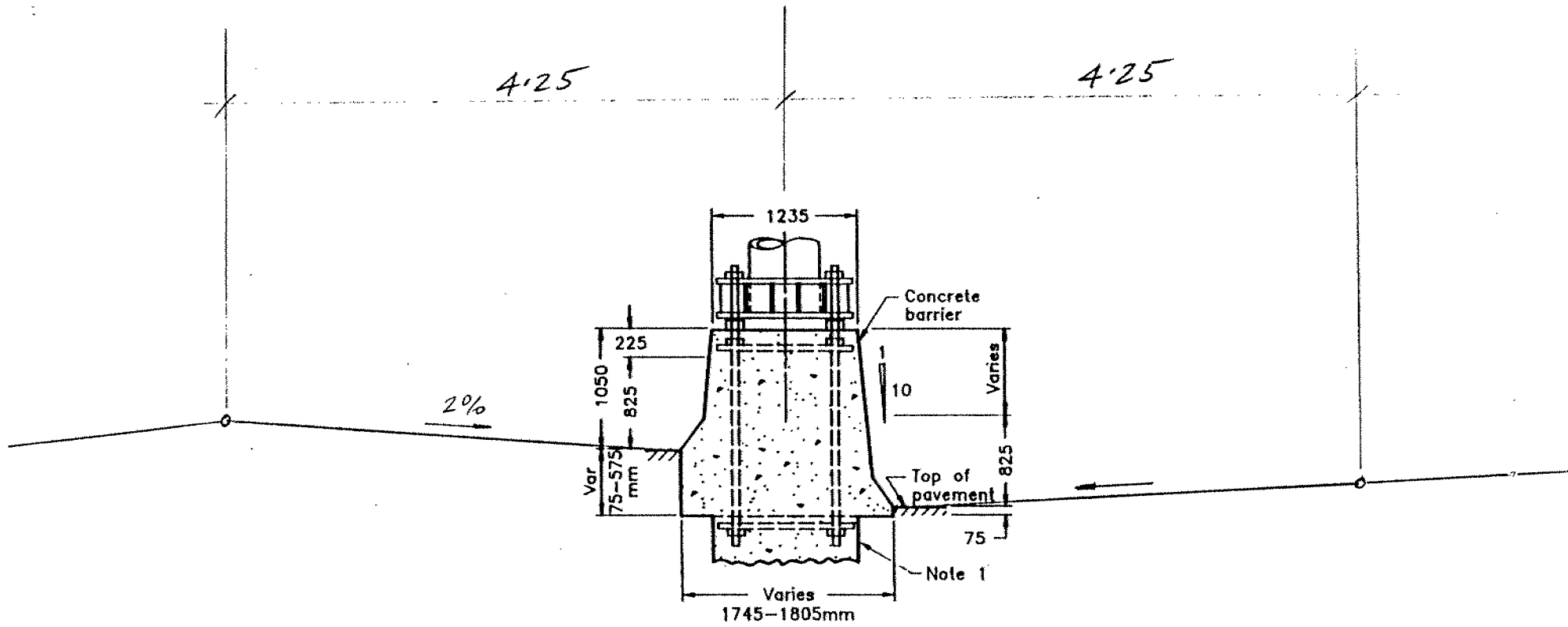
16 beyond the scope of this project.

13 + 215 ?





STA. 13+050



APPENDIX B

NSSP FOR HIGH MAST POLE CONSTRUCTION

(AN EXAMPLE FROM ANOTHER REPORT)

WP NO 368-87-00 CONTRACT NO _____ DISTRICT NO 6 HWY NO 407

LOCATION 407/427 Interchange TYPE OF WORK _____

1. This S P is new (✓) ☐

This S P replaces No N/A

Remarks:

Explanation of Intent:

To define High Mast Pole foundation construction

2.	Item No	Spec No	Title or Item Description
	45	631	CONCRETE FOOTING FOR HIGH MAST POLES

CONSTRUCTION

The Contractor is advised that variable types of subsurface material may be encountered at the high mast light pole locations; for additional information regarding soil conditions the Contractor is referred to the Foundation Investigation Report.

For bidding purposes it may be assumed that:

- Ground water is at or near the surface.
- If cohesionless material is encountered, it would be susceptible to disturbance under conditions of unbalanced hydrostatic head.
- If glacial deposits are encountered, there is a probability that occasional cobbles and boulders may be encountered within the deposit.

The Contractor is responsible for constructing the high mast pole foundations without disturbing the material at the sides or bases of the foundations. The Contractor shall submit eight copies of the proposed construction method to the Engineer for review a minimum of 15 working days prior to the commencement of construction of these foundation elements.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to do the work.

3. Structural Section

D. Wong

Initiated by _____

Detailed by _____

Approved by _____

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N}

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_c - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 159-81-01 LOCATION Coords.: N 4 829 178, E 284 604 ORIGINATED BY LO
DIST 6 HWY 401 BOREHOLE TYPE Solid Stem COMPILED BY LO
DATUM Geodetic DATE 1994 06 14 CHECKED BY KA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100		
182.9	Ground Surface												
0.0	Silty Sand With Some Gravel Compact (Probable Fill)		1	SS	10	182							
181.6													
1.3	Sandy Silt With Some Gravel and Organics Loose		2	SS	7	181							
180.8													
2.1	Saturated Sand Loose		3	SS	1	180							
180.1			3a	SS	5	180							
2.8			4	SS	100	180							
						179							
						178							
	Red Weathered Shale		5	SS	100	178							
						177							
176.6			6	SS	100	177							
6.3	End of Borehole												

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 159-81-01 LOCATION Coords.: N 4 829 034, E 284 407 ORIGINATED BY LO
DIST 5 HWY 401 BOREHOLE TYPE Solid Stem COMPILED BY LO
DATUM Geodetic DATE 1994 06 15 CHECKED BY KA

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
178.3	Ground Surface															
0.0	Clayey Silt to Silty Clay With Some Gravel V. Stiff to Hard (Glacial Till)		1	SS	28	/42cm										
			2	SS	100											
			3	SS	100											
			4	SS	102											
174.3	Red Weathered Shale		5	SS	58	/36cm										
4.0			6	SS	103											
			7	SS	100											
			8	SS	100											
170.6	End of Borehole															

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 159-81-01 LOCATION Coords: N 4 829 009, E 283 869 ORIGINATED BY LO
DIST 6 HWY 401 BOREHOLE TYPE Solid Stem COMPILED BY LO
DATUM Geodetic DATE 1994 06 14 CHECKED BY KA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
190.8	Ground Surface																
0.0	Clayey Silt With Some Sand and Gravel V. Stiff (Probable Fill)		1	SS	16	DRY *	190										
189.5			2	SS	47		189										
1.3	Silty Clay With Some Sand and Gravel Hard (Glacial Till)		3	SS	70		188										
			4	SS	65		187										
186.8			5	SS	104	/18cm	186										
4.0	Red Weathered Shale		6	SS	100	/18cm	185										
184.7			7	SS	100	/3cm											
6.1	End of Borehole																

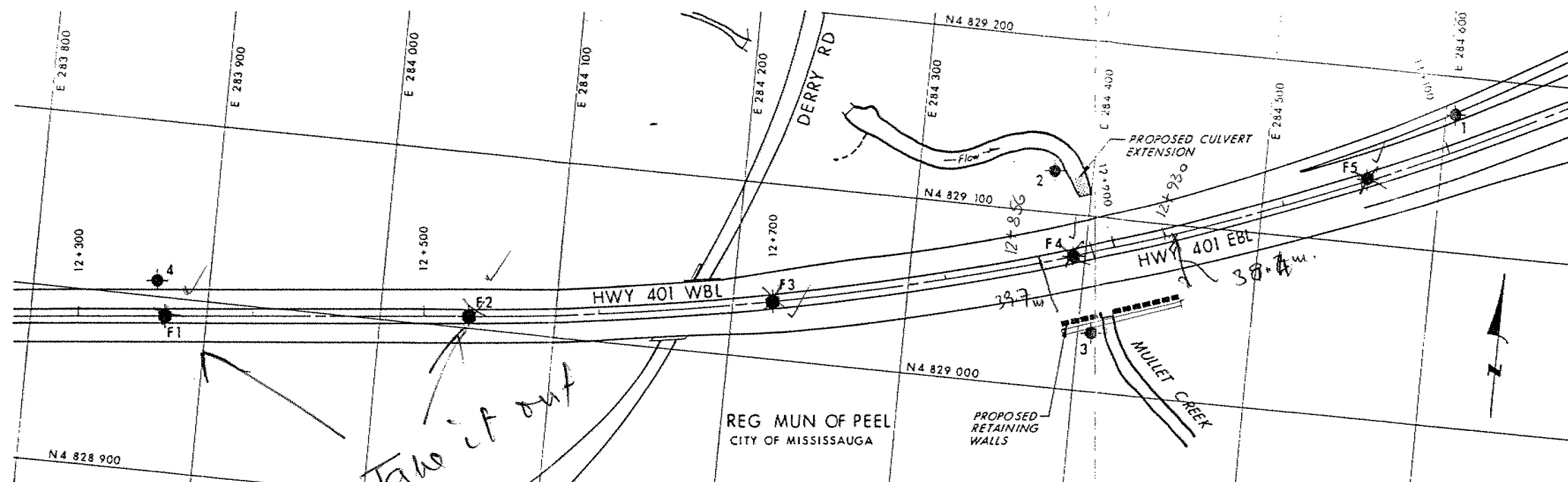
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 159-81-01



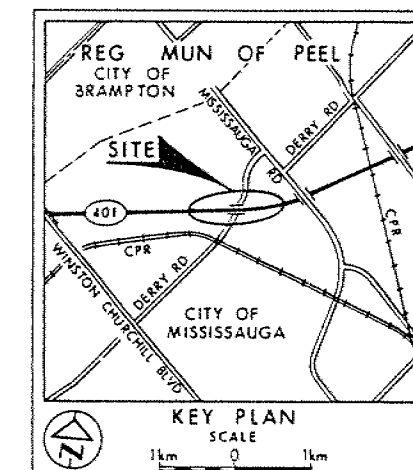
HWY 401 & DERRY RD
CULVERT EXT'N, RETAINING WALL & HML
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



PLAN
SCALE
20m 0 20m

Note:
For Subsoil information Refer to
Record of Borehole sheets



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊗ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation 1994 06
- High Mast Lighting Pole

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	182.9	4 829 178	284 604
2	178.4	4 829 124	284 377
3	178.3	4 829 034	284 407
4	190.8	4 829 009	283 869

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

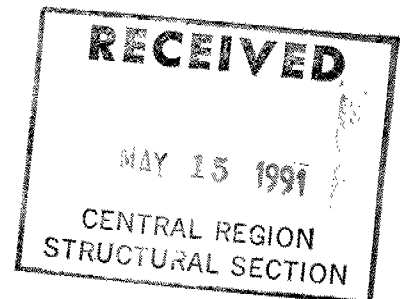
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen Cond.



REV.	DATE	BY	DESCRIPTION

Geocres No 30M12-220

HWY No 401	CHECKED	DATE 1994 07 14	DIST 6
SUBMD KA	CHECKED	DATE 1994 07 14	SITE 24-252
DRAWN DT	CHECKED	DATE 1994 07 14	DWG 1598101-A



T11659

REPORT TO

FENCO ENGINEERS INC.
WILLOWDALE ONTARIO

GEOTECHNICAL SUBSURFACE INVESTIGATION
HIGHWAY 401-DERRY ROAD UNDERPASS
MISSISSAUGA ONTARIO

WP 159-81-01

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September, 1990



Geocon

GEOTECHNICAL CONSULTANTS

GEOCON INC.
3210 AMERICAN DRIVE, MISSISSAUGA
ONTARIO, CANADA L4V 1B3
TEL.: (416) 673-1664
FAX.: (416) 673-0282

September 6, 1990

Fenco Engineers Inc.
Atria North - Phase II
2235 Sheppard Avenue East
Willowdale, Ontario
M2J 5A6

Attention: Mr. R. Temple, P.Eng.

Re: Geotechnical Subsurface Investigation
Highway 401-Derry Road Underpass
Mississauga, Ontario

Dear Sirs:

We are pleased to submit two (2) copies of the geotechnical design report on the above project. Comments received from yourself and members of your staff on the draft copy of this report have been incorporated. Should you have any questions or require clarification on any matters please do not hesitate to contact us.

In the meantime, we would like to take this opportunity to thank you for your co-operation during the preparation of this report and look forward to being of further assistance on the construction phase of this project and on other similar projects.

Yours very truly
GEOCON INC.

R.D. Powell, P.Eng.
General Manager

RDP:dtj
T11659/53772

Lavalin

DERRY ROAD AND 401

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1.0

INTRODUCTION

reconstruction

Geocon Inc. (Geocon) was retained by Fenco Engineers Inc. (Fenco) to perform a geotechnical investigation at the intersection of Highway 401 and Derry Road West, Mississauga, Ontario for the proposed widening of the existing Derry Road underpass. The purpose of the investigation was to establish the subsurface conditions present at the site and based on that information provide geotechnical recommendations pertinent to the foundation elements of the project.

The work was carried out pursuant to Fenco's authorization on April 17th, 1990 and in general accordance with our proposal letter of April 12, 1990.

2.0 SITE AND PROJECT DESCRIPTION

The site of the proposed work is located at the intersection of Highway 401 and Derry Road West, Mississauga, Ontario (Drawing T11659-1). At this location, an existing two lane underpass conveys Derry Road under Highway 401 (8 lanes). The natural terrain in the area is relatively flat with a gentle slope towards the North. An eastward flowing drainage ditch passes under Derry Road to the north of Highway 401 and eventually flows under Highway 401 via a culvert approximately 200 m east of the Derry Road/Highway 401 underpass.

The Highway 401 approach fills to the existing Derry Road underpass are of the order of 6 to 7 m high with side slopes of 2 Horizontal(H):1 Vertical(V).

The proposed upgrading works at this location comprise widening the Derry Road underpass to 4 lanes and at the same time widening Highway 401 from 8 to 10 lanes. To achieve this the existing single span Derry Road underpass will be replaced by a new two span structure. In addition, the underpass structure will be lengthened about 20 m to the north of its present location to encompass the proposed widening of the Highway 401 embankments (Drawing No. T11659-1). It is understood that the proposed east abutment of the new underpass will be located approximately 2.0 m east of the existing east abutment of the present underpass. The proposed new west abutment is located approximately 42 m west of this location with a central pier located at mid-span.

The proposed method of construction of the underpass will be achieved using a two stage construction approach. Firstly, the northern half of the new underpass will be completed, followed by that of the southern portion. Stage 1 construction will also encompass the widening of the Highway 401 embankments to the north. During construction of both stages, the adjacent embankments supporting Highway 401 will require temporary support measures to

maintain its continued operation during construction. The length of embankment requiring support is considerably greater during Stage 1 construction than that of Stage 2.

3.0 GEOLOGY

Based on the available geological information given in published reports and on our experience in the general area, it is anticipated that fine grained glacial till of the Peel Till Plain overlies red shale bedrock of the Queenston Formation at relatively shallow depth. Locally, it is known that pockets of silt to silty sand till also exist within the usually fine grained glacial till.

4.0 INVESTIGATION PROCEDURE

4.1 Field Investigation

The fieldwork portions of this investigation were carried out between May 9 and May 15, 1990 using a CME 75 Bombardier mounted drill rig supplied and operated by Longyear Canada Inc., Concord, Ontario. During that time 9 boreholes and 3 augerholes were drilled at the locations shown on Drawing T11659-1. Boreholes 1 to 6, drilled at the proposed locations of the bridge foundations, varied from a minimum depth of 2.0 m at Borehole 1 to a maximum of 9.6 m at Borehole 4. Boreholes 7, 8 and 9 and Augerholes 2 and 3 were drilled at the location of the proposed fill to the north of Highway 401. Augerhole 1 was drilled in the middle of the drainage channel underneath the proposed extension of the Highway 401 culvert.

In addition to the Boreholes and Augerholes, 3 Portable Dynamic Cone Penetration Tests (PDCPT) were put down at the locations shown on Drawing No. T11659-1. The PDCPT involves dropping an 8 kg hammer from a vertical height of 575 mm to advance a 20 mm diameter 60° cone. The energy per blow applied to the projected tip area of the cone is roughly equivalent to that of the more common Standard Penetration Test (SPT) modified to take account of the truncated nature of the cone used in the SPT test. Hence, the number of blows required during the PDCPT to advance the cone 0.3 m can be considered equivalent to the SPT "N" value. A record of the PDCPT results obtained during this investigation is presented in Appendix A.

Boreholes 1 to 6 were advanced using 200 mm hollow stem augers to auger refusal. At Boreholes 3 and 4, the boreholes were advanced into the underlying bedrock using rock coring techniques with an NQ size core barrel. During drilling of the overburden materials, Standard Penetration Testing (SPT) with associated split spoon sampling was performed at regular intervals. The undrained shear

strength of cohesive soils were determined on recovered split spoon samples using a pocket penetrometer. Detailed records of the subsurface conditions at each borehole location, together with the results and location of the in-situ testing, are presented on the Boring Logs presented in Appendix A at the rear of this report. A summary of the borehole information is presented in Table C1 of Appendix C.

Augerholes 1 to 3 were advanced using hand and power auger methods. The encountered soil conditions at these locations are also presented in Table A1 of Appendix A.

At Borehole 4, due to lost core between the depth interval 6.4 and 8.0 m, a second borehole, designated Borehole 4A, was drilled a short distance away to define the stratigraphy in this lost portion (Appendix A).

Boreholes 7 to 9 were drilled using solid stem augers with continuous sampling of the subsurface materials from the flights of the auger. A record of the subsurface conditions at each of these locations is presented in Appendix A and summarized in Table C1 of Appendix C.

Standpipe piezometers, to permit monitoring of the ambient groundwater conditions, were installed in Boreholes 2 to 6.

The drilling was supervised at all times by a member of our engineering staff who directed drilling operations, logged the samples, installed the piezometers and generally insured adherence to good drilling practices. All recovered samples were returned to our Mississauga laboratory for further examination and testing (Section 4.2). The recovered soil and rock samples will be retained until February 1, 1991 at which time they will be discarded unless we receive instructions otherwise.

Borehole locations and elevations were determined by Geocon personnel. Elevations were referenced to benchmark No. T.T.B.M. No. 833 located on the face of the east abutment at the south end of the existing Derry Road underpass (Drawing T11659-1). The elevation of the benchmark was reported as 181.006 m (593.883 ft.) by the drafting department of the City of Mississauga.

4.2 Laboratory Testing

Laboratory testing of the recovered samples was limited to soil index testing to determine the natural moisture content, Atterberg Limits and grain size distribution of the encountered strata. The results of moisture content and Atterberg Limit tests are presented on the Boring Logs (Appendix A). The results of three grain size distribution analyses are presented on Figure B1 of Appendix B.

5.0 SUMMARIZED SUBSURFACE CONDITIONS

5.1 Bridge Underpass (Boreholes 1 to 6)

The stratigraphy at the site of the proposed underpass can be described as comprising a layer of fill overlying glacial till which in turn overlies shale bedrock of the Queenston Formation. A detailed description of these units, based on the borehole information and laboratory testing is presented in Appendix C and summarized briefly below.

Fill

Varying in thickness from 0.5 m at Borehole 1 to 2.0 m at Borehole 4, this layer is variable in nature but generally comprises a mixture of clayey silt with some sand and gravel and organics. Generally this material was in a loose state of relative density.

Glacial Till

The thicknesses of the till layer varied from a minimum of 1.0 m at Borehole 1 to a maximum of 7.4 m at Borehole 6 and generally shows a trend of increasing thickness towards the North and East. A summary of the inferred surface elevations and thicknesses of the till are presented in Table C1. In general, the till layer comprised a near surface cohesive silty clay till underlain by a cohesionless silty sand till. Both deposits contained varying amounts of sand and gravel.

SPT values within the till unit were generally in excess of 30. The undrained shear strength of the cohesive till was generally found to be in excess of the pocket penetrometer range, i.e. in excess of 225 kPa.

Shale

Brownish red shale of the Queenston Formation was encountered below the till. In accordance with the thickening till deposit towards the North and East, the elevation of the surface of the shale exhibits a similar trend (Drawing T11659-2, Table C1 - Appendix C). The upper 1.0 to 1.5 m of the shale was found to be weathered whereas the underlying sound shale, frequently interbedded with thin limestone layers, exhibited RQD values generally in the range of 55 to 70% indicating the material to be of fair quality. Table C1 (Appendix C) presents the approximate elevations of the weathered and sound shale. Unconfined compressive strength of the sound shale, based on quoted values in the literature, is estimated to be of the order of 5 to 15 MPa.

5.2 Highway Embankment (Boreholes 7, 8 and 9; Augerholes A2 and A3; PDCPT 3)

The soil conditions under the proposed Highway 401 embankments to the west of Derry Road, comprised up to 2.6 m of fill and topsoil material overlying glacial till. To the east, soil conditions comprised primarily glacial till at or near the ground surface.

5.3 Highway 401 Culvert (Augerhole A1, PDCPT 1 and 2)

Similar to the soil conditions over the rest of the site, the proposed location of the culvert extension is underlain by glacial till. The surface elevation of the till in the vicinity of culvert extension, underneath the existing gabion lined channel, is approximately 176.2 m.

5.4 Groundwater Conditions

The groundwater level, as measured on May 29, 1990 (about 2 weeks after drilling) in standpipe piezometers installed at Boreholes 2, 3, 4, 5 and 6 is relatively uniform across the site at elevations 179.6 to 180.0 m (Table C1 - Appendix C).

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 Foundation Allowable Bearing Pressures

Based on the observed stratigraphy at the site, two viable foundation alternatives are worthy of consideration, i.e. spread foundations placed near the surface of the till or alternatively, if the loads are high, the use of caissons socketed into the underlying sound shale.

Spread Foundations Within the Till

The allowable bearing pressure at the Ultimate Limit State (ULS), for vertically loaded foundations, is 650 kPa. To limit settlements to a maximum of 25 mm (differential settlement would be of the order of half this value), an allowable bearing pressure at the Serviceability Limit State (SLS) Type II of 400 kPa is recommended. Allowable bearing pressures at the ULS condition should be reduced in accordance with Figure 6-7.3.3.5 of the Ontario Highway Bridge Design Code if the foundation will be subjected to inclined loading.

For the above foundation bearing pressures, the base of the proposed spread footings must be located at least 0.5 m below the surface of the till layer and a minimum foundation width of 1.0 m employed. Further, the base of the footing should be inspected by a geotechnical engineer to ensure that the base of the foundation will be formed within competent material. Any localised soft zones or zones containing deleterious matter should be subexcavated and backfilled with suitable material (MTO Granular B, or better). In light of the fine grained nature of the till, and the high groundwater regime, consideration should be given to applying a thin mass concrete layer to the base of the excavation immediately upon excavation, but after inspection, to prevent wetting and softening.

Bored, Cast In Place Caissons

For foundations of this type, an allowable end bearing pressure at the ULS of 2.5 MPa can be assumed for caissons at least 0.6 m in diameter and socketed at least 0.5 m into the sound shale. The elevation of the sound shale varies from approximately elevation 176 m at the south end of the underpass to elevation 173 m at the north end. Foundations of this type, because of the unyielding nature of the bedrock, need not be checked for settlement at the SLS Type II condition.

The base of each caisson should be inspected visually by a qualified geotechnical engineer to ensure that the base of the caisson is free from loose debris and that the caisson is founded in competent unweathered shale bedrock. Caisson inspection should only be performed under the protection of a safety liner.

Caisson installation through the till may require a lining to prevent the ingress of water from the sandier portions of the till. Further, boulders were encountered at random locations within the till during drilling and may create some problems during caisson excavation.

6.2 Permanent Retaining Walls

6.2.1 Lateral Earth Pressure

Providing free draining material is placed within 3.0 m from the abutments or wing walls, lateral earth pressures acting against the back of the abutment may be estimated using the relationship,

$$p = K \gamma H$$

where p = Pressure acting at depth H

γ = Unit weight of embankment fill material

H = Depth below the final grade of the embankment

K = Earth Pressure Coefficient

Providing large vibratory compactors are not used within 3.0 m from the back of the abutment, an "at rest" soil condition can be assumed to exist behind the abutment and a K value of 0.5 can be used for the SLS condition and a value of 0.58 at the ULS condition. Alternatively, if horizontal movements of the top of the abutment of at least 0.1% of the wall height are permitted, an active soil condition can be assumed to exist and a value of 0.33 used for K at the SLS condition and a value of 0.41 used at the ULS condition. A unit weight of retained soil of 20 kN/m^3 may be used in the computation.

6.2.2 Basal Sliding Resistance

For the estimation of the basal sliding resistance, friction coefficients between the base of the foundation and the underlying material of 0.40 and 0.32 may be used at the SLS and ULS condition, respectively. If additional resistance is required, the passive resistance in front of the foundation may be computed using a K value of 3.03 and 2.44 at the SLS and ULS, respectively. If passive resistance in front of the toe of the foundation is allowed for in the design, it is important that this area not be disturbed during construction. Alternatively, a key may be incorporated into the base of the foundation. Because of the generally high water table, a submerged unit weight of 10 kN/m^3 should be used in the computation of the passive resistance detailed above.

6.2.3 Backfill

As mentioned previously, backfill material against abutments and/or wing walls should comprise free draining granular material (MTO Granular B or better). Further, heavy compaction equipment should not be used within 3.0 m from the back of the wall.

To reduce settlement of the fill adjacent to the abutment and relative to that of the existing Highway 401 fills, it is

recommended that the backfill be compacted to 95% of its Modified Proctor maximum dry density value. Individual lifts should not exceed 225 mm in thickness. Further, it is recommended that the upper 1.0 m of the backfill below the road subbase layers be compacted to 98% of its Modified Proctor maximum dry density value.

6.3 Temporary Earth Support

Based on the geometry and proposed construction sequence, a significantly larger amount of temporary earth supports is required for Stage 1 construction than that of Stage 2. Accordingly, support recommendations for both stages are discussed separately below.

6.3.1 Stage 1 Construction

For this stage of construction, the use of a tied-back conventional soldier pile and lagging wall is recommended. During installation of this wall it is recommended that the soldier piles be keyed at least 0.5 m into the sound shale (approximate elevation 173.0 m). Alternatively, the required penetration of the soldier piles below the base of the excavation to prevent base failure may be determined by assuming each soldier pile has a horizontal resistance given by the relationship,

$$P_p = \frac{3 W D^2 K_p \gamma'}{2} \text{ where}$$

- P_p = Total estimated Horizontal Resistance
- W = Width of the flange of the soldier pile
- D = Depth of Penetration of the Soldier Pile below the base of the excavation.
- K_p = Coefficient of Passive Resistance
- γ' = Submerged unit weight of the Overburden Material

Values of 3 and 10 kN/m³ may be used in the design for K_p and the submerged unit weight respectively. A Factor of Safety of at least 1.5 should be provided against this mode of failure. If the soldier piles are installed in pre-drilled holes and subsequently backfilled with concrete, the term "W" in the above equation may be replaced by the diameter (D) of the concrete column.

Lateral earth pressures against the soldier pile and lagging wall, may be estimated using a triangular distribution given by the relationship,

$$P = \gamma K H$$

where

$$P = \text{Unit pressure at a depth } H \text{ below the top of the ground surface behind the wall}$$

$$\gamma = \text{Unit weight of the retained material}$$

$$K = \text{Earth pressure coefficient}$$

For this proposed wall type, an active earth pressure condition can be assumed to exist behind the wall and hence, a value for K of 0.33 can be used in the design. The unit weight of the backfill may be taken as 20 kN/m³. The observed water elevation of 180.0 m across the site is at or close to the base elevation of the proposed excavation and therefore need not be considered in the design.

The maximum unsupported height of the exposed material between the soldier piles, prior to the installation of the wood lagging, should not exceed 1.0 m. The dimensions of the wood lagging at depth H, should be sufficient to withstand the pressure as calculated using the above relationship, integrated over the surface area of the lagging.

Tie-Back Design

For this phase of construction, it will be possible to install anchors through the existing Highway 401 fill into the underlying till or sound shale. For anchors formed within the sound shale and providing an expanding grout with a minimum crushing strength of 30 MPa is used, an allowable working bond stress of 300 kPa between the grout and the rock can be used in the calculation of the anchor working load. In the computation of the anchor working load, the upper weathered 1.5 m (vertical) of the shale should not be considered. Based on the observed elevations of the sound shale at Boreholes 3 and 4, it is estimated that the required anchor free length to reach the sound shale for a 30° anchor installed from elevation 185.0 m is of the order of 20 m and 14 m for a 45° anchor. Hence, some consideration may be given to forming the fixed length of the anchor within the till and weathered shale layers to reduce the required anchor lengths (required free length for the above 30° anchor to reach the surface of the till layer is of the order of 12 m).

For preliminary design purposes, provided an expanding grout with a minimum crushing strength of 30 MPa is used, bond stress between the grout and the till/weathered shale of 80 kPa may be assumed to calculate the ultimate capacity of the anchor. At the preliminary design stage, a Factor of Safety of 3 should be applied to the ultimate anchor capacity to obtain the allowable working load. However, for contractual arrangements, it is recommended that anchors of this type be the subject of a performance specification. We would be pleased to assist you in the preparation of this specification if required.

Due to possible strain incompatibilities of the grout/soil and grout/sound shale interfaces, the anchor fixed lengths should be formed either entirely within the till and weathered shale or within the sound shale but should not span the interface between these two material types. Based on the information obtained at the

location of Boreholes 1 to 6 (Drawing T11659-1) isolated boulders are present within the till and these may interfere with the installation of the tie-backs.

For anchors installed in the sound shale it is recommended that a prototype of each type of anchor should be tested to a minimum of 150% of the working load. If soil anchors are used two test anchors are recommended. All anchors should be proof loaded to at least 125% of the working load prior to "locking off" at 110% of the working load. We would be pleased to discuss acceptance criteria and testing methods at a later date. The above maximum test loads are applicable where a minimum factor of safety of 1.5 has already been incorporated in the calculation of the anchor working load.

The maximum depth of excavation prior to the installation of the tie-backs, must be chosen to limit the outward horizontal movement (towards the excavation) of the top of the free standing portion of the wall to an acceptable magnitude. The required structural analysis, in conjunction with the sizing and spacing of the soldier piles (wall rigidity), to determine this value is beyond the scope of this report. However, based on the relatively non-sensitive nature of the enclosed Highway 401 embankment to small movements, we recommend that a value of 15 mm be adopted as the maximum permissible horizontal movement of the top of the free standing portion of the wall prior to the installation of the tie-backs.

In order to limit the amount of movement of the soldier piles into the retained soil upon stressing of the tie-backs, it is recommended that wood lagging be installed to at least 0.6 m below the level of the tie-backs prior to their stressing. Further, during stressing of the anchors it is recommended that the movement of the face of the soldier pile wall be monitored and stressing stopped if these movements exceed 25 mm.

6.3.2 Stage 2 Construction

It is understood for this stage of construction only small lengths of temporary support are required adjacent to the recently constructed new underpass abutment. Tied-back soldier pile and lagging walls, installed and design as above are recommended.

As a result of the deep elevation and northward dipping nature of the underlying sound shale, the required anchor lengths to form the anchors wholly within this material is not considered economical. Therefore, the use of soil anchors with their fixed lengths formed wholly within the till deposit (and weathered shale) are recommended. Preliminary design of these anchors may be achieved following the method outlined in 6.3.1.

Alternatively, to avoid the use of the anchors, a deadman anchor block may be installed during backfilling operations to both bridge abutments. Design parameters of these anchors depend on the depth of embedment and location of the anchor relative to the back of the wall. However, for an anchor block installed 3 m below the finished grade of the proposed Highway 401 embankments and with a minimum height of 1.0 m, an allowable horizontal thrust of 180 kN/metre width of the anchor block may be assumed (at depth of 4.0 m this value may be increased to 320 kN/m). We would be pleased to discuss this option at a later date, when more detailed information regarding the proposed geometry is available.

6.4 Highway 401 Embankment Widening

Prior to embankment construction, all fill and topsoil materials should be removed to expose the surface of the underlying glacial till. Proofrolling of the exposed till surface should be performed to identify "soft spots", which should be subexcavated. From that level, the embankment can be constructed using side slopes not steeper than 2H:1V. Embankment material should meet the gradation requirements of MTO Granular B (or better). To limit differential

settlement between the new embankment and that of the old, minimum compaction requirements of 95% of the Modified Proctor Dry Density value are recommended. In addition, the upper 1 m of fill underneath the road subbase layer should be compacted to at least 98% of the Modified Proctor Maximum Dry Density value.

Materials excavated to expose the underlying glacial till will have to be assessed during construction for their suitability for use as a general fill. However, based on the observations of this material made during this investigation, it most likely will contain too much organic material for this use.

6.5 Highway 401 Culvert Extension

Based on the subsurface information, Section 5.3, the surface elevation of the glacial till underneath the existing gabion lined drainage channel is approximately 176.2 m with good quality till commencing at elevation 176.1 m. For spread foundations placed at or below elevation 176.1 m an allowable bearing pressure of 650 kPa and 400 kPa can be assumed at the ULS and SLS condition, respectively. The base of the foundation should be inspected by a geotechnical engineer prior to pouring of concrete.

6.6 General Construction Recommendations

6.6.1 Temporary Excavations

Temporary excavations within the existing Highway 401 embankment and other fill materials at the site will remain stable if constructed with side slopes not steeper than 2H:1V. Alternatively, steeper slopes of 1.5H:1V may be employed but slopes at this angle may require some maintenance as a result of sloughing and ravelling of the exposed face of the slope, especially when exposed to water. Should this steeper slope be preferred, it is recommended that the face of the slope be investigated immediately upon excavation by a qualified geotechnical engineer who can assess

the nature of the exposed material and suggest additional precautions that may be instigated to improve the performance of the slope, if required.

Temporary excavations within the underlying till materials up to a maximum depth of 2.0 m will remain stable if excavated with side slopes not steeper than 1H:1V. All temporary excavations should conform with the Occupational Health and Safety Act.

6.6.2 Dewatering

The groundwater elevation across the proposed site is of the order of 180.0 m or from 0.3 to 1.0 m below existing ground surface. Excavations within the fill materials present at the site below the level may experience some ingress of water. However, this should be readily handled by a properly filtered sump and pump arrangement. Excavations within the till, because of the cohesive and generally dense nature of this material, should experience very limited seepage quantities, with the exception of locally heavier inflows through coarser grained zones.

6.6.3 Frost Penetration

The estimated depth of frost penetration at the proposed site location is 1.2 m. All foundation elements should be provided with at least this minimum cover.

6.6.4 Supervision

Various elements of the works detailed within this report should be performed under the supervision of a geotechnical engineer. Specifically, this should include but not be limited to, inspection of foundation bases and the installation and testing of the anchors.

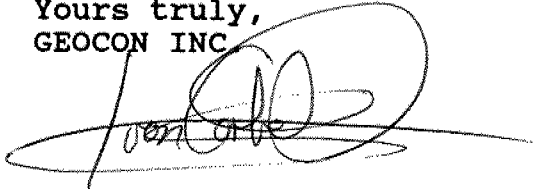
7.0 CLOSURE

This report was written by Mr. I. Corbett, P.Eng. and reviewed by Mr. B. Cooke, P.Eng.

We trust this report is sufficient for your present needs. However, should you have any questions please contact us.

The contents of the report are subject to the attached Terms and Conditions.

Yours truly,
GEOCON INC

A handwritten signature in black ink, appearing to read 'I. Corbett', is written over a horizontal line. The signature is stylized with a large loop and a long horizontal stroke extending to the left.

I. Corbett, P.Eng.
Geotechnical Engineer

IC:dtj
T11659/53772

GEOCON INC.

GEOTECHNICAL REPORT

GENERAL CONDITIONS AND LIMITATIONS

A. USE OF THE REPORT

- A.1 The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation or if the project is not initiated within eighteen months of the date of the report Geocon should be given an opportunity to confirm that the recommendations are still valid.
- A.2 The comments given in this report are intended only for the guidance of the design engineer. The number of test holes to determine all the relevant underground conditions which may affect construction costs, techniques and equipment choice, scheduling and sequence of operations would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual test hole data, as to how subsurface conditions may affect their work.

B. FOLLOW-UP

- B.1 All details of the design and proposed construction may not be known at the time of submission of Geocon's report. It is recommended that Geocon be retained during the final design stage to review the design drawings and specifications related to foundations, earthworks, retaining systems and drainage, to determine that they are consistent with the intent of Geocon's report.
- B.2 Retention of Geocon during construction is recommended to confirm and document that the subsurface conditions throughout the site do not materially differ from those given in Geocon's report and to confirm and document that construction activities did not adversely affect the design intent of Geocon's recommendations.

C. SOIL AND ROCK CONDITIONS

- C.1 Soils and rock descriptions in this report are based on commonly accepted methods of classification and identification employed in professional geotechnical practice. Classification and identification of soil and rock involves judgement and Geocon does not guarantee descriptions as exact, but infers accuracy only to the extent that is common in current geotechnical practice.
- C.2 The soils and rock conditions described in this report are those observed at the time of the study. Unless otherwise noted, those conditions form the basis of the recommendations in the report. The condition of the soil and rock may be significantly altered by construction activities (traffic, excavation, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil and rock must be protected from these changes or disturbances during construction.

D. LOGS OF TEST HOLES AND SUBSURFACE INTERPRETATIONS

- D.1 Soil and rock formations are variable to a greater or lesser extent. The test hole logs indicate the approximate subsurface conditions only at the locations of the test holes. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of boring, the frequency of sampling, the method of sampling and the uniformity of subsurface conditions. The spacing of test holes, frequency of sampling and type of boring also reflect budget and schedule considerations.
- D.2 Subsurface conditions between test holes are inferred and may vary significantly from conditions encountered at the test holes.

- D.3 Groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities on the site or adjacent sites.

E. CHANGED CONDITIONS

- E.1 Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the use or reliance by the client of this report that Geocon is notified of the changes and provided with an opportunity to review the recommendation of this report. Recognition of changed soil and rock conditions requires experience and it is recommended that an experienced geotechnical engineer be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

F. DRAINAGE

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage can have serious consequences. Geocon can take no responsibility for the effects of drainage unless Geocon is specifically involved in the detailed design and follow-up site services during construction of the system.

APPENDIX A

Boring Logs

**Figures 1 to 3 - Portable Dynamic Cone
Penetration Test Results**

Table A1 - Summary of Augerholes

EXPLANATION OF THE FORM BORING LOG

This form summarizes both field information and selected laboratory test results obtained from each boring. An explanation of the various columns of the form follows.

DEPTH

This column gives the depth scale of the boring.

ELEVATION AND DEPTH

This column gives the elevation and depth of inferred geologic contacts. The elevation is referred to the datum shown in the general heading.

DESCRIPTION

This column gives a description of the soil based on visual examination of the samples and laboratory tests. Each stratum is described according to the following classification and terminology:

<u>Classification*</u>	<u>Particle Size</u>	<u>Particle Size or Sieve No. (U.S. Standard)</u>
Clay	less than 0.002 mm	less than 0.002 mm
Silt	from 0.002 to 0.075 mm	from 0.002 mm to #200 sieve
Sand	from 0.075 to 4.75 mm	from #200 sieve to #4 sieve
Gravel	from 4.75 mm to 75 mm	from #4 sieve to 3 in.
Cobbles	from 75 to 200 mm	from 3 in. to 8 in.
Boulders	larger than 200 mm	over 8 in.

<u>Terminology</u>	<u>Proportion</u>
Trace, or occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

*Unified Soil Classification System (ASTM D2487-75).

The relative density of cohesionless soils and the consistency of cohesive soils are defined by the following:

<u>Relative Density</u>	<u>Penetration Resistance "N" Blows 0.3 m or Blows foot</u>	<u>Consistency</u>	<u>Undrained Shear Strength**</u>	
			<u>kPa</u>	<u>psf</u>
Very loose	0 to 4	Very soft	0 to 12	0 to 250
Loose	4 to 10	Soft	12 to 25	250 to 500
Compact	10 to 30	Firm	25 to 50	500 to 1000
Dense	30 to 50	Stiff	50 to 100	1000 to 2000
Very dense	over 50	Very stiff	100 to 200	2000 to 4000
		Hard	over 200	over 4000

** The compressive strength obtained from the quick (Q) triaxial test is equal to twice the shear strength of the clay.

SYMBOL

These standard symbols describe the stratigraphy of the soil and rock strata.

(Continued on reverse)

EXPLANATION OF THE TERM

ROCK QUALITY DESIGNATION (RQD)

The description of bedrock quality for engineering purposes can be inferred from a modified core recovery logging procedure designated as RQD, developed by D.U. Deere.* This classification is based on a modified diamond drill core recovery percentage in which only the pieces of sound core over 4 inches (10 cm) long are counted as recovery. The core must be carefully examined to discount fresh irregular breaks caused by the drilling process (fresh broken pieces are fitted together and counted as one piece). The remaining fragments less than 4 inches (10 cm) length are considered to be due to very close bedding, jointing, fracturing, shearing, or weathering in the rock mass and are not counted. The procedure penalizes the rock where recovery is poor. This is appropriate because poor core recovery usually depicts poor quality rock. In the case of certain shaley sedimentary or thinly foliated metamorphic rocks, the method is not as exact as for other rock types and rock quality requires interpretation by a specialist for the particular engineering application. To minimize the occurrence of core breaks from drilling procedures RQD logging is normally run on core obtained by double or triple tube core barrels and generally of "N" size or greater.

The table below may be used as a general indicator to correlate (RQD) and rock mass quality.

RQD	DESCRIPTION OF ROCK QUALITY
90 - 100	Excellent - intact, very sound, massive
75 - 90	Good - moderately jointed or sound
50 - 75	Fair - blocky and seamy, fractured
25 - 50	Poor - shattered and very seamy or blocky, severely fractured
0 - 25	Very poor - crushed, very severely fractured

*See, for instance:

K.G. Stagg and O.C. Zienkiewicz, "Rock Mechanics in Engineering Practice". New York, Wiley, 1968, Chapter I.

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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 1
Bridge, Mississauga PAGE 1 OF 1
 CONTRACT NO. T11659/53772 BORING DATE May 9/90
 DATUM Geodetic CASING HSA

SAMPLE CONDITION



SAMPLE TYPES	
SS	SPLIT SPOON
ST	THIN WALLED OPEN (SHELBY)
PS	PISTON SAMPLER
WS	WASH SAMPLE
RC	ROCK CORE

BU - BULK
AS - AUGER
T - SPLIT TUBE

ABBREVIATIONS

GS	GRAIN SIZE ANALYSIS	K	PERMEABILITY (cm/s)
Y	WET UNIT WEIGHT (kN/m ³)	DS	DIRECT SHEAR
C	CONSOLIDATION	Q	TRIAXIAL QUICK
P.P.-	POCKET PENETROMETER	ROQ	ROCK QUALITY DESIGNATION

STRATIGRAPHY

TESTS

SAMPLES

[illegible]



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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 2
 Bridge, Mississauga PAGE 1 OF 1
 CONTRACT NO. T11659/53772 BORING DATE May 9/90
 DATUM Geodetic CASING HSA

SAMPLE CONDITION



SAMPLE TYPES
 SS - SPLIT SPOON
 ST - THIN WALLED OPEN (SHELBY)
 PS - PISTON SAMPLER
 WS - WASH SAMPLE
 RC - ROCK CORE

BU - BULK
 AS - AUGER
 T - SPLIT TUBE

ABBREVIATIONS

GS - GRAIN SIZE ANALYSIS
 Y - WET UNIT WEIGHT kN/m^3
 C - CONSOLIDATION
 P.P. - POCKET PENETROMETER IAPL
 K - PERMEABILITY cm/s
 DS - DIRECT SHEAR
 Q - TRIAXIAL QUICK
 RQD - ROCK QUALITY DESIGNATION

STRATIGRAPHY

TESTS

SAMPLES

DEPTH m	ELEVATION m	DEPTH m	DESCRIPTION	SYMBOL	WATER LEVEL	UNDRAINED SHEAR STRENGTH kPa	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION - N BLOWS/0.3m
						O TEST FIELD VANE INTACT REMOULDED LAB VANE INTACT REMOULDED WATER CONTENT - W% LIQUID LIMIT - W _L % PLASTIC LIMIT - W _P % DYNAMIC PENETRATION TEST - BLOWS/0.3 m X-X-X 20 40 60 80						
0	180.83	0.00	Ground Surface									
		180.13	Compact, greyish white gravelly sand (possible Fill).						SS 1	1		11
1		0.70	Hard, greyish brown to brown silty clay Till. Some sand and gravel. Occasional oxidized layers. Shale fragments below 1.85 m.						SS 2	63		10
2									SS 3	63		30
									SS 4	100		38
3	171.70	3.05	Highly weathered, brownish red Shale.						SS 5	100		65
	171.32	3.51							SS 6	100		78
4			End of Borehole Auger Refusal									

W.L. IN STANDPIPE @ EL. 180.00 ON MAY 29/90

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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 3
Bridge, Mississauga PAGE 1 OF 2
CONTRACT NO. T11659/53772 BORING DATE May 9/90
DATUM T11659/53772 CASING HSA

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
	DISTURBED	SS - SPLIT SPOON	BU - BULK	GS - GRAIN SIZE ANALYSIS	K - PERMEABILITY cm/s
	GOOD	ST - THIN WALLED OPEN (SHELBY)	AS - AUGER	7 - WET UNIT WEIGHT - kN/m ³	DS - DIRECT SHEAR
	LOST	WS - WASH SAMPLE	T - SPLIT TUBE	C - CONSOLIDATION	Q - TRIAXIAL QUICK
		RC - ROCK CORE			RQD - ROCK QUALITY DESIGNATION
STRATIGRAPHY		TESTS		SAMPLES	
DEPTH E	ELEVATION DEPTH E	DESCRIPTION	SYMBOL	WATER LEVEL	UNDRAINED SHEAR STRENGTH kPa
					Q TEST ● FIELD VANE ◇ INTACT ◆ REMOULDED LAB VANE ▽ INTACT ▼ REMOULDED
					WATER CONTENT - W% DYNAMIC PENETRATION TEST - BLOWS/0.3 m X-X-X 20 40 60 80
					LIQUID LIMIT - W _L % PLASTIC LIMIT - W _P %
0	86.60	Ground Surface			
0	0.00	Firm, greyish brown mixture of clayey silt till and weathered shale. Some roots (Fill)			
1	79.45				
1	1.15	Very stiff to hard, brown, silty clay till, some sand and gravel.			
2	78.77				
2	1.83	Very dense, reddish brown silt Till. Some sand and gravel. Trace clay and shale fragments.			
3					
4	177.00				
4	3.60	Bedrock See Rock Boring Log			
5					
6					
7	173.86				
7	6.74	End of Borehole			



ROCK BORING LOG

PROJECT Highway 401 and Derry Road East BORING 3
Bridge, Mississauga PAGE 2 OF 2
CONTRACT NO. T11659/53772 BORING DATE May 9, 1990
DATUM Geodetic CASING NQ

SAMPLE TYPES

RC - ROCK CORE
(INCH) - SIZE OF CORE

ABBREVIATIONS

SCR - SOLID CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION
DS - DIRECT SHEAR
T - TRIAXIAL TEST
UCS - UNCONFINED COMPRESSION STRENGTH
S.G. - SPECIFIC GRAVITY
γ - UNIT WEIGHT kN/m³

JOINT SPACES

SPACING	< 50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MODERATE	WIDE	VERY WIDE

STRATIGRAPHY

ROCK CHARACTERISTICS

[illegible]



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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 4
Bridge, Mississauga PAGE 1 OF 2
CONTRACT NO. T11659/53772 BORING DATE May 9/90
DATUM Geodetic CASING HSA

SAMPLE CONDITION



SAMPLE TYPES

SS - SPLIT SPOON
ST - THIN WALLED OPEN (SHELBY)
PS - PISTON SAMPLER
WS - WASH SAMPLE
RC - ROCK CORE
BU - BULK
AS - AUGER SAMPLE
T - SPLIT TUBE

ABBREVIATIONS

GS - GRAIN SIZE ANALYSIS
Y - WET UNIT WEIGHT - kN/m³
C - CONSOLIDATION
PP - POCKET PENETROMETER
K - PERMEABILITY - cm/s
DS - DIRECT SHEAR
Q - TRIAXIAL QUICK
RQD - ROCK QUALITY DESIGNATION

STRATIGRAPHY

TESTS

SAMPLES

DEPTH	ELEVATION DEPTH	DESCRIPTION	SYMBOL	WATER LEVEL	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION - N BLOWS/0.3m
0	181.09	Ground Surface								
1	0.00 180.48 0.61	Loose, grey sandy silt and clay. Some organics and wood chips (Fill). Compact, brown to blackish brown silt, some sand, trace clay and organics. Cohesive below 1.2 m. (possible Fill)								
2	179.09 2.00	Dense to very dense, reddish brown, sandy silt Till. Trace to some gravel. Trace to some Clay. 150 mm thick sand layer at 5.3 m.								
3		Some weathered shale fragments at bottom of layer.								
4										
5										
6										
7	174.04 7.05	Bedrock (see Rock Boring Log and Borehole 4A).								
8										
9										
10	171.48 9.61	End of Borehole -								

W.L. IN STANDPIPE @ EL. 180.00m ON MAY 29/90

SEE ROCK BORING LOG

ROCK BORING LOG

PROJECT Highway 401 and Derry Road East BORING 4
Bridge, Mississauga PAGE 2 OF 2
CONTRACT NO. T11659/53772 BORING DATE May 14, 1990
DATUM Geodetic CASING NQ

SAMPLE TYPES
RC - ROCK CORE
(NXL) - SIZE OF CORE

ABBREVIATIONS

SCN	-	SOLID CORE RECOVERY
ROD	-	ROCK QUALITY DESIGNATION
DS	-	DIRECT SHEAR
T	-	TRIAXIAL TEST
UCS	-	UNCONFINED COMPRESSION STRENGTH
S.G.	-	SPECIFIC GRAVITY
γ	-	UNIT WEIGHT kN/m^3

JOINT SENIORS

SPACING	< 50mm	50 - 100mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MODERATE	WIDE	VERY WIDE

STRATIGRAPHY

ROCK CHARACTERISTICS

[illegible]



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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 4A
Bridge, Mississauga PAGE 1 OF 1
CONTRACT NO. T11659/53772 BORING DATE May 15/90
DATUM Geodetic CASING HSA

SAMPLE CONDITION



SAMPLE TYPES

SS - SPLIT SPOON
ST - THIN WALLED OPEN (SHELBY)
PS - PISTON SAMPLER
WS - WASH SAMPLE
RC - ROCK CORE
BU - BULK
AS - AUGER
T - SPLIT TUBE

ABBREVIATIONS

GS - GRAIN SIZE ANALYSIS
γ - WET UNIT WEIGHT kN/m³
C - CONSOLIDATION
K - PERMEABILITY - cm/s
DS - DIRECT SHEAR
Q - TRIAXIAL QUICK
RQD - ROCK QUALITY DESIGNATION

STRATIGRAPHY

TESTS

SAMPLES

DEPTH - m	ELEVATION - m	DEPTH - m	DESCRIPTION	SYMBOL	WATER LEVEL	Q TEST	UNDRAINED SHEAR STRENGTH - kPa	LAB VANE	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION - N BLOWS/0.3m
0	181.04	0.00	Ground Surface											
1			See Borehole 4 for description of soil stratigraphy to 6.10 m											
2														
3														
4														
5														
6	174.94	6.10	Very dense reddish brown sandy silt Till. Some weathered, brownish red, shale.											
7	174.04	7.00	Weathered brownish red shale with grey shale interbeds.											
7	173.57	7.47	End of Borehole											
8														

SS 1 75 >100
SS 2 100 100/100mm
SS 3 50 1100/150mm

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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 5
Bridge, Mississauga PAGE 1 OF 1
 CONTRACT NO. T11659/53772 BORING DATE May 14/90
 DATUM Geodetic CASING HSA

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS			
	DISTURBED	SS - SPLIT SPOON	BU - BULK	GS - GRAIN SIZE ANALYSIS	K - PERMEABILITY - cm/s		
	GOOD	ST - THIN WALLED OPEN (SHELBY)	AS - AUGER	γ - WET UNIT WEIGHT - kN/m³	DS - DIRECT SHEAR		
	LOST	WS - WASH SAMPLE	T - SPLIT TUBE	C - CONSOLIDATION	Q - TRIAXIAL QUICK		
		RC - ROCK CORE		AP - POCKET PENETROMETER	RQD - ROCK QUALITY DESIGNATION		
STRATIGRAPHY				TESTS			
DEPTH E ELEVATION F DEPTH F	DESCRIPTION	SYMBOL	WATER LEVEL	UNDRAINED SHEAR STRENGTH kPa			OTHER TESTS
				Q TEST	FIELD VANE	LAB VANE	
				* P.P.	INTACT REMOULDED	INTACT REMOULDED	
				WATER CONTENT - W%	LIQUID LIMIT - WL%	PLASTIC LIMIT - WP%	
				DYNAMIC PENETRATION TEST - BLOWS/0.3 m X-X-X-X			
				20	40	60	80
0	180.15						
0.00	Loose, grey sand and gravel, traces of clay below 0.15 m (possible Fill)						
179.54							
0.61	Hard, brown silty clay Till. Some sand and gravel.						
1							
2							
3							
4	176.34						
3.81	Very dense, reddish brown silty sand Till. Some gravel.						
5							
6							
7							
72.89	Some shale fragments at bottom of layer.						
7.26	End of Borehole						
8	Auger Refusal						



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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 6
Bridge, Mississauga PAGE 1 OF 1
 CONTRACT NO. T11659/53772 BORING DATE May 11/90
 DATUM Geodetic CASING HSA

SAMPLE CONDITION



SAMPLE TYPES

SS - SPLIT SPOON
 ST - THIN WALLED OPEN (SHELBY)
 PS - PISTON SAMPLER
 WS - WASH SAMPLE
 RC - ROCK CORE
 BU - BULK
 AS - AUGER SAMPLE
 T - SPLIT TUBE

ABBREVIATIONS

GS - GRAIN SIZE ANALYSIS
 Y - WET UNIT WEIGHT - kN/m³
 C - CONSOLIDATION
 P.P. - POCKET PENETROMETER
 K - PERMEABILITY - cm/s
 DS - DIRECT SHEAR
 Q - TRIAXIAL, QUICK
 RQD - ROCK QUALITY DESIGNATION

STRATIGRAPHY

TESTS

SAMPLES

DEPTH E	ELEVATION M DEPTH M	DESCRIPTION	SYMBOL	WATER LEVEL	Q TEST	UNDRAINED SHEAR STRENGTH kPa FIELD VANE INTACT REMOULDED	LAB VANE INTACT REMOULDED	OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION N BLOWS/0.3m
0	179.79	Ground surface											
	0.00	Loose, greyish brown sand and gravel. (Fill)			⊙						SS 1	75	7
1	179.18	Hard, greyish brown to brown silty clay Till. Some sand and gravel.									SS 2	67	34
2	0.61										SS 3	92	44
3		Higher sand content below 2.45 m.									SS 4	67	75
4		Some boulders at 3.0 - 3.6 m.									SS 5	58	7100/250mm
5	175.22										SS 6	17	100/125mm
6	4.57	Very dense, reddish brown silty sand Till. Some gravel.									SS 7	67	7100
7		Some weathered shale fragments toward bottom of layer.									SS 8	100	7100
8											SS 9	100	7100
9											SS 10	0	75/75mm
10	171.79	End of Borehole									SS 11	83	7100/250mm
	8.00	Auger Refusal											

W.L. IN STAND PIPE @ EL. 179.65 ON MAY 29/90



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BORING LOG

PROJECT Highway 401 and Derry Road East BORING 7
Bridge, Mississauga PAGE 1 OF 1
CONTRACT NO. T11659/53772 BORING DATE May 15/90
DATUM Geodetic CASING SSA

SAMPLE CONDITION



SAMPLE TYPES

SS - SPLIT SPOON
ST - THIN WALLED OPEN (SHELBY)
PS - PISTON SAMPLER
WS - WASH SAMPLE
RC - ROCK CORE
BU - BULK
AS - AUGER SAMPLE
T - SPLIT TUBE

ABBREVIATIONS

GS - GRAIN SIZE ANALYSIS
W - WET UNIT WEIGHT LN/m^3
C - CONSOLIDATION
K - PERMEABILITY cm/s
DS - DIRECT SHEAR
Q - TRIAXIAL QUICK
RQD - ROCK QUALITY DESIGNATION

STRATIGRAPHY

TESTS

SAMPLES

DEPTH ELEVATION DEPTH	DESCRIPTION	SYMBOL	WATER LEVEL	UNDRAINED SHEAR STRENGTH KPa				OTHER TESTS	CONDITION	TYPE	NUMBER	RECOVERY %	STANDARD PENETRATION BLOWS/30
				Q TEST	FIELD VANE	LAB VANE							
				<input type="checkbox"/> WATER CONTENT $\text{W}\%$	<input type="checkbox"/> LIQUID LIMIT $\text{W}\%$	<input type="checkbox"/> PLASTIC LIMIT $\text{W}\%$							
				DYNAMIC PENETRATION TEST BLOWS/0.3 m X - X - X									
				20	40	60	80						
0.00	Ground Surface												
0.43	Brownish black, sandy silt with organics. (Topsoil)									AS	1	—	—
0.46	Greyish black to grey silt. Some sand and gravel Trace roots and organics. (Fill)									AS	2	—	—
1.79										AS	3	—	—
2.43	Grey silt Till. Some sand and gravel. Trace clay.									AS	4	—	—
3.53	End of Borehole									AS	5	—	—

Geocon

BORING LOG

PROJECT Highway 401 and Derry Road East BORING 8
Bridge, Mississauga PAGE 1 OF 1
CONTRACT NO. T11659/53772 BORING DATE May 15/90
DATUM Geodetic CASING SSA

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
	DISTURBED	SS - SPLIT SPOON	BU - BULK	GS - GRAIN SIZE ANALYSIS	K - PERMEABILITY cm/s
	GOOD	ST - THIN WALLED OPEN (SHELBY)	AS - AUGER SAMPLE	Y - WET UNIT WEIGHT kN/m³	DS - DIRECT SHEAR
	LOST	WS - WASH SAMPLE	T - SPLIT TUBE	C - CONSOLIDATION	Q - TRIAXIAL QUICK
		RC - ROCK CORE			ROD - ROCK QUALITY DESIGNATION
STRATIGRAPHY		TESTS		SAMPLES	
DEPTH m	ELEVATION m	DESCRIPTION	SYMBOL	WATER LEVEL	OTHER TESTS
					CONDITION
					TYPE
					NUMBER
					RECOVERY %
					STANDARD PENETRATION 2 BLOWS/30 cm
0	181.71	Ground surface			
0.00	181.40	Topsoil			AS 1
0.31		Blackish grey sandy silt, some gravel, trace clay. (Fill). Topsoil and peat between 1.98 and 2.59 m.			AS 2
1					AS 3
2					AS 4
179.12	2.59	Blackish grey to grey sandy silt Till. Some gravel.			AS 5
3	178.66				AS 6
3.05		End of Borehole			



BORING LOG

PROJECT Highway 401 and Derry Road East BORING 9
Bridge, Mississauga PAGE 1 OF 1
 CONTRACT NO. T11659/53772 BORING DATE May 15/90
 DATUM Geodetic CASING SSA

SAMPLE CONDITION



SAMPLE TYPES	
SS -	SPLIT SPOON
ST -	THIN WALLED OPEN (SHELBY)
PS -	PISTON SAMPLER
WS -	WASH SAMPLE
RC -	ROCK CORE

BU - BULK
AS - AUGER
T - SPLIT TUBE

ABBREVIATIONS

GS	GRAIN SIZE ANALYSIS	K	PERMEABILITY	cm/s
Y	WET UNIT WEIGHT - kN/m ³	DS	DIRECT SHEAR	
C	CONSOLIDATION	Q	TRIAXIAL, QUICK	
		ROD	- ROCK QUALITY DESIGNATION	

STRATIGRAPHY

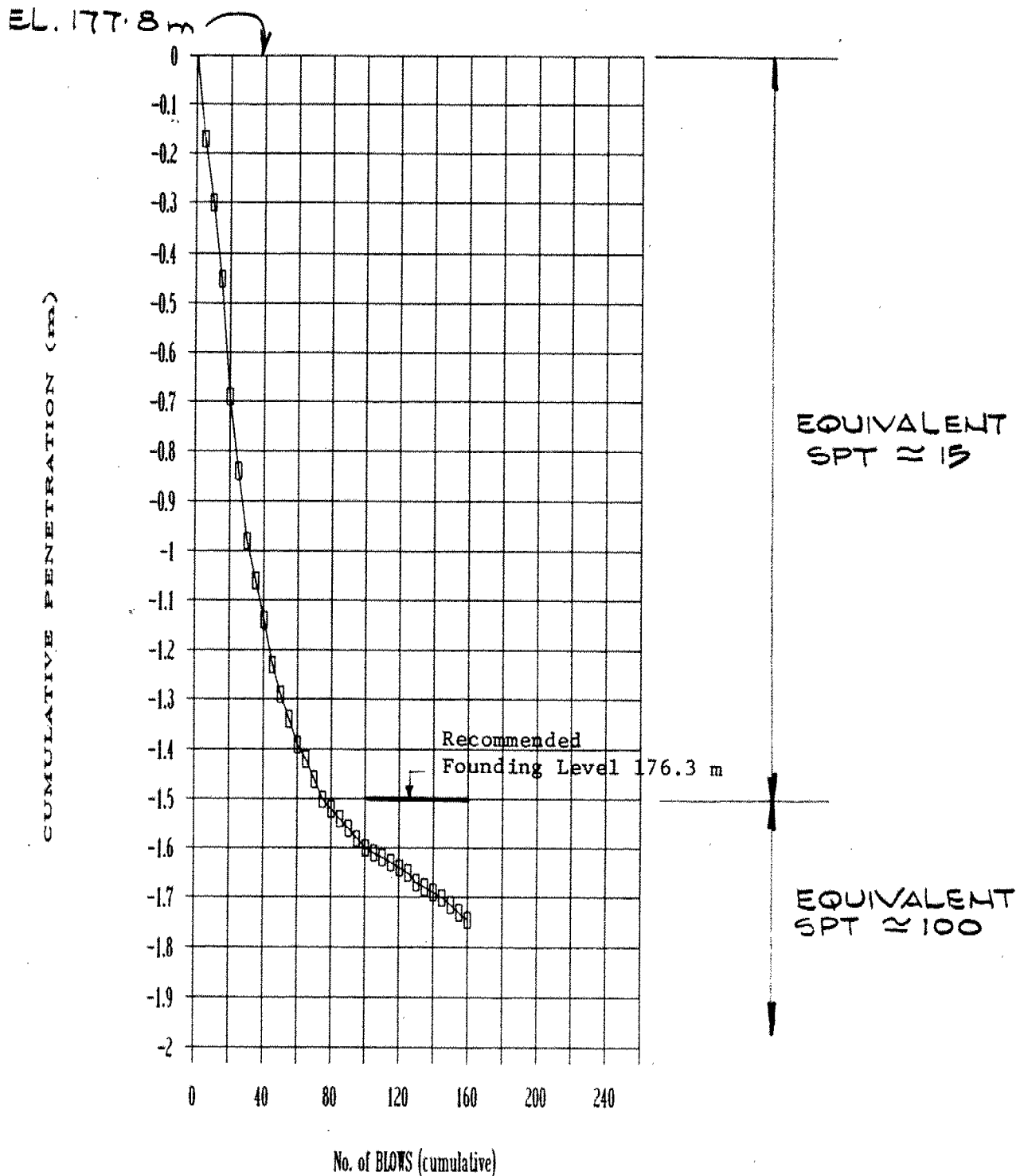
TESTS

SAMPLES

[illegible]

PORTABLE DYNAMIC CONE PENETRATION TEST #1

APPENDIX A
FIGURE 1
PROJECT T11659

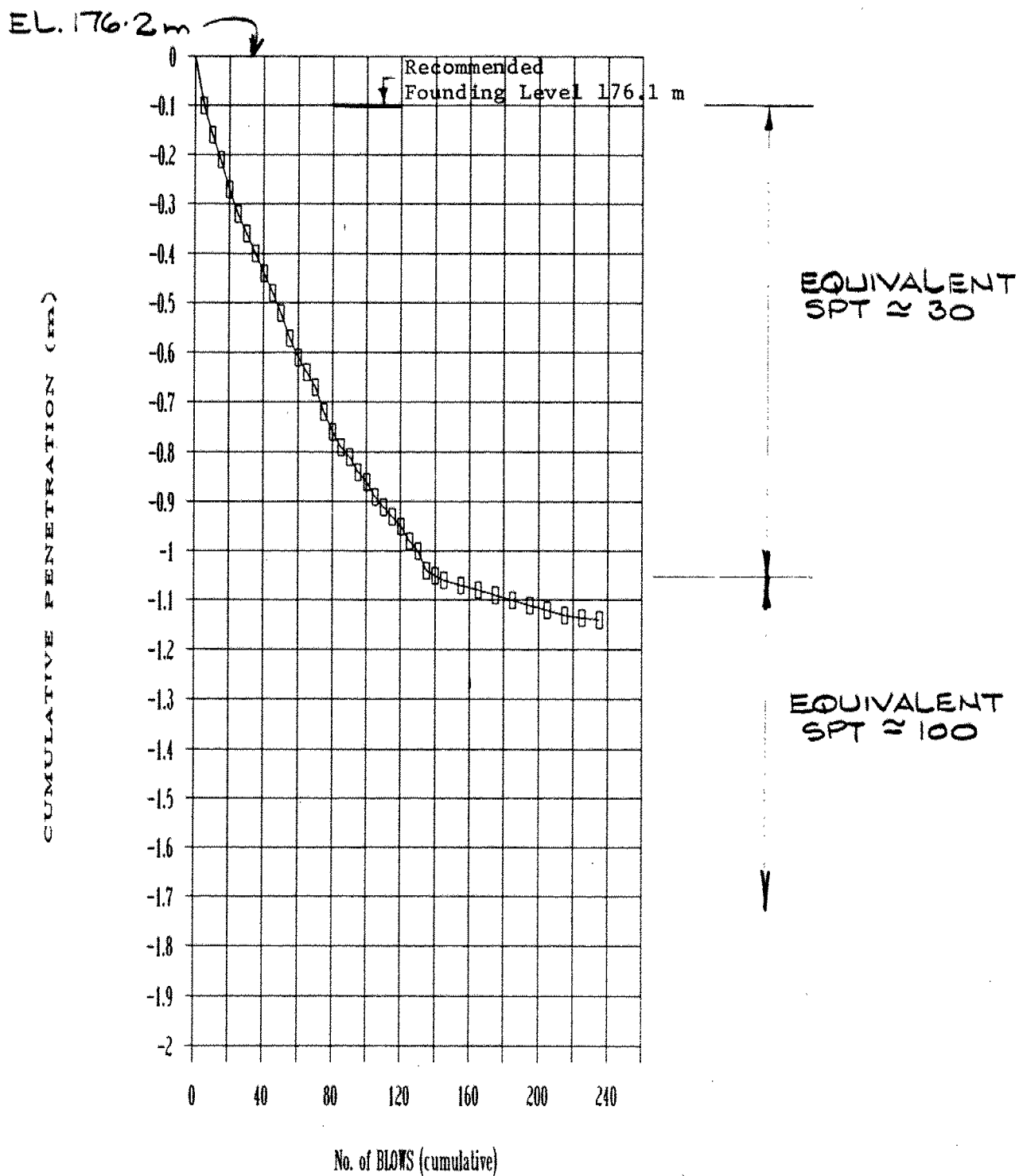


Note: For location of PDCPT 1 refer to Drawing No. T11659-1.

GEOCON

PORTABLE DYNAMIC CONE PENETRATION TEST #2

APPENDIX A
FIGURE 2
PROJECT T11659

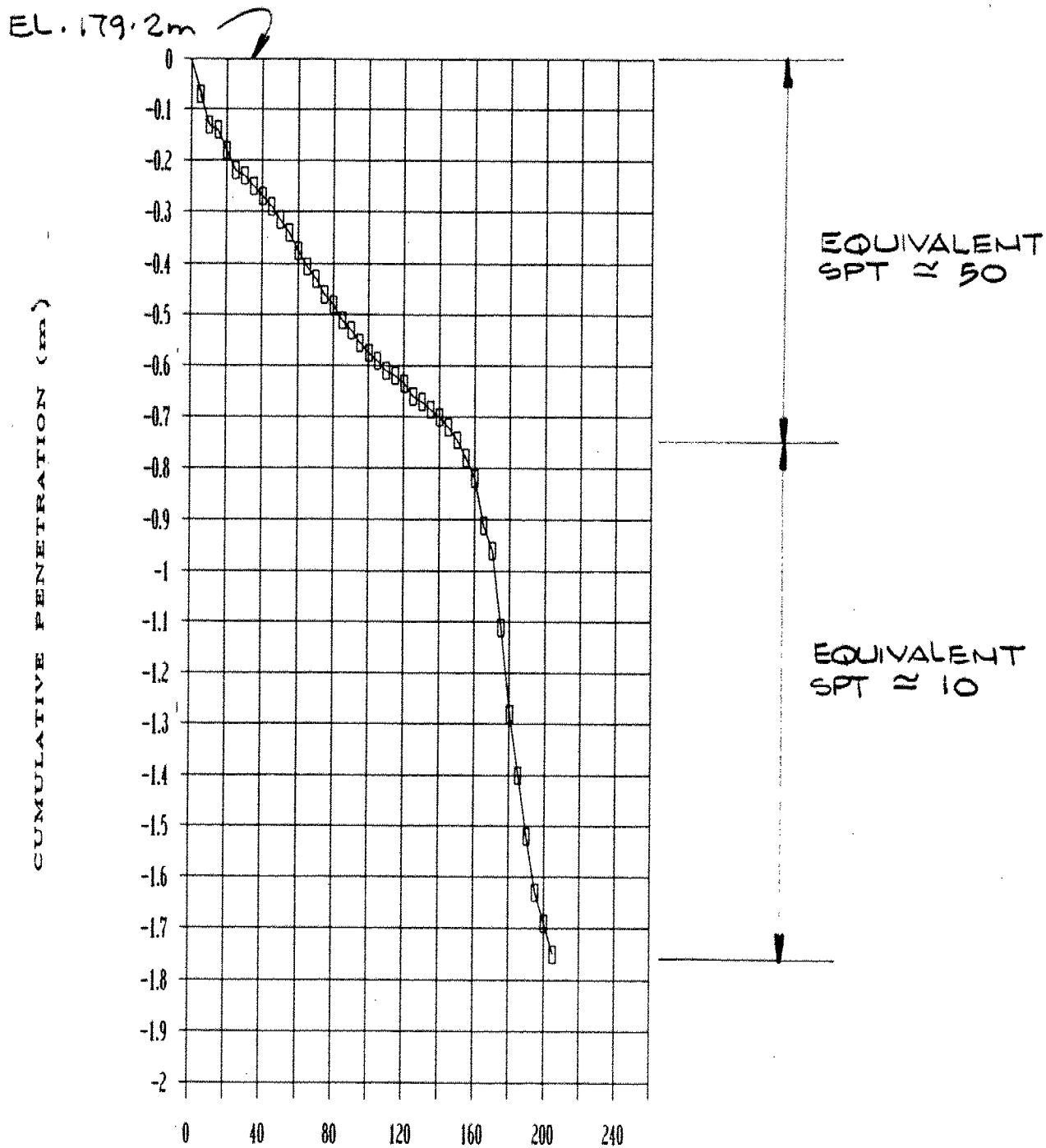


Note: For location of PDCPT 2 refer to Drawing No. T11659-1.

GEOCON

PORTABLE DYNAMIC CONE PENETRATION TEST # 3

APPENDIX A
FIGURE 3
PROJECT T11659



Note: For location of PDCPT 3 refer to Drawing No. T11659-1.

GEOCON

TABLE A1

SUMMARY OF AUGERHOLES

Augerhole 1 - Ground Surface Elevation 176.7 m

0.00 m	- 0.45 m	Cobbles (Gabion Mat)
0.45 m	- 0.50 m	Brown Gravel
0.50 m	- 0.70 m	Blueish grey, silty clay Till
	- 0.70 m	Auger Refusal

Augerhole 2 - Ground Surface Elevation 179.2 m

0.00 m	- 0.70 m	Light brown, sandy silt Till
		Some Gravel
	- 0.70 m	Power Auger Refusal

Augerhole 3 - Ground Surface Elevation 179.6 m

0.00 m	- 0.10 m	Topsoil
0.10 m	- 1.00 m	Light brown, sandy silt Till
		Some Gravel
	- 1.00 m	Refusal to Auger Advance

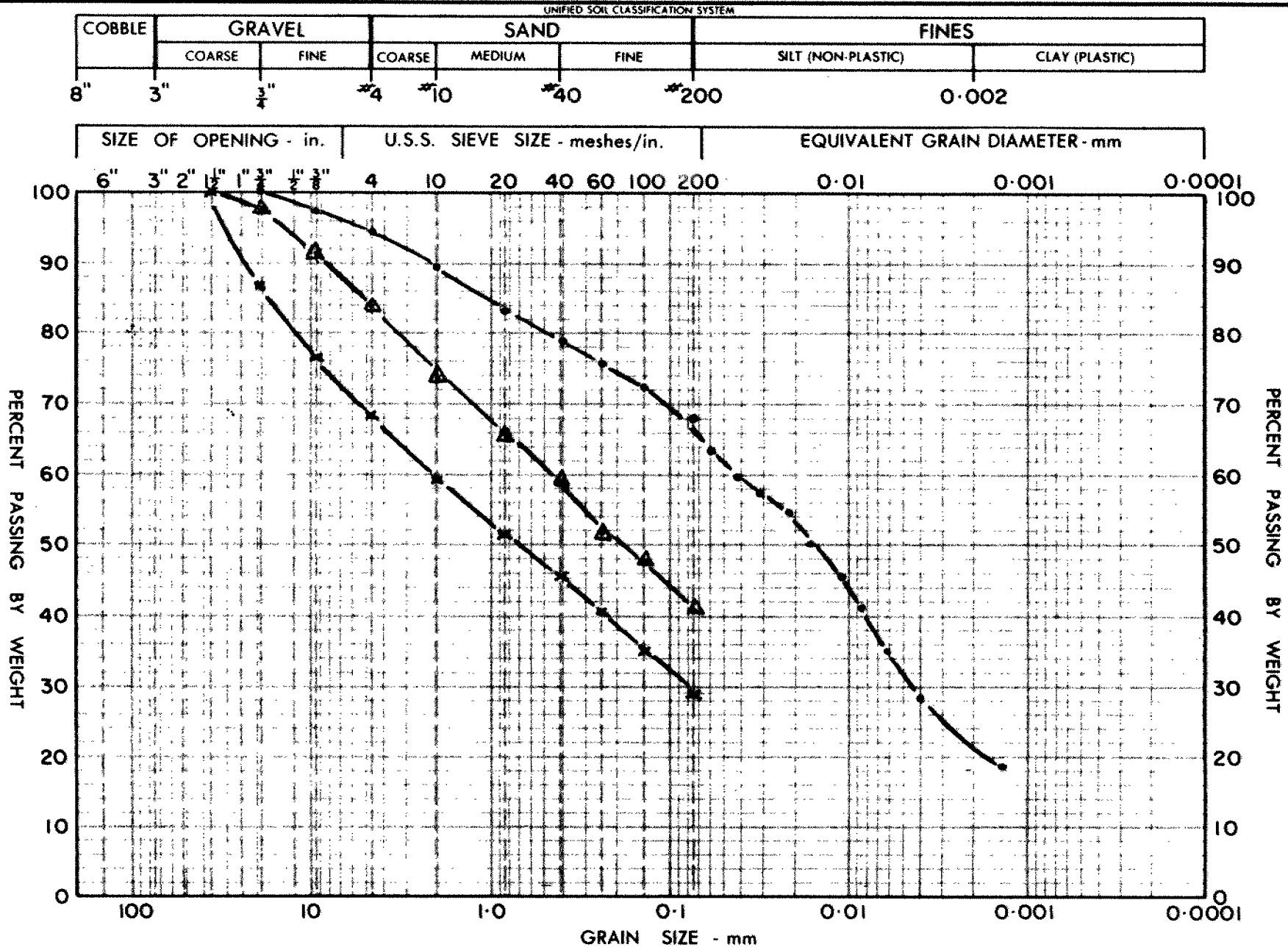
APPENDIX B

Laboratory Test Results



GRAIN SIZE DISTRIBUTION

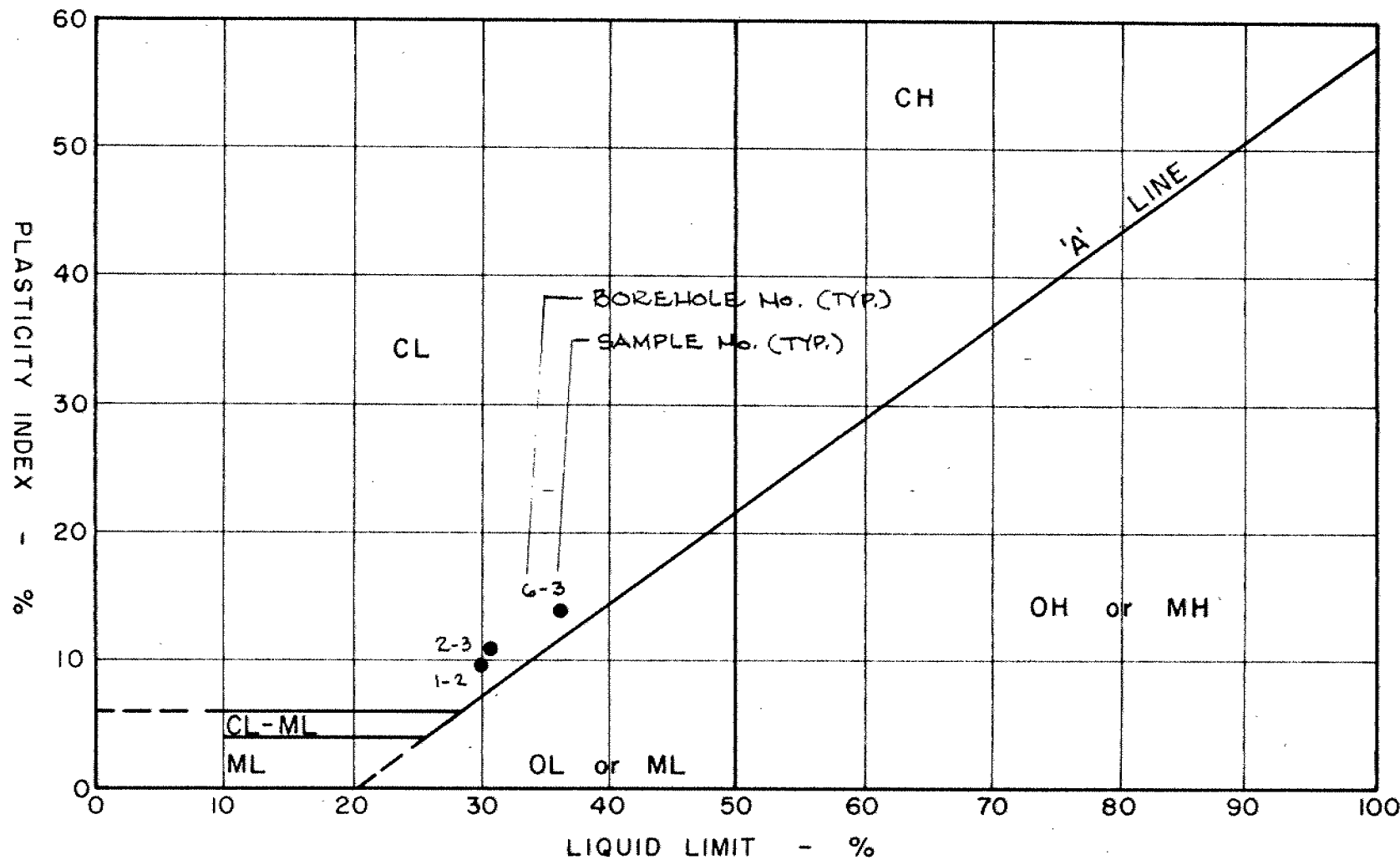
APPENDIX B
FIGURE 1
PROJECT T11659



BORING	SAMPLE	DEPTH (ft,m)	SYMBOL	CLASSIFICATION	BORING	SAMPLE	DEPTH (ft,m)	SYMBOL	CLASSIFICATION
4	S		•	SANDY SILT, some CLAY					
5	B		x	SILTY SAND					
4	B		Δ	SAND AND SILT					

PLASTICITY CHART

APPENDIX B
FIGURE 2
PROJECT T11659



CL - INORGANIC CLAYS
(Low to Med. Plasticity)
CH - INORGANIC CLAYS
(High Plasticity)
ML - INORGANIC SILTS

CL - ML INORGANIC CLAYS AND SILTS
OL or ML ORGANIC OR INORGANIC SILTS
OH or MH ORGANIC CLAYS OR COMPRESSIBLE
INORGANIC SILTS

APPENDIX C

Detailed Subsurface Conditions

1.0 INTRODUCTION

The site of the proposed Derry Road underpass and associated structures was investigated between May 9, 1990 and May 15, 1990. During that time a total of 9 boreholes, 3 augerholes and 3 Portable Dynamic Cone Penetration Tests (PDCPT) were performed at locations shown on Drawing No. T11659-1 (main report). A detailed description of the investigation procedure is presented in Section 4.0 of the main report. Detailed Boring Logs, augerhole information and PDCPT data are presented in Appendix A of the main report with laboratory test data presented in Appendix B (main report). A summary of pertinent borehole information is presented in Table C1 presented at the rear of this appendix.

2.0 BRIDGE UNDERPASS (Boreholes 1 to 6)

2.1 Stratigraphy

Investigated with Boreholes 1 to 6 (Drawing No. T11659-1), the stratigraphy at the site can be described as comprising a layer of fill overlying glacial till which in turn overlies shale bedrock of the Queenston Formation. A description of the geotechnical properties of these layers is presented below. Generalized stratigraphic sections summarizing the encountered soil conditions are presented in Drawing No. T11659-2 (main report).

2.1.1 Fill

Present in all boreholes, this layer varied from a minimum thickness of 0.5 m at Borehole 1 to 2.0 m at Borehole 4. This material is quite variable but generally comprises a mixture of clayey silt, some sand and gravel and some organics. Measured SPT values within this layer range from a low of 2 to a high of 15 but

generally the layer can be considered to be in a loose state of relative density.

Three natural moisture content determinations performed on samples retrieved from this layer indicate values which vary from a low of 8 percent to a high of 32 percent. This quite high variability may be attributed to the variable amount of clay within this layer.

2.1.2 Glacial Till

Glacial till of the Peel Till plain was encountered in all boreholes immediately underneath the layer of surficial fill and overlying the shale bedrock. The thickness of the till layer varies from a minimum of 1.0 m at Borehole 1 to a maximum of 7.4 m at Borehole 6 and generally exhibits a trend of increasing thickness towards the North and East (Drawing No. T11659-2). Based on the observed stratigraphy at the borehole locations, the surface elevation of the till layer is of the order of 180 m at the southern end of the existing underpass and approximately 179 m at the northern end of the proposed underpass. More detailed information on the elevation of the till layer is presented in Table C1.

The till layer in general comprises an upper cohesive silty clay till underlain by a cohesionless, predominantly silty sand till. Both deposits contain varying amounts of sand and gravel. The upper silty clay till extends to a maximum depth of 4.6 m at Borehole 6. The underlying cohesionless till, present in all boreholes except Boreholes 1 and 2, attains maximum thicknesses in the deeper till profiles at the location of Boreholes 4, 5 and 6. Shale fragments are present within the till towards the bottom of the layer adjacent to the underlying shale.

SPT values within the upper cohesive till are generally in excess of 30 indicating that the consistency of this material can be described as hard. The description of consistency is in accordance with estimates of the undrained shear strength of the material as determined using the pocket penetrometer on recovered split spoon samples, which indicate values generally in excess of the range of the penetrometer i.e., in excess of 225 kPa. However, measured SPT values and undrained shear strengths are generally lower in the upper 0.5 m of the cohesive till.

Atterberg Limit determinations performed on samples of the cohesive till retrieved from Boreholes 1, 2 and 6 (see Boring Logs) indicate liquid and plastic limit values of the order of 31 (range 31-36) and 20 (range 20-23) respectively with a concomitant plasticity index of 11 (range 10-14). As shown on Figure B2 (Appendix B - main report) this material can be described as an inorganic (CL) clay. Natural moisture content values of this material range from 11 to 17 percent indicating values which are less than its plastic limit.

Within the cohesionless till, which comprises generally a mixture of silty sand with a trace to some gravel. SPT values are everywhere in excess of 30 and generally in excess of 50 indicating that this material is generally in a very dense state of relative density. Natural moisture contents are low with values ranging from 6 to 12 percent.

2.1.3 Shale

Brownish red shale of the Queenston formation was encountered in all boreholes below the glacial till. In accordance with the general thickening of the till deposit towards the North and East, the elevation of the surface of the shale exhibits a similar trend (Drawing No. T11659-2; Table C1). Based on the encountered

conditions during augering and more specifically on the rock coring at Boreholes 3 and 4, the upper portions of the shale are weathered. The thickness of the weathered zone was observed to be 1.5 m and 0.9 m at Boreholes 3 and 4 respectively. Maximum penetration of the augers into this layer before refusal was 0.5 m at Boreholes 1 and 2. SPT attempts within this layer generally resulted in less than 100 mm penetration for 100 blows (SPT refusal). Based on this information and from visual observations of the recovered core from Boreholes 3 and 4, this material can be described as a very weak rock (UCS 0.6 - 1.25 MPa).

Unweathered shale was confirmed at depths of 5.2 m (elevation 175.4 m) and 8.0 m (elevation 173.1 m) at Boreholes 3 and 4 respectively. Based on the observed rock cores, this material is frequently interbedded with limestone. The maximum thickness of these interbeds observed during the rock coring was 50 mm although thicker layers may be present. Measured RQD values are in the range of 55 to 70%, indicating the rock to be of fair quality. Based on quoted values in the literature, the unconfined compressive strength of the shale is expected to vary between 5 and 15 MPa.

2.2 Groundwater Conditions

Based on measurements of the groundwater level conducted on May 29, 1990 (about 2 weeks after drilling) in standpipe piezometers installed at Boreholes 2 to 6, the ambient groundwater level across the site of the proposed underpass is relatively uniform with elevations varying from 179.6 m to 180.0 m (Table C1). This level is generally less than 1 m below that of the existing ground.

3.0 HIGHWAY EMBANKMENT (Boreholes 7 to 9, Augerholes 2 and 3 and PDCPT 3)

The area of the proposed Highway 401 embankment widening to the west of Derry Road was investigated with Boreholes 7, 8 and 9 with the similar area to east of Derry Road up to the existing Highway 401 culvert being investigated with Augerholes 2 and 3 and PDCPT 3 (Drawing No. T11659-1).

The subsurface conditions under the proposed highway 401 embankment to the west of Derry Road comprises a layer of topsoil (0.3 to 0.5 m) overlying fill overlying glacial till. The fill material comprises predominantly a mixture of sandy silt with some sand and gravel and organics. Further, the layer of fill at Boreholes 8 and 9 is underlain by topsoil. The total thickness of fill and topsoil materials varies from a minimum of 0.6 m at Borehole 9 to 2.6 m at Borehole 8 with corresponding surface elevation of the till of 179.1 and 180.0 m at Boreholes 8 and 9, respectively. However, based on observation of the surface vegetation (bullrushes) a short distance to the North of Boreholes 7, 8 and 9, the ground conditions may comprise slightly deeper soft peat deposits.

To the east of Derry Road, the soil conditions comprise a thin cover of topsoil (less than 0.1 m) overlying glacial till. The elevation of the till surface at Augerholes 2 and 3 is 179.2 m and 179.6 m, respectively. PDCPT 3 (Figure A3) indicates approximately 0.8 m of very dense material (equivalent SPT of 50) overlying compact material (equivalent SPT of the order of 10). This zone of compact material is considered to be quite anomalous and its origin is unknown.

4.0 Highway 401 Culvert (Augerhole 1, PDCPT 1 and 2)

The northern end of the proposed extension of an existing culvert passing underneath Highway 401 was investigated using Augerhole 1

and PDCPT 1 and 2 (Drawing No. T11659-1). Augerhole 1 and PDCPT 2 were put down from the base of an existing gabion lined channel with PDCPT 1 advanced from the bank adjacent to the channel. At Augerhole 1 till was confirmed to underlie the base of the channel at elevation 176.2 m. PDCPT 2 confirms the consistency of the till layer with equivalent SPT values of the order of 30 commencing at elevation 176.3 m. This elevation compares favourably with the results of PDCPT 1. The material of the till deposit was not investigated but is expected to be similar to that observed at Boreholes 4, 5 and 6.

TABLE C1
Highway 401 and Derry Road
Borehole Summary

Borehole No.	Ground Surface Elevation (m)	Total Depth (m)	Stratigraphic Upper Elevation (m) (layer thickness in brackets (m))					Ground Water Information	
			Topsoil	Fill	Till	Weathered Shale	Sound Shale	Tip Elevation (m)	Groundwater Elevation (m)
1	180.96	1.98	- (0.0)	180.96 (0.53)	180.43 (0.99)	179.44 [0.46]			
2	180.83	3.51	- (0.0)	180.83 (0.70)	180.13 (2.35)	177.78 [0.46]		178.08	180.0
3	180.60	6.74	- (0.0)	180.60 (1.15)	179.45 (2.45)	177.00 (1.55)	175.45 [1.59]	174.09	179.9
4	181.09	9.61	- (0.0)	181.09 (2.00)	179.09 (5.05)	174.04 (0.94)	173.10 [1.62]	172.24	180.0
5	180.15	7.26	- (0.0)	180.15 (0.61)	179.54 (6.65)	172.89 [0.0]		172.89	179.8
6	179.79	8.00	- (0.0)	179.79 (0.61)	179.18 (7.39)	171.79 [0.0]		172.16	179.6
7	181.89	3.53	181.89 (0.46)	181.43 (1.97)	179.46 [1.10]				
8	181.71	3.05	181.71 (0.31)	181.40 (2.28)	179.12 [0.46]				
9	180.61	3.05	180.31 (0.31)	180.61* (0.31)	180.00 [2.44]				

Notes:

[] Layer thickness not fully penetrated

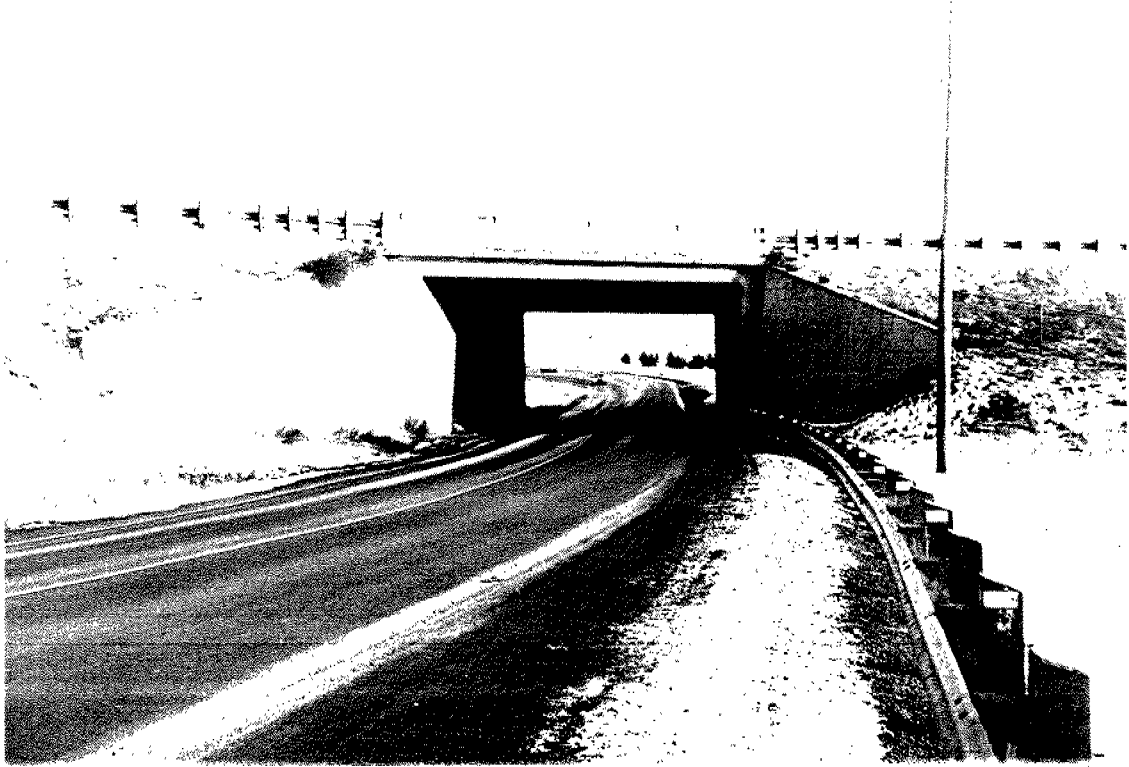
* Fill overlies topsoil at this location

Borehole 4 summary includes information from adjacent Borehole 4A.

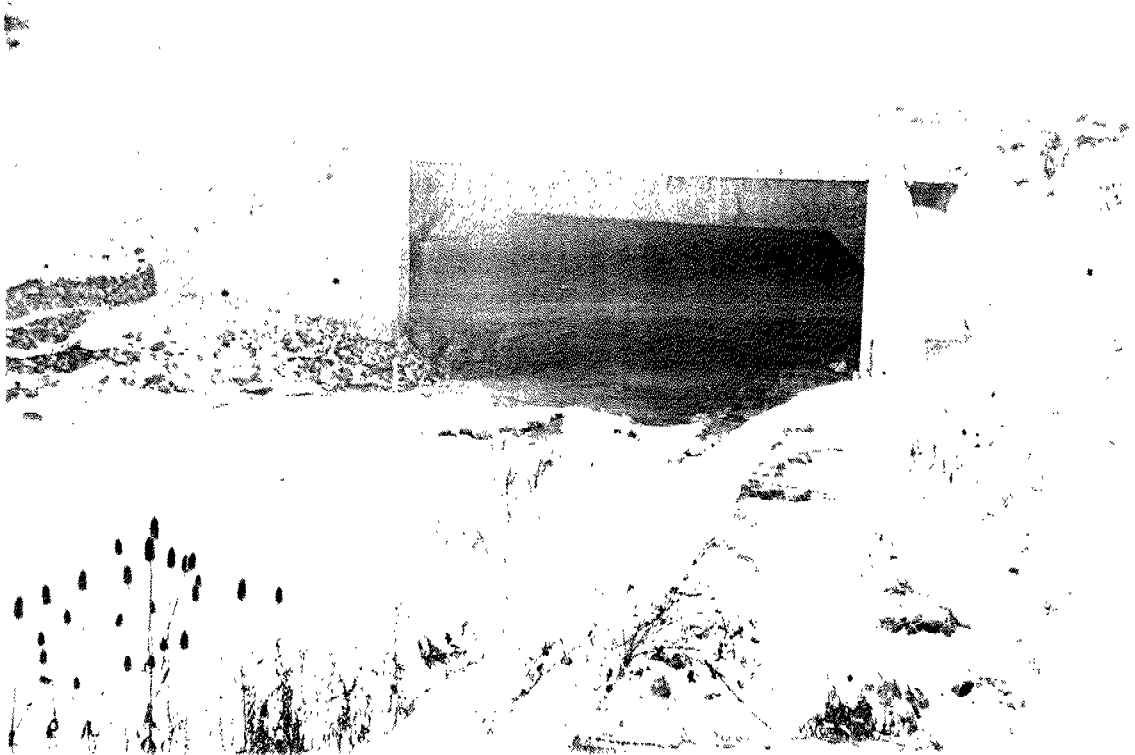
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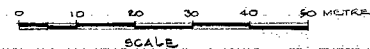
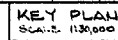
Hwy 401 Derry Road Overpass Structure Replacement
Mullet Creek Culvert, site 24-252
Hwy 401, District 6



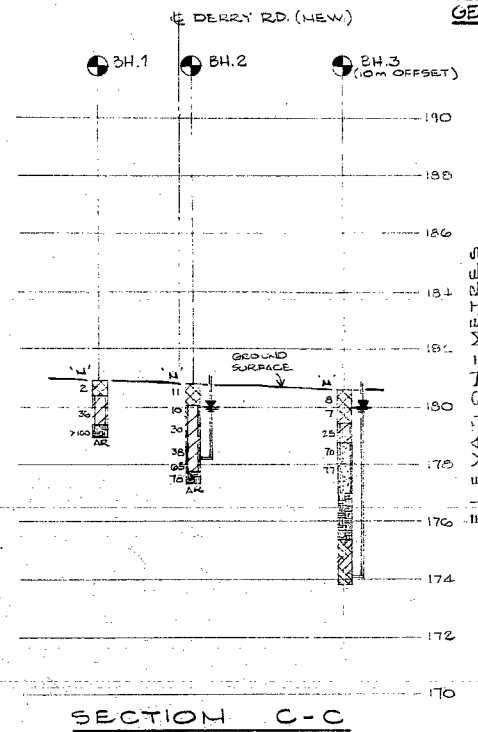
Looking north, Derry Road Overpass



North end of culvert



53772 T11659-1



DATE	SCALE	HORIZ. 1:500 VERT. 1:100	
AUG. 30, 1990			
DRAWN BY	CHECKED BY:	APPROVED BY:	
M.C. ZIDELBERG	IC	R.D.P	
PROJECT NO	DESIGNING NO	REV.	
53772	T11659-2		

SPECIAL NOTES

DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES MAY VARY FROM THAT SHOWN.

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.