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W. O. No.

STR. SITE No.

HWY. No. 417

LOCATION Hintonburg West  
Storm Sewer Tunnel, Ottawa

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

# FOUNDATION INVESTIGATION REPORT

CONTRACT NO 82 - 91



Ministry of  
Transportation and  
Communications



Index

<u>Page No.</u>	<u>Description</u>
1	Index
2	Abbreviations & Symbols
3- 23	Foundation Investigation Report For W.P. 174-80-02, District 9 Proposed Storm Sewer Tunnel in Ottawa (Queensway & Carling Ave.)

NOTE: For purposes of the Contract this report supercedes all other foundation reports done by or for the Ministry in connection with the above-mentioned project.

## EXPLANATION OF TERMS USED IN REPORT

2

**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 (R Q D), 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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$\alpha$	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
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

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

## FOUNDATION INVESTIGATION REPORT

For

The Proposed Storm Sewer Tunnel in Ottawa

Queensway &amp; Carling Ave.

W.P. 174-80-02

Hwy. 417, District 9, OttawaINTRODUCTION:

This report contains the results of the foundation investigation for the proposed 259 metre long section of large diameter (2.1m) storm sewer running from near the intersection of Carling Avenue Westbound and Tweedsmuir Avenue to connect to the existing Cave Creek storm sewer tunnel immediately west of the intersection of Carling Avenue westbound and Kirkwood Avenue.

As the proposed alignment crosses both the Ottawa Queensway and Carling Avenue westbound, present plans call for the sewer to be constructed by means of tunnelling techniques.

The purpose of this investigation was to determine the overburden-bedrock line along the length of the proposed sewer route in order that the sewer may be constructed by tunnelling techniques and secondly, to determine the bedrock parameters as they relate to the design of the tunnel section and lining, including any special construction considerations which could influence design decisions.

The field work was carried out from April 5 to April 7, 1982. The borings were advanced by hollow stem augers and consisted of 7 sampled boreholes and two supplementary probe holes to depths where augers met practical refusal. Standard drive open samples were taken throughout the overburden strata in all 7 sampled holes and bedrock was cored in NXL size to below invert level of the proposed sewer.

An additional borehole No. 28 carried out by Golder Associates on April 1, 1975 for the Hintonburgh West Storm Collector has also been incorporated into this report.

During core drilling, records were kept of drilling water return, core recovery and R.Q.D. Also, in BH#3 in situ pressure packer tests

were carried out to determine the permeability characteristics of the bedrock. Following completion of the field work, laboratory testing was carried out on both the overburden and bedrock samples.

### Site Description

The site stretches from east of the intersection of Carling Avenue and Tweedsmuir Avenue across Hwy. 417 to just west of the intersection of Kirkwood Avenue and Carling Avenue in Ottawa. In the site vicinity, the terrain is flat and classified as urban area.

Physiographically, the site is located within the Ottawa Valley Clay Plains.

### SUBSOIL CONDITIONS

#### General

The subsoil investigation revealed that 1 to 6 metres of fill underlain by a thin layer (0.5 metre) of black organics extends over the entire proposed sewer route. Beneath this fill along the eastern section of the route in boreholes 1, 1A, 2 and 3, a 1 to 1.8 metre thick layer of silty clay with some sand was encountered. Below the silty clay in boreholes 1 and 1A, a 1.6 to 1.8 metre thick layer of silty sand with a trace of gravel and clay was found. Underlying the silty sand in the eastern section of the route and the fill material along the remaining section of the route extends a 2 to 4 metre thick layer of glacial till which in turn is underlain by limestone bedrock.

The boundaries between the various soil types and bedrock, as well as laboratory test results and ground water levels are shown on the attached Record of Borehole Sheets. The locations and elevations of the borings, along the stratigraphical profile are shown on Drawing No. 1748002-A.

The various soil types and the bedrock are described in the following paragraphs.

### Surficial Fill

The surficial fill material encountered across the site beneath 10 to 15 cm. of top soil consists of a loose to compact silty sand with gravel and a trace of clay. This fill varies in thickness from about 1 metre along the route to 6 metres at the Queensway embankment. Standard penetration 'N' values varied from 4 to 51 blows per 0.3 m indicating a very loose to very dense relative density.

The fill is underlain by a thin layer of black-organic material which probably represents the original ground.

### Silty Clay

Underlying the fill in boreholes 1, 1A, 2 and 3 (eastern part of route) is a 1 to 1.8 metre thick deposit of silty clay with some sand and a trace of gravel.

The results of Atterberg Limit and water content testings plotted on the Plasticity Chart Fig. #1, indicate the deposit to be silty clay of low plasticity. The test results are summarized below:

		RANGE
Natural Moisture Content	(W)%	16 - 31
Liquid Limit	(W <sub>l</sub> )%	16 - 27
Plastic Limit	(W <sub>p</sub> )%	10 - 14
Plastic Index	(W <sub>i</sub> )%	6 - 13

Based on interpretation of Standard Penetration Test 'N' Values and augering operations, the consistency of the silty clay deposit varies from very soft to soft.

### Silty Sand

Underlying the silty clay in boreholes 1 and 1A is a 1.6 to 1.8 metre thick deposit of silty sand with a trace of gravel and clay. Grain size analyses carried out on samples of the silty sand recovered from boreholes 1 and 1A are shown on Figure 2.

Based on Standard Penetration Test 'N' values ranging from 11 to 51 blows per 0.3 m, the relative density for this deposit is compact to very dense.

#### Glacial Till

Glacial till occurs below the fill, silty clay and silty sand in all boreholes. This deposit ranges in thickness from 2 to 4 metres and is classified as sandy silt with gravel and a trace of clay. However, it should be noted that the samples as tested were recovered in the standard 51 mm O.D. split spoon sampler and may not reflect the presence of cobbles and boulders within this glacial deposit. The cobble and boulder content was found to increase with depth in the granular till deposit in boreholes 1, 1A, 3 and 8, and it was necessary to employ diamond drilling techniques to advance boreholes through the lower portion of this stratum. The results of grain size distribution testing performed on samples obtained from 51 mm O.D. split spoon sampler from the deposit are plotted on Figure 3.

Based on the Standard Penetration Test 'N' values ranging from 23 to 120 blows per 0.3 m, the relative density of this non-cohesive glacial till deposit may be described as compact to very dense.

#### Bedrock Conditions

The site is underlain by limestone of the Ottawa Formation. Boreholes drilled along the line of the proposed tunnel encountered bedrock at depths ranging from about 5.1 m to 10.5 m below existing ground surface. The bedrock surface which is at its highest elevation (i.e. 72.2 m) in the vicinity of Station 10+054 (Borehole 6) towards the western end of the proposed tunnel line drops off to its lowest measured elevation (i.e. 67.9 m) in the vicinity of Station 10+246.6 (Borehole 1) to the east. Bedrock could be present at lower elevations between boreholes.

Boreholes indicate that the rock is predominantly a grey, fine to medium grained limestone containing dolomitic and shaley zones and partings. Shale is estimated to comprise about 5 percent of the rock



mass and is present mainly as randomly, interbedded partings about 1 to 3 mm thick.

High core recoveries (generally greater than 95 percent) and R.Q.D. (Rock Quality Designation) measurements indicate that the rock mass is in general 'good' (R.Q.D.=75 to 90%) with 'fair' (R.Q.D.=50 to 75%) and occasional poor sections. In most boreholes the upper 0.6 to 1.5 m of the rock mass is generally more intensely fractured than the rock at depth, and is generally 'fair' with 'poor' and 'very poor' sections.

Point load strength tests on selected samples (see Table 1) indicate that the uniaxial compressive strength of the intact rock could vary from about 40 to 182 MPa, average 103 MPa. The relationship between point load and uniaxial compressive strengths is only approximate. The rock mass is intersected mainly by closely to moderately spaced bedding joints and some near vertical crossjoints. In general joint surfaces are rough planar and tight. Occasional zones of very closely spaced, intersecting joints were encountered in the borehole core.

Water pressure tests were carried out in packed-off sections in Borehole 3 at depths of 6.1 to 7.6 m and 7.6 m to 9.1 m below the existing ground surface in order to assess the permeability of the rock mass. The rock mass immediately above the proposed tunnel crown at Borehole 3 (i.e. the 7.6 to 9.1 m test section), has a mass permeability in the order of  $2 \text{ to } 5 \times 10^{-4} \text{ cm/sec}$ . The rock mass in the vicinity of the upper test section i.e. 6.1 to 7.6 m below existing ground surface has a permeability in the order of about  $8 \times 10^{-4} \text{ to } 1 \times 10^{-3} \text{ cm/sec}$ . Packer tests undertaken previously by Golder Associates for the City of Ottawa in Borehole 28 at the eastern end of the proposed tunnel line were conducted in relatively sound limestone. The permeability of the rock mass at depths of between 10.3 m and 18.6 m was  $1 \times 10^{-6} \text{ cm/sec}$ . It is anticipated that the permeability of the rock mass in the vicinity of Stations 10+246.6 and 10+224.7 (Borehole 1 and 1A) could be up to an order greater (i.e.  $1 \times 10^{-3} \text{ cm/sec}$ ).

Stratigraphic marker horizons in the borehole core indicate that the rock mass does not appear to have been disrupted by faulting. The unpublished geological map of the Ontario Geological Survey by D. Williams dated 1974 indicates no geological faults in the vicinity of the site.

Although geological evidence suggests that localized areas of high, horizontal, residual stress do occur in the Ottawa area, no evidence of distress in concrete structures in this area has been reported. This was confirmed through consultations with representatives of Ontario Geological Survey, Building Research Division of the National Research Council, and Ontario Hdyro.

#### GROUNDWATER CONDITIONS

Stabilized borehole water level readings as taken in two open boreholes indicated the groundwater level varied from elevation 72.2 m at the east end of the sewer route to 75 m at the west end of the sewer route.

No stabilized water level was observed in the remaining boreholes due to the short period the boreholes remained open.



A handwritten signature in dark ink, appearing to read "M. Devata".

M. Devata, P. Eng.  
Senior Foundations Engineer

TABLE 1  
POINT LOAD TEST RESULTS

BOREHOLE #	Is <sub>50</sub> MPa Range (average)	Approximate equivalent uniaxial compressive strength MPa Range (average)
1	2.5 - 7.5 (4.0)	61 - 180 (107)
1A	3.9 - 5.4 (4.6)	94 - 130 (111)
2	1.7 - 7.2 (4.0)	40 - 173 ( 96)
3	0.7 - 5.9 (2.7)	17 - 142 ( 65)
5	2.2 - 7.0 (3.9)	53 - 167 ( 92)
8	4.1 - 7.6 (6.2)	98 - 182 (149)

## A P P E N D I X



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# RECORD OF BOREHOLE No 1

METRIC 11

W P 174-80-02 LOCATION Co-ords. N 5 027 028.0; E 364 292.3 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 to 82 04 06 CHECKED BY CP.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100								WATER CONTENT (%)	GR SA SI CL
								SHEAR STRENGTH									
74.9	Ground Surface																
0.0	Topsoil																
74.0	Silty Sand, Some Gravel Trace of Clay (Fill)																
73.5	Org. of High Plasticity		1	SS	12								Om 15.2%				
1.4	Silty Clay with Traces of Gravel, V. Soft		2	SS	8									5 45 32 18			
72.4																	
2.5	Silty Sand with Traces of Clay Dense to V. Dense		3	SS	30												
70.8			4	SS	51									0 85 13 2			
4.1	Silty Sand with Gravel & Trace of Clay Dense (Till)		5	SS	32									63 24 9 4			
69.3	Bouldery Till		C.R.% R.Q.D.%											NXL CORE			
67.9			49		28												
7.0	Limestone (95%) Grey; with randomly interbedded Shale (5%) partings black to dark grey, about 1 to 3 mm thick		100		43												
			100		83												
			100		68												
			100		82												
62.6																	
12.3	End of Borehole																

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 1A

METRIC 12

W P 174-80-02 LOCATION Co-ords. N 5 027 019.3; E 364 272.4 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 to 82 04 07 CHECKED BY *EP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
75.0	Ground Surface																
0.0	Topsoil					*											
74.0	Silty Sand Some Gravel						74										
73.6	Trace of Clay (Fill)		1	SS	2												
1.4	Org. of High Plasticity																
72.6	Silty Clay, Some Sand		2	SS	3												
72.6	Trace of Gravel & Organics V. Soft																
2.4	Silty Sand		3	SS	11		72										
70.7	Some Gravel Compact		4	SS	26												
4.3	Silty Sand, Some Gravel Dense		5	SS	38		70										
60.5	(Till)																
5.5	Bouldery Till		C.R.Z R.C.D.Z														
68.5	Limestone, Grey		89		44		68										
67.7	Limestone (75%)		96		70												
7.3	Grey; with randomly interbedded Shaley (?%) zones and partings about 1 to 5 mm thick		100		97		66										
62.9			100		83		64										
12.1	End of Borehole																
	*Water Level Not Established																

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



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# RECORD OF BOREHOLE No 2

METRIC 13

W P 174-80-02 LOCATION Co-ords. N 5 026 999.6; E 364 226.8 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 to 82 04 06 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
75.0	Ground Surface																
0.0	Topsoil																
74.1	Silty Sand, Some Gravel Trace of Clay (Fill)																
73.5	Org. of High Plasticity		1	SS	8		74									Dm 13.17	
1.5	Silty Clay, Some Sand Trace of Organics																
72.6	Very Soft		2	SS	4												
2.4	Silty Sand with Gravel & Trace of Clay Dense to Very Dense (Till)		3	SS	23		72										
			4	SS	56												
			5	SS	25	3 cm											53 24 18 5
69.9							70										
5.1	Limestone, Grey		C.R.Z		R.Q.D.%			WEATHERING		STRENGTH		JOINTING					
69.1								Slight		High		Very Close					
5.9	Limestone (95%) Grey with randomly inter- bedded Shaley (5%) partings, about 1 to 3 mm thick		100		77		68	Unweathered		High		Predominantly bedding, moderate spacing, rough planar, tight, occasional very closely spaced joints					
			93		83												
			100		50												
			100		65		66										
64.0																	
11.0	End of Borehole																

+3, x5: Numbers refer to  
Sensitivity

20  
15  $\div$  5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 3

METRIC 14

W P 174-80-02 LOCATION Co-ords. N 5 026 986.8; E 364 209.7 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 06 to 82 04 07 CHECKED BY *GP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
75.4	Ground Surface																
0.0	topsoil																
74.5	Silty Sand, Some Gravel Trace of Clay. (Fill)																
73.9	Org. of High Plasticity		1	SS	9		74									22.0%	
1.5	Silty Clay with Some Sand and Trace of Gravel		2	SS	8											3.8%	0 37 43 20
72.0	V. Soft to Soft		3	SS	2		72										
3.4	Bouldery Till		C.R.% 82	R.O.D.% 31													NXL CORE
53																	
70.0																	
5.4	Limestone, Grey with Occasional Shale Interbeds		100	53			70										
68.0			91	11			68										
7.4	Limestone (95%), Grey and Green; with ran- domly interbedded Shalcy (5%) partings about 1 to 3 mm thick		95	55													
100							66										
64.7			100	46													
10.7	End of Borehole																

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE





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# RECORD OF BOREHOLE No 4

METRIC 15

W P 174-80-02 LOCATION Co-ords. N 5 026 980.3; E 364 182.4 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 07 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH					
80.0	Ground Surface													
0.0														
							78							
							76							
							74							
							72							
71.1														
8.9	Refusal - Probably Bouldery Till End of Borehole													

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



Ministry of  
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# RECORD OF BOREHOLE No 5

METRIC

16

W P 174-80-02 LOCATION Co-ords. N 5 026 970.0; E 364 158.2 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 06 to 82 04 07 CHECKED BY *CP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
80.1	Ground Surface							SHEAR STRENGTH						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL x LAB VANE						
								WATER CONTENT (%)						
								20	40	60				
80.1	Silty Sand with Layers of Silty Clay (Fill) Compact to Very Dense		1	SS	23		80							0 83 10 7
			2	SS	22		78							
			3	SS	51	5 cm	76							0 66 25 9
			4	SS	41		74							
			5	SS	24		72							
74.1			6	SS	15		70							Om 4.8%
6.0	Silty Sand with Black Organics		7	SS	120		68							49 32 15 4
73.1			8	SS	62		66							47 30 15 8
7.0	Silty Sand with Gravel Trace of Clay (Till) Very Dense													
69.6														
10.5	Limestone, buff to Grey		100	R.Q.D.Z	16									NXL CORE
67.9														
12.2	Limestone (80%) Grey, with Shaley (20%) zones, randomly interbedded, about 1 to 5 mm thick		100		44									
			100		68									
64.8														
15.3	End of Borehole													

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
S (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 6

METRIC 17

W P 174-80-02 LOCATION Co-ords. N 5 026 951.7; E 364 115.7 ORIGINATED BY HS & EN  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EN  
DATUM Geodetic DATE 82 04 07 CHECKED BY *ef*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
79.9 0.0	Ground Surface					*											
74.4																	
5.5	Bouldery Till (Boulders at about 5.5 to 5.8 m and 7.0 to 7.4 m)																
72.2																	
7.7	Limestone, Grey		C.R.Z.	R.O.D.Z.													
71.2			100	63													
8.7	End of Borehole																
	*Water Level Not Established																



# RECORD OF BOREHOLE No 7

METRIC

18

W P 174-80-02 LOCATION Co-ords. N 5 026 944.9; E 364 099.8 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
79.6	Ground Surface																
0.0	Topsoil					*											
	Silty Sand Loose to Compact (Fill)		1	SS	4		78						o				0 72 25 3
			2	SS	4								o				
			3	SS	15								o				0 77 15 8
75.9							76										
3.7	Silty Sand With Gravel		4	SS	32								o				
74.5	Dense (Till)		5	SS	39								o				39 40 14 7
5.1	Refusal - Probably Bouldery Till *Water Level Not Established																

+3, x5: Numbers refer to  
Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE No 28

DONE BY  
GOLDER & ASSOC.

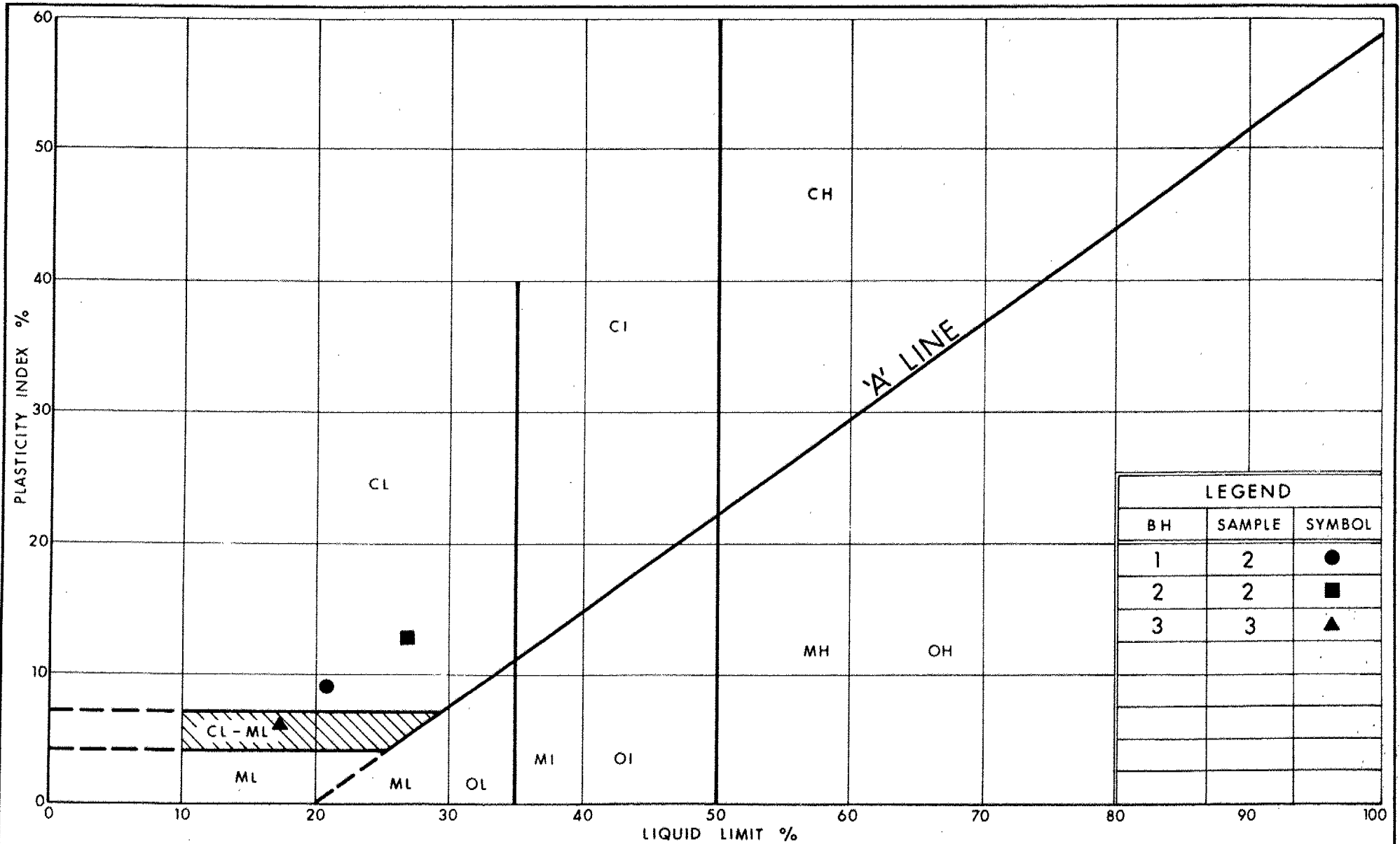
METRIC 20

W P 174-80-02 LOCATION Co-ords. N 5 027 051.0; E 364 301.0 ORIGINATED BY Golder  
DIST 9 HWY 417 BOREHOLE TYPE Wash Boring NX Casing, BX Rock Core COMPILED BY  
DATUM Geodetic DATE 75 03 31 to 75 04 01 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			20	40	60	80	100		
74.8	Ground Surface												
0.0	Topsoil		1	SS	7								
	Silty Sand With Gravel Loose to Very Dense		2	SS	6								
			3	SS	11								
			4	SS	39								
70.5													
4.3	Sandy Silt With Gravel		5	SS	>100								
69.6	Very Dense (Till)												
5.2	Limestone, Grey, With Some Interbedded Shale Layers, Black to Dark Grey		C.R.Z 60	R.Q.D. 0									
67.1			77	22									
7.7	Limestone, Grey, With Irregular Shale Bands And Some Dolomitic Layers		100	37									
			100	33									
			98	36									
			100	56									
			100	65									
			98	38									
			98	70									
56.2													
18.6	End of Borehole												

+3, x5: Numbers refer to  
Sensitivity

20  
15  $\phi$  5 (%) STRAIN AT FAILURE  
10



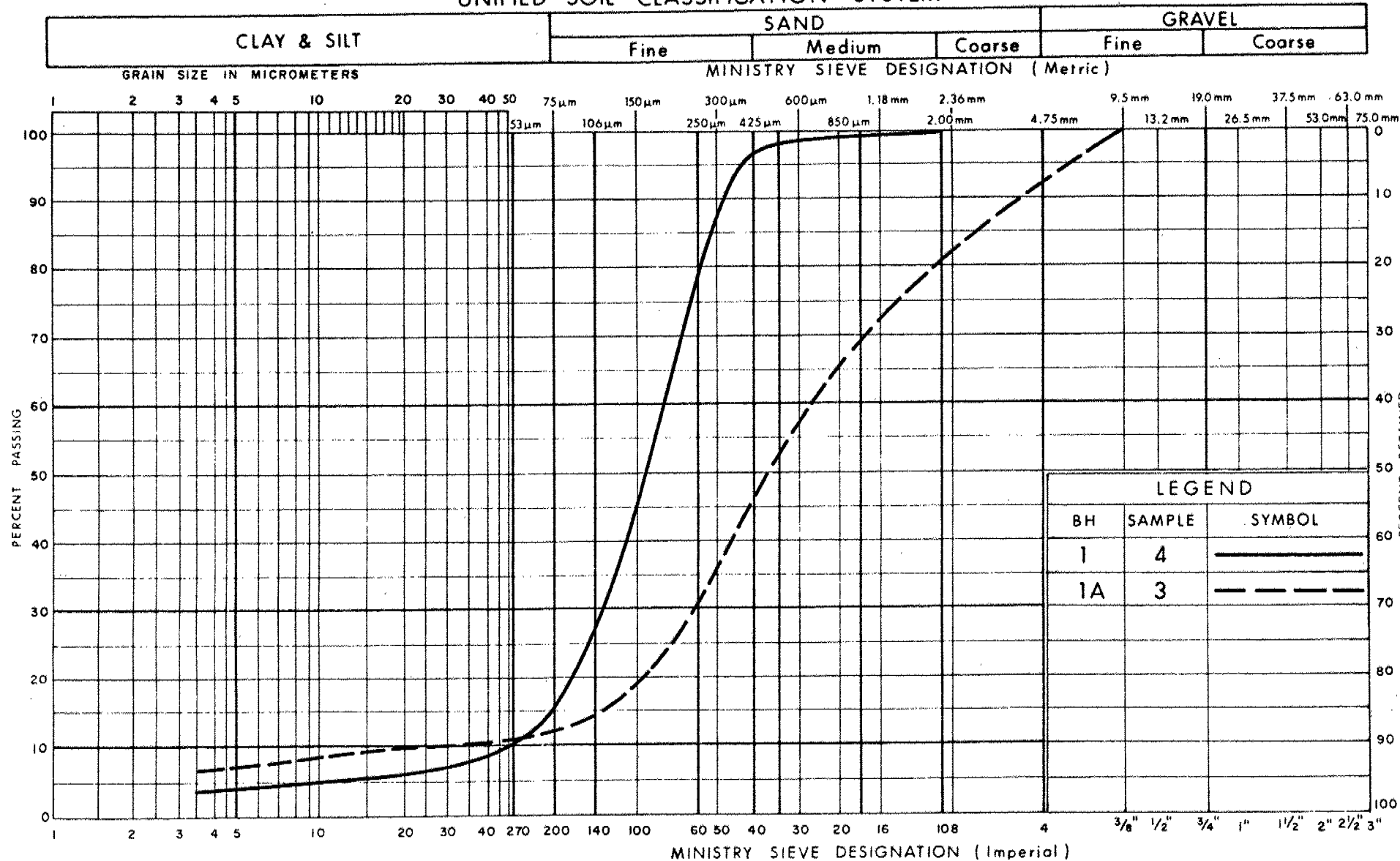
Ministry of  
Transportation and  
Communications

# PLASTICITY CHART SILTY CLAY OF LOW PLASTICITY

FIG No 1

W P 174-80-02

## UNIFIED SOIL CLASSIFICATION SYSTEM



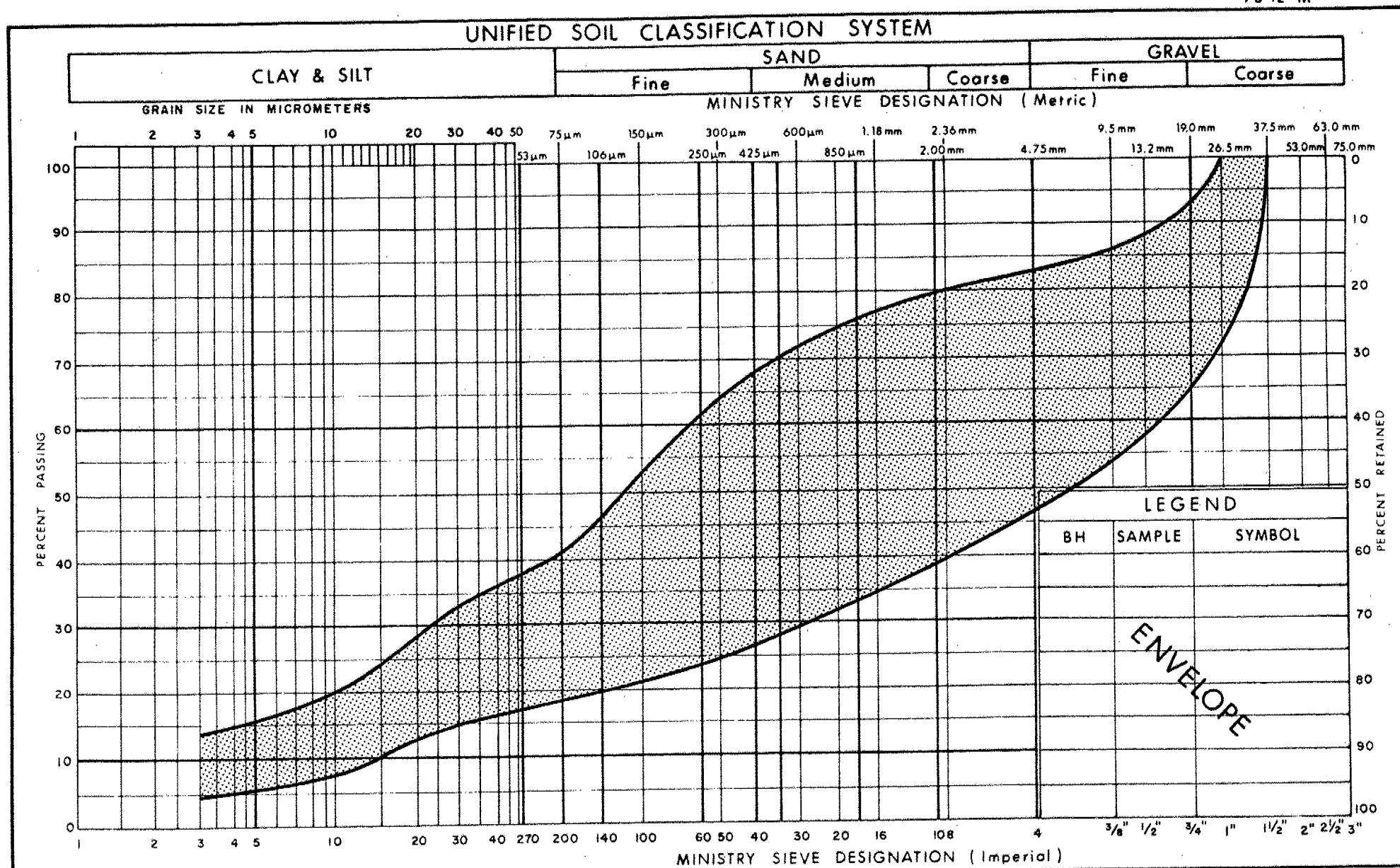
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**GRAIN SIZE DISTRIBUTION**  
**SILTY SAND, TRACE OF GRAVEL & CLAY**

FIG No 2

W P 174-80-02





Ministry of  
Transportation and  
Communications

**GRAIN SIZE DISTRIBUTION**  
**SILTY SAND, WITH GRAVEL & TRACE OF CLAY**  
**(Glacial Till)**

FIG No 3

W P 174-80-02

ENGINEERING MATERIALS OFFICE  
PAVEMENT & FOUNDATION DESIGN SECTION

WP 174-80-02

DIST 9

HWY 417

STR SITE Nil

Report on Claim Storm Sewer Tunnel  
Queensway and Carling Avenue, Ottawa

Contract 82-91

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REPORT ON SUBCONTRACTOR'S CLAIM  
STORM SEWER TUNNEL  
QUEENSWAY AND CARLING AVENUE  
OTTAWA  
W.P. 174-80-02  
CONTRACT 82-91  
HWY. 417, DISTRICT 9, OTTAWA

INTRODUCTION

The contractor for this project, Taggart Construction Limited, subcontracted the construction of the inlet shaft and tunnel to Underground Construction Limited (UCL). The subcontractor has submitted a claim to the Ministry of Transportation and Communications (M.T.C.), the basis of which is that unexpected high water inflows during tunnel construction resulted in lost time and additional pumping costs (for details, see actual claim).

During construction, the subcontractor reported that he was experiencing high water inflows. Messrs. Devata and Magni of M.T.C. visited the site twice during construction (82 11 25 and 82 12 17) to evaluate water inflow conditions. On the advice of Mr. Devata, and with the concurrence of the Regional Construction Office, Mr. J. Nunan, consultant hydrogeologist with Hydrology Consultants Ltd., was present during the second visit to independently assess the water inflow.

The Pavement and Foundation Design Section undertook the original geotechnical investigation in April, 1982, and compiled the Foundation Investigation Report. As the basis of the claim is changed ground conditions, it was considered appropriate for this Section to comment on the claim by U.C.L. and on the report by the consultant to U.C.L. Mr. J. Nunan, consultant to M.T.C. is preparing separate reports on (i) his site visit, and (ii) his comments on U.C.L.'s claim.

Detailed Review of Contractor's Contention

Pg. 2, paragraph 2: "IT IS U.C.L.'S CONTENTION THAT, GIVEN THE FAVOURABLE GEOTECHNICAL INFORMATION AND THE LOW MASS PERMEABILITIES INDICATED IN THE FOUNDATION REPORT, THE M.T.C. DID NOT ANTICIPATE HIGH WATER INFLOWS INTO THE TUNNEL, HENCE THEY CHOSE TO DESIGN THE TUNNEL MINING DOWN HILL FROM THE INLET SHAFT, AS WELL AS ANTICIPATED NO UNUSUAL LOSS OF PRODUCTION BY THEIR CHOOSING OF THE OPTIMISTIC EARLY COMPLETION DATE".

This paragraph appears to be the thrust of the contractor's argument, and tends to misquote M.T.C. to the contractor's advantage. Underlined sections above are discussed further:

"LOW MASS PERMEABILITIES" - The Foundation Investigation Report does not classify the rock as having a low mass permeability. Various hydrogeological experts have various interpretations of low mass permeabilities. The foundation investigation report presents permeability data, and does not qualify the permeability of the rock mass.

"M.T.C. DID NOT ANTICIPATE HIGH WATER INFLOWS" - In the Foundation Investigation Report, Drawing No. 174-80-02 indicates distinctly a zone of very poor rock in the vicinity of Borehole 5, from bedrock surface down to tunnel elevation. Zones of higher permeability should have been anticipated by the subcontractor in this section of the tunnel. In addition, soil overlying bedrock is generally granular in nature, and direct hydraulic communication to this very poor rock mass should have been anticipated. This is, in fact, the section of tunnel in which zones of high inflow were encountered during tunnelling. In addition, the Foundation Investigation Report, pg. 9, says "Local grouting of intensely jointed sections may be necessary to seal these fractures", suggesting such zones could be anticipated.

"HENCE THEY (M.T.C.) CHOSE TO DESIGN THE TUNNEL MINING DOWN HILL" - Choice of down hill tunnelling had nothing to do with anticipated groundwater conditions, but was, in fact, entirely due to access problems on to the property of Summit Ford. In addition, U.C.L. bid this project with a knowledge of the location of the access shaft, and that tunnelling was to be undertaken downhill.

"THEIR CHOOSING OF THE OPTIMISTIC EARLY COMPLETION DATE" - The Ministry's Regional Construction is best suited to decide if it was, in fact, optimistic.

Pg. 2, paragraph 3: U.C.L., IN THE PREPARATION OF ITS TENDER, HAVING ANALYSED THE GIVEN INFORMATION, CAME TO THE SAME CONCLUSION AS THE M.T.C. WITH RESPECT TO ANTICIPATED WATER INFLOWS AND PRODUCTIVITY. It appears the contractor is referring to statements made in the previous paragraph, in which case they are totally misquoting the Ministry without evidence to their advantage.

Pg. 3, paragraph 3: U.C.L. WAS ADVISED BY THE M.T.C. THAT IT WOULD HAVE TO "TECHNICALLY" DEMONSTRATE THAT THE FOUNDATION INVESTIGATION REPORT DIFFERED FROM ACTUAL CONDITIONS ENCOUNTERED.

L.J. Rak Engineering, geotechnical consultants, were retained by U.C.L. for this purpose. Mr. A. Prior of L.J. Rak Engineering prepared a report for U.C.L. commenting on the Foundation Investigation Report, with particular emphasis on groundwater conditions, without visiting the site. We are of the opinion that such comments are irrelevant unless actual conditions are thoroughly evaluated on site.

Pg. 3, paragraph 3: RAK ALSO CONCLUDES "THAT THERE IS NO EVIDENCE IN THE FOUNDATION INVESTIGATION REPORT THAT HEAVY INFLOWS COULD BE INFERRED OR ANTICIPATED". As previously stated in this report, Drawing No. 174-80-02 in the Foundation Investigation Report clearly indicates a zone of very poor to poor rock in the vicinity of Borehole 5. Zones of higher permeability should have been anticipated by the contractor in this section of the tunnel. In addition, pg. 9 of the Foundation Investigation Report states "Local grouting of intensely jointed sections may be necessary to seal these fractures", indicating such zones could be anticipated.

Pg. 4, paragraph 2: ANALYSIS OF U.C.L.'S ACTUAL SCHEDULE SHOWS THAT, IN THE INITIAL MINING ON NOVEMBER 8, AND LATER DURING PRODUCTION OF THE WORK

ON JANUARY 4, JANUARY 5, JANUARY 12, JANUARY 13, JANUARY 21, JANUARY 26, JANUARY 28, FEBRUARY 3, AND FEBRUARY 7, U.C.L. MAINTAINED ITS ANTICIPATED 10' PER SHIFT, WHICH DEMONSTRATES ITS ABILITY TO HAVE MET THE M.T.C. SCHEDULE IN THE ABSENCE OF THE WATER INFLOW. The 10 feet per shift was not a Ministry requirement, but rather the advance rate proposed by the contractor, and the Ministry Foundation Investigation Report certainly did not suggest an absence of water inflow.

DETAILED REVIEW OF REPORT BY L.J. RAK ENGINEERING LTD.

Pg. 1, paragraph 2: "WE HAVE NOT SEEN THE SITE, BUT YOU HAVE PROVIDED US WITH A COPY OF THE FOUNDATION INVESTIGATION REPORT FOR THE CONTRACT, PREPARED BY THE M.T.C. AND A COPY OF THE CONTRACT DRAWINGS WHICH INCLUDES SHEET 10 SHOWING THE LOCATIONS OF A CROSS-SECTION THROUGH THE BOREHOLES. We are of the opinion that it would have been necessary for the contractor's consultant to have seen actual groundwater conditions, in order to have commented on these conditions.

Pg. 3, paragraph 2: "WATER PRESSURE TESTS WERE CARRIED OUT IN 2 BOREHOLES (NO. 3 AND GOLDER'S). IN BOREHOLE 3, THE PACKED-OFF SECTION BETWEEN 7.6 M TO 9.1 M DEPTHS, JUST ABOVE THE TUNNEL CROWN HAD A MASS PERMEABILITY OF THE ORDER OF  $2 \text{ TO } 5 \times 10^{-4}$  CM/SEC. OVER THE SAME DEPTH, THE OVERALL CORE RECOVERY WAS 95%, AND THE R.Q.D. 55%. IN THE SAME BOREHOLE, A PACKED OFF SECTION BETWEEN 6.1 AND 7.6 M DEPTHS, WELL ABOVE THE CROWN, HAD A PERMEABILITY OF THE ORDER  $8 \times 10^{-4}$  TO  $1 \times 10^{-3}$  CM/SEC. OVER THIS DEPTH, THE OVERALL CORE RECOVERY WAS 91%, AND THE R.Q.D. 11%. THE RESULTS APPEAR TO BE RELATIVELY COMPATIBLE TO THE EXTENT THAT THE POORER ROCK (91% VS 95% AND 11% VS 55%) APPEARS TO BE FROM 1.5 TO 5 TIMES MORE PERVIOUS THAN THE SOUNDER ROCK. A PRESSURE TEST WAS ALSO CARRIED OUT IN THE GOLDER HOLE WITH A REPORTED PERMEABILITY OF  $1 \times 10^{-6}$  CM/SEC BETWEEN DEPTHS OF 10.3 AND 18.6 M, I.E. WITHIN AND BELOW THE DEPTH OF THE TUNNEL. WITHIN THE DEPTH OF THE TUNNEL, THE GOLDER BOREHOLE SHOWS OVERALL CORE RECOVERIES OF 98 AND 100%, AND R.Q.D.'S OF 33 AND 36%.

The contractor's consultant (Rak) attempts to draw a comparison between R.Q.D. (a function of joint spacing) and permeability. He shows that, in Borehole 3, the lower the R.Q.D. (and the poorer the rock mass), the higher the permeability. However, he fails to point out that the information obtained from the Golder borehole (BH6), which he reports in this paragraph, does not conform with this trend as is shown below:

BOREHOLE	DEPTH	CORE RECOVERY	R.Q.D.	PERMEABILITY (CM/SEC)
3	7.6- 9.1 m	95%	55%	$2 \text{ to } 5 \times 10^{-4}$
3	6.1- 7.6 m	91%	11%	$8 \times 10^{-4} \text{ to } 1 \times 10^{-3}$
6	10.3-18.6 m	98-100%	33-36%	$1 \times 10^{-6}$

It can be seen that the R.Q.D. values measured in Borehole 6 (i.e. 33 to 36%) are intermediate between the R.Q.D. values measured at the two levels in Borehole 3. According to Rak, it would be anticipated that the permeability value obtained in Borehole 6 would also be intermediate between the

permeability values obtained for the two levels in Borehole 3. This is not the case, and Rak's attempt to relate R.Q.D. and permeability is, therefore, not at all conclusive from the information available. Rak, in fact, appears to contradict himself by a statement on pg. 5, paragraph 2: "INSITU PERMEABILITY TESTS CARRIED OUT ELSEWHERE GAVE LOW TO VERY LOW PERMEABILITIES IN ROCK WITH CONSIDERABLY SMALLER R.Q.D.'S AND OVERALL RECOVERIES". On page 4, paragraph 1, of Rak's report, he attempts to use this argument (i.e. the relationship between R.Q.D. and permeability) to dispute the Ministry's assessment of permeability in the vicinity of Boreholes 1 and 1A. In view of the fact that Rak has contradicted himself twice on this subject, it is our opinion that his argument should be considered invalid.

Pg. 3, paragraph 3: "THE FOUNDATION INVESTIGATION REPORT MENTIONS THAT THE GOLDER PERMEABILITY TESTS WERE CARRIED OUT IN RELATIVELY SOUND LIMESTONE, AND THAT IS ANTICIPATED THAT THE PERMEABILITY OF THE ROCK MASS IN THE VICINITY OF BOREHOLES 1 AND 1A (CLOSE TO THE GOLDER BOREHOLE) COULD BE UP TO AN ORDER GREATER THAN  $1 \times 10^{-6}$  CM/SEC. (i.e.  $1 \times 10^{-3}$  CM/SEC)". Rak has misread and misquoted the Foundation Investigation Report page 5. The section "than  $1 \times 10^{-6}$  cm/sec", underlined above, does not appear in the Foundation Investigation Report. This extract from our report refers to the permeability value obtained in the lower test section in Borehole 3, and does not refer to the values obtained in the Golder borehole.

Hence, the misunderstanding by Rak, pg. 4, paragraph 1: "WE DO NOT UNDERSTAND WHY THE FOUNDATION INVESTIGATION REPORT SUGGESTS THAT THE PERMEABILITY NEAR BOREHOLES 1 AND 1A WOULD BE  $10^{-3}$  CM/SEC VS THAT OF THE GOLDER BOREHOLE WHICH HAS MEASURED TO BE  $10^{-6}$  CM/SEC. IT IS POSSIBLE, HOWEVER, THAT THE REPORT MAY CONTAIN A TYPING ERROR, AND THAT "AN ORDER GREATER THAN  $10^{-6}$  CM/SEC" SHOULD READ  $10^{-5}$  CM/SEC, RATHER THAN  $10^{-3}$  CM/SEC." This is not a typing error as explained above.

Pg. 5, paragraph 1: "THE ROCK IN THE OTTAWA AREA, HOWEVER, DOES NOT HAVE A REPUTATION AS A PROBLEM AREA FOR MINING": This appears to suggest that the groundwater encountered in the tunnel was a major problem. We are of the opinion that groundwater inflow was not excessive, and any experienced contractor would have anticipated occasional higher zones of inflow.

Pg. 5, paragraph 1: "CURRENT GEOLOGICAL INFORMATION INDICATES THAT ROCK IN THE OTTAWA AREA CONSISTS OF FINE TO MEDIUM GRAINED, LAYERED AND MODERATELY JOINTED LIMESTONE WITH A COMPRESSIVE STRENGTH BETWEEN ABOUT 6000 AND 17000 PSI AND PRACTICALLY WATERTIGHT. THE ROCK STRUCTURE RATING IS HIGH, AND SUPPORT DURING EXCAVATION IS DEEMED TO BE NOT OR ONLY marginally REQUIRED. JOINT CONDITIONS ARE REPORTED TO BE TIGHT". This source of information is not mentioned or referenced. General geological information of this nature has little bearing on a site specific problem where detailed foundation data is provided.

Pg. 5, paragraph 4: "WE FURTHER BELIEVE THAT HEAVY WATER INFLOW IS UNCOMMON IN THE OTTAWA AREA". In our opinion, the inflows experienced are not "heavy".

Pg. 5, paragraph 4: "THERE IS NO EVIDENCE IN THE FOUNDATION INVESTIGATION REPORT THAT HEAVY INFLOWS COULD BE INFERRED OR ANTICIPATED. As discussed previously, Drawing No. 174-80-02 in the Foundation Investigation Report clearly indicates a zone of very poor to poor rock in the vicinity of Borehole 5. Zones of higher permeability should, therefore, have been

anticipated by the contractor in this section of the tunnel. In addition, pg. 9 of the Foundation Investigation Report states "Local grouting of intensely jointed sections may be necessary to seal these fractures", which suggests such zones could have been anticipated.

## DISCUSSION

During our site visit on 82 12 17, the following observations were made:

Significant water inflows in the tunnel from the portal to about Station 10 + 130 occurred in two localized areas i.e. about Station 10 + 50 to 10 + 55, and near Station 10 + 70. Small seepages were observed along most of the tunnel although this became obviously less between about Stations 10 + 70 to the face at 10 + 130. At the local zones of higher inflow, rust staining, larger open fractures, and open vugs (holes due to leaching of calcite by water) were observed.

The majority of the inflow into the tunnel was from the local zones described above, and only smaller seepages occurred over the rest of the tunnel. This is apparently confirmed by the subcontractor in a statement in his claim: "Tunnel mining was started on Thursday, October 28, 1982, and some fifty feet into the tunnel excavation, U.C.L. encountered severe water inflow at the tunnel face. U.C.L. pumped 300 gpm from this point on until completion of the tunnel". This suggests that water inflow did not significantly increase with tunnel advancement, and that most inflow was from specific fracture zones.

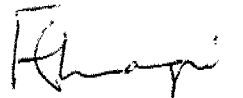
Rak Engineering Ltd. makes the following statement in its report: "Rock is not necessarily homogenous, and it is conceivable that a tunnel heading might intersect local heavier inflows in areas where it is known that the rock is highly fractured". Rak attempts to prove further that the Foundation Investigation Report does not indicate that the rock mass is "highly fractured". As discussed above, observations made during construction indicate that, in general, the rock mass is not highly fractured, and that inflow is through local permeable zones. Drawing No. 174-80-02 in the Foundation Investigation Report indicates distinctly a zone of very poor to poor rock in the vicinity of Borehole 5. Zones of higher permeability should have been expected by the subcontractor in this section of the tunnel. In addition, on page 9, "Local grouting of intensely jointed sections may be necessary to seal these fractures". This statement suggests that higher permeability zones could be anticipated.

Rak Engineering bases their calculation of estimated total inflow into the tunnel on a completely homogeneous rock mass with an average permeability of  $5 \times 10^{-4}$  cm/sec (based on packer test results from the Foundation Investigation Report). Based on this permeability value, Rak Engineering estimates that an inflow of 40 gpm for 100 feet of tunnel could be anticipated i.e. for a homogeneous rock mass. According to the subcontractor, the actual inflow was about 300 gpm for the entire tunnel length, about 850 feet, or 35 gpm for 100 feet. The total predicted and actual inflows for the entire tunnel appear, therefore, to be very similar. The consultant hydrogeologist for the Ministry measured an actual flow of 268 gpm for the first 426 foot section of tunnel i.e. 63 gpm for 100 feet. This section incorporated the zones of higher inflow. It appears, in addition, therefore, that the total predicted and actual inflows encountered in the vicinity of the more permeable zones were not significantly dissimilar.

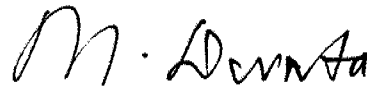
CONCLUSION

It is our opinion that the subcontractor has overemphasized the problem of water inflow during tunnelling, and that the quantity reported and observed should have routinely and easily been handled by an experienced tunnel contractor. It is our contention that the rock mass permeability values obtained in Borehole 3 i.e.  $2$  to  $5 \times 10^{-4}$  cm/sec, or probably even lower values of  $1 \times 10^{-4}$  to  $1 \times 10^{-5}$  cm/sec are applicable to the majority of the rock through which the tunnel was constructed. Most of the water inflow was from specific zones, and only smaller seepages occurred from the rest of the tunnel. We understand that the subcontractor made no provision, in his estimates, for possible zones of higher inflow, and also made no attempt to prevent flow by, for example, grouting. We contend that, within reason, the Foundation Investigation Report accurately portrays rock and groundwater conditions that were actually encountered during construction.

This report was prepared by E.R. Magni, and reviewed by M. Devata.



E.R. Magni,  
Geologist.

M. Devata, P. Eng.  
Senior Foundation Engineer.



ENGINEERING MATERIALS OFFICE  
PAVEMENT & FOUNDATION DESIGN SECTION

WP 174-80-02

DIST 9

HWY 417

STR SITE

The Proposed Storm Sewer Tunnel in Ottawa  
Queensway & Carling Avenue

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# FOUNDATION INVESTIGATION REPORT

For

The Proposed Storm Sewer Tunnel in Ottawa

Queensway & Carling Ave.

W.P. 174-80-02

Hwy. 417, District 9, Ottawa

## INTRODUCTION:

This report contains the results of the foundation investigation for the proposed 259 metre long section of large diameter (2.1m) storm sewer running from near the intersection of Carling Avenue Westbound and Tweedsmuir Avenue to connect to the existing Cave Creek storm sewer tunnel immediately west of the intersection of Carling Avenue westbound and Kirkwood Avenue.

As the proposed alignment crosses both the Ottawa Queensway and Carling Avenue westbound, present plans call for the sewer to be constructed by means of tunnelling techniques.

The purpose of this investigation was to determine the overburden-bedrock line along the length of the proposed sewer route in order that the sewer may be constructed by tunnelling techniques and secondly, to determine the bedrock parameters as they relate to the design of the tunnel section and lining, including any special construction considerations which could influence design decisions.

The field work was carried out from April 5 to April 7, 1982. The borings were advanced by hollow stem augers and consisted of 7 sampled boreholes and two supplementary probe holes to depths where augers met practical refusal. Standard drive open samples were taken throughout the overburden strata in all 7 sampled holes and bedrock was cored in NXL size to below invert level of the proposed sewer.

An additional borehole No. 28 carried out by Golder Associates on April 1, 1975 for the Hintonburgh West Storm Collector has also been incorporated into this report.

During core drilling, records were kept of drilling water return, core recovery and R.Q.D. Also, in BH#3 in situ pressure packer tests

were carried out to determine the permeability characteristics of the bedrock. Following completion of the field work, laboratory testing was carried out on both the overburden and bedrock samples.

#### Site Description

The site stretches from east of the intersection of Carling Avenue and Tweedsmuir Avenue across Hwy. 417 to just west of the intersection of Kirkwood Avenue and Carling Avenue in Ottawa. In the site vicinity, the terrain is flat and classified as urban area.

Physiographically, the site is located within the Ottawa Valley Clay Plains.

#### SUBSOIL CONDITIONS

##### General

The subsoil investigation revealed that 1 to 6 metres of fill underlain by a thin layer (0.5 metre) of black organics extends over the entire proposed sewer route. Beneath this fill along the eastern section of the route in boreholes 1, 1A, 2 and 3, a 1 to 1.8 metre thick layer of silty clay with some sand was encountered. Below the silty clay in boreholes 1 and 1A, a 1.6 to 1.8 metre thick layer of silty sand with a trace of gravel and clay was found. Underlying the silty sand in the eastern section of the route and the fill material along the remaining section of the route extends a 2 to 4 metre thick layer of glacial till which in turn is underlain by limestone bedrock.

The boundaries between the various soil types and bedrock, as well as laboratory test results and ground water levels are shown on the attached Record of Borehole Sheets. The locations and elevations of the borings, along the stratigraphical profile are shown on Drawing No. 1748002-A.

The various soil types and the bedrock are described in the following paragraphs.

### Surficial Fill

The surficial fill material encountered across the site beneath 10 to 15 cm. of top soil consists of a loose to compact silty sand with gravel and a trace of clay. This fill varies in thickness from about 1 metre along the route to 6 metres at the Queensway embankment. Standard penetration 'N' values varied from 4 to 51 blows per 0.3 m indicating a very loose to very dense relative density.

The fill is underlain by a thin layer of black-organic material which probably represents the original ground.

### Silty Clay

Underlying the fill in boreholes 1, 1A, 2 and 3 (eastern part of route) is a 1 to 1.8 metre thick deposit of silty clay with some sand and a trace of gravel.

The results of Atterberg Limit and water content testings plotted on the Plasticity Chart Fig. #1, indicate the deposit to be silty clay of low plasticity. The test results are summarized below:

		RANGE
Natural Moisture Content	(W)%	16 - 31
Liquid Limit	(W <sub>l</sub> )%	16 - 27
Plastic Limit	(W <sub>p</sub> )%	10 - 14
Plastic Index	(W <sub>i</sub> )%	6 - 13

Based on interpretation of Standard Penetration Test 'N' Values and augering operations, the consistency of the silty clay deposit varies from very soft to soft.

### Silty Sand

Underlying the silty clay in boreholes 1 and 1A is a 1.6 to 1.8 metre thick deposit of silty sand with a trace of gravel and clay. Grain size analyses carried out on samples of the silty sand recovered from boreholes 1 and 1A are shown on Figure 2.

Based on Standard Penetration Test 'N' values ranging from 11 to 51 blows per 0.3 m, the relative density for this deposit is compact to very dense.

#### Glacial Till

Glacial till occurs below the fill, silty clay and silty sand in all boreholes. This deposit ranges in thickness from 2 to 4 metres and is classified as sandy silt with gravel and a trace of clay. However, it should be noted that the samples as tested were recovered in the standard 51 mm O.D. split spoon sampler and may not reflect the presence of cobbles and boulders within this glacial deposit. The cobble and boulder content was found to increase with depth in the granular till deposit in boreholes 1, 1A, 3 and 8, and it was necessary to employ diamond drilling techniques to advance boreholes through the lower portion of this stratum. The results of grain size distribution testing performed on samples obtained from 51 mm O.D. split spoon sampler from the deposit are plotted on Figure 3.

Based on the Standard Penetration Test 'N' values ranging from 23 to 120 blows per 0.3 m, the relative density of this non-cohesive glacial till deposit may be described as compact to very dense.

#### Bedrock Conditions

The site is underlain by limestone of the Ottawa Formation. Boreholes drilled along the line of the proposed tunnel encountered bedrock at depths ranging from about 5.1 m to 10.5 m below existing ground surface. The bedrock surface which is at its highest elevation (i.e. 72.2 m) in the vicinity of Station 10+054 (Borehole 6) towards the western end of the proposed tunnel line drops off to its lowest measured elevation (i.e. 67.9 m) in the vicinity of Station 10+246.6 (Borehole 1) to the east. Bedrock could be present at lower elevations between boreholes.

Boreholes indicate that the rock is predominantly a grey, fine to medium grained limestone containing dolomitic and shaley zones and partings. Shale is estimated to comprise about 5 percent of the rock

mass and is present mainly as randomly, interbedded partings about 1 to 3 mm thick.

High core recoveries (generally greater than 95 percent) and R.Q.D. (Rock Quality Designation) measurements indicate that the rock mass is in general 'good' (R.Q.D.=75 to 90%) with 'fair' (R.Q.D.=50 to 75%) and occasional poor sections. In most boreholes the upper 0.6 to 1.5 m of the rock mass is generally more intensely fractured than the rock at depth, and is generally 'fair' with 'poor' and 'very poor' sections.

Point load strength tests on selected samples (see Table 1) indicate that the uniaxial compressive strength of the intact rock could vary from about 40 to 182 MPa, average 103 MPa. The relationship between point load and uniaxial compressive strengths is only approximate. The rock mass is intersected mainly by closely to moderately spaced bedding joints and some near vertical crossjoints. In general joint surfaces are rough planar and tight. Occasional zones of very closely spaced, intersecting joints were encountered in the borehole core.

Water pressure tests were carried out in packed-off sections in Borehole 3 at depths of 6.1 to 7.6 m and 7.6 m to 9.1 m below the existing ground surface in order to assess the permeability of the rock mass. The rock mass immediately above the proposed tunnel crown at Borehole 3 (i.e. the 7.6 to 9.1 m test section), has a mass permeability in the order of  $2$  to  $5 \times 10^{-4}$  cm/sec. The rock mass in the vicinity of the upper test section i.e. 6.1 to 7.6 m below existing ground surface has a permeability in the order of about  $8 \times 10^{-4}$  to  $1 \times 10^{-3}$  cm/sec. Packer tests undertaken previously by Golder Associates for the City of Ottawa in Borehole 28 at the eastern end of the proposed tunnel line were conducted in relatively sound limestone. The permeability of the rock mass at depths of between 10.3 m and 18.6 m was  $1 \times 10^{-6}$  cm/sec. It is anticipated that the permeability of the rock mass in the vicinity of Stations 10+246.6 and 10+224.7 (Borehole 1 and 1A) could be up to an order greater (i.e.  $1 \times 10^{-3}$  cm/sec).

Stratigraphic marker horizons in the borehole core indicate that the rock mass does not appear to have been disrupted by faulting. The unpublished geological map of the Ontario Geological Survey by D.Williams dated 1974 indicates no geological faults in the vicinity of the site.

Although geological evidence suggests that localized areas of high, horizontal, residual stress do occur in the Ottawa area, no evidence of distress in concrete structures in this area has been reported. This was confirmed through consultations with representatives of Ontario Geological Survey, Building Research Division of the National Research Council, and Ontario Hdyro.

#### GROUNDWATER CONDITIONS

Stabilized borehole water level readings as taken in two open boreholes indicated the groundwater level varied from elevation 72.2 m at the east end of the sewer route to 75 m at the west end of the sewer route.

No stabilized water level was observed in the remaining boreholes due to the short period the boreholes remained open.

## DISCUSSIONS AND RECOMMENDATIONS

### a) General

It is proposed to construct a 2.1 m diameter storm sewer parallel to Hwy. 417 along the northerly right-of-way limit from 0.3 km east of Maitland and Carling Avenue with a sewer outlet tunnel constructed within the bedrock diagonally across Hwy. 417 to outlet into the existing Cave Creek sewer at Kirkwood Ave. This report deals mainly with the portion of the sewer beneath Hwy. 417 (Queensway) and Carling Avenue where it will be constructed in tunnel. The proposed sewer tunnel has invert elevations of 65.2 and 65.0 m at Stations 10+000.6 and 10+263.5 (i.e. Boreholes 8 and 28) respectively.

Subsurface conditions along the line of the tunnel consist in general of 4.8 to 10.5 m of overburden (fill, organics, silty clay and sandy till with boulders) overlying limestone bedrock of the Ottawa Formation. Groundwater levels varied from elevation 75.0 to 72.2 m towards the western and eastern ends of the tunnel respectively.

For the purposes of discussion, the proposed tunnel may be divided into two sections according to the amount of bedrock present above the proposed elevation of the tunnel crown:-

- (i) bedrock cover greater than 1 tunnel diameter i.e. from Sta.10+000.6 to Sta.10+175 (BH 8 to BH 2)
- (ii) bedrock cover less than 1 tunnel diameter i.e. from Sta.10+175 to Sta.10+263.5 (BH 2 to BH28)

### b) Tunnel Section from Sta.10+000.6 to Sta.10+175 (BH 8 to BH 2)

The Geomechanics Rock Mass Rating (R.M.R.) System for tunnels classifies the majority of the rock mass through which the proposed tunnel will be driven as 'good' with 'fair' and occasional 'poor' sections. The rock mass above the crown of the tunnel is in general 'fair' with 'poor' and occasional 'good' sections. Jointing above the crown is more closely spaced than at tunnel level and vertical jointing appears to be more dominant.



A careful and well-designed blast method will be necessary to minimize overbreak and backbreak (damage and loosening of wall rock) during excavation. Extensive backbreak will tend to reduce the self-supporting capacity of the rock mass, and additional primary support will be required to support this damaged rock mass. The actual blast design should be the responsibility of an experienced contractor, although it is suggested that a smooth blasting or presplitting method be utilized in order to obtain the best final shape and cause the least damage to the rock mass. It is anticipated that some local scaling of loose blocks will be necessary after blasting.

It is recommended that a damage (crack) survey of adjacent buildings be undertaken prior to blasting in order that it may be compared with post blasting conditions. During blasting, it is recommended that vibration monitoring apparatus be located on adjacent buildings and a record of blast vibrations be obtained.

According to the Geomechanics Rock Mass Rating System, the rock in the crown should in general be self supporting for the construction period. Rock loads during construction will, therefore, effectively be zero.

It is recommended, however, that the primary lining be installed a distance of about one tunnel diameter behind the advancing face. It is recommended that systematic rock bolting be undertaken in the crown and walls, and this should consist of 1.5 to 2.0 m long, fully bonded, resin grouted bolts at about 1.5 to 2.5 m centers. The spacing will vary locally according to rock mass conditions. Wiremesh and 50 mm of shotcrete is recommended in the upper third of the tunnel. Locally in more intensely jointed portions of the rock mass e.g. in the vicinity of Sta. 10+100.3 (Borhole 5), the primary support could be increased e.g. 100 mm of shotcrete and possible use of steel strapping. Observational control of rock mass conditions during excavation is considered essential so that primary support can be varied as required. Convergence monitoring of a number of cross-sections within the tunnel is recommended to confirm the adequacy of primary support.

Packer tests in boreholes indicate the rock mass above the tunnel crown has permeabilities in the range  $1 \times 10^{-4}$  to  $10^{-5}$  cm/sec and in the tunnel section about  $1 \times 10^{-5}$  to  $10^{-6}$  cm/sec. Groundwater inflow during construction is not expected to be excessive, but will require efficient tunnel drainage and pumping methods. Local grouting of intensely jointed sections may be necessary to seal these fractures. Inflow of soil is not expected along this portion of the tunnel. It is recommended that consideration be given to the possible effects of groundwater drawdown on the stability of neighboring structures, embankments etc., although given the locality of the adjacent buildings and the clayey nature of the soils, settlement of these structures is considered unlikely.

Considering Terzaghi's rock mass classification, the rock at crown level along this portion of the tunnel can best be classified as "moderately blocky and seamy". The permanent tunnel liner should therefore be designed for the following vertical rock pressure:

$$\text{Vertical rock load} = 0.25 \gamma D \text{ to } 0.35 \gamma (2D)$$

In the long term, the permanent liner should be designed for all-round external hydrostatic pressure. As discussed previously, there appears to be no evidence of distress in concrete structures due to rock creep in the Ottawa area. It appears therefore that the permanent lining does not have to be designed to accommodate potential rock creep.

c) Tunnel Section from Sta.10+175 to Sta.10+263.5 (BH 2 to BH 28)

The bedrock cover above tunnel crown elevation along this section of the proposed tunnel varies in thickness from about 0.8 to 2.0 m, although it could possibly be less between boreholes. The Geomechanics Rock Mass Rating (R.M.R.) System classifies the majority of the rock mass through which this section of the tunnel will be driven as 'fair' with 'good' and 'poor' sections. The crown rock material is classified mainly as 'poor' and 'very poor'. Intact rock in the crown is generally of medium strength (15 to 50 MPa) and zones of the rock mass are intensely fractured.

In view of the poor rock mass conditions and the thin rock cover along the line of this section of the tunnel, extreme care with excavation and liner installation will be necessary. The need for a well-designed blast method is again emphasized in order to minimize backbreak and in this case, particularly, overbreak. Crown stability along this section of tunnel is crucial. Comments previously made on excavation and blasting in connection with the other section of the tunnel are applicable to this section as well.

The standup time of rock in the crown will be limited and could be as little as a few hours. The rate of advance of tunnelling should consequently be such as to allow adequate support close to the advancing face. It is suggested that blast rounds be limited in length to about 1.5 m to allow the immediate installation of primary support. "Rib and lagging" type primary support should be installed with the steel sets at about 1.0 m centres. Primary support should be to within about 0.5 m of the advancing face and should be installed as soon as possible i.e. within 1 or 2 hours of excavation. It is recommended that pressure grouting be undertaken between the lagging and ground immediately after installation of support to limit ground movement. Probing should be undertaken ahead of the advancing face by means of upward inclined drilling to investigate rock conditions in and above the crown. Probing and observational control is considered essential so that the primary support installation can be varied as required.

It is estimated that the rock mass above the crown of the proposed tunnel will have permeabilities of about  $1 \times 10^{-3}$  to  $10^{-4}$  cm/sec. Groundwater inflows could be high, but it is anticipated that this water will be controllable. Local zones of higher inflow are possible and inwashing of soil could occur on open joints particularly in the areas of least bedrock cover. Should the soil/bedrock interface undulate in such a manner that the crown of the tunnel has to be constructed in soil or a thin cover of very poor rock, a significant inflow of water should be anticipated into the tunnel. The implementation of a special method to control groundwater inflow will have to be introduced.

The use of compressed air or the lowering of the groundwater appear to be feasible methods, although either method will be difficult to implement and expensive. It is recommended that the method selected be the responsibility of a contractor experienced in that field. As discussed in the previous section, the possible effect of groundwater drawdown on the stability of adjacent structures must be considered.

The permanent tunnel liner should be designed for a vertical load equal to the full overburden weight, and for long-term conditions should be designed for an all-round external hydrostatic pressure. It is suggested that the following bulk unit weights be used in this design

Silty sand, sands	19.7 kN/m <sup>3</sup>	(125 pcf)
Silty clays	17.3	(110 pcf)
Till	21.2	(135 pcf)
Rock	26.7	(170 pcf)

#### d) Alternative Tunnel Alignments

It is strongly recommended that consideration be given to constructing the proposed tunnel at slightly greater depths in order to provide at least 2 metres of predominantly 'good' or 'fair' rock over the tunnel crown for its entire length. Alternative alignments could include:

- i) steepening the gradient of the tunnel so that it is at least 2 metres deeper at the eastern end of the tunnel i.e. western end remains at the same elevation, or
- ii) maintaining the proposed gradient but driving the entire tunnel 2 metres deeper than proposed.

The improved rock conditions and crown stability means that the primary lining system recommended for the proposed tunnel section from Sta.10+000.6 to Sta.10+175 (BH 8 to BH 2), i.e. rock bolts and shotcrete can in general be employed for the support of the entire length of the tunnel. Locally in more intensely jointed portions of the rock mass

e.g. in the vicinity of Sta.10+100.3 (Borehole 5) it may be necessary to increase primary support requirements i.e. bolts at closer centres, thicker shotcrete, possible strapping etc. In addition, it is anticipated that along the full length of the tunnel groundwater inflow during construction will not be excessive, and will in general be controllable with efficient tunnel drainage and pumping methods. Comments provided previously in the report for the tunnel section from Sta.10+000.6 (BH 8) to Sta.10+175 (BH 2) are applicable to these alternative alignments.

e) Shafts

It is understood that a shaft is to be constructed at each end of the proposed tunnel. The presence of a boulder concentration immediately above bedrock will probably prevent the driving of interlocking steel sheet piling and groundwater inflow during excavation is consequently unlikely to be prevented.

It is recommended therefore that a conventional soldier pile and lagging system be used to support the sides of the excavation. Groundwater lowering will be required to install this support system. It is essential that the soldier piles penetrate the bouldery till and are end bearing on bedrock. For stability purposes, it is recommended that the soldier piles are located about 0.3 to 0.5 m back from the edge of the excavation in bedrock. It is recommended that organic soil in the vicinity of the shaft at Sta. 100+000 be removed and replaced with suitably compacted granular material to facilitate construction work at this location.

In the shafts, vertical faces in the bedrock should stand unsupported during the construction period. Spot-bolting and wiremesh may be necessary to support localized blocks of rock or zones of more intensely fractured bedrock.

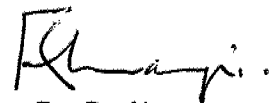
Recommended earth pressure design is presented in Figure 4.

MISCELLANEOUS

The field work supervision and report writing was jointly done by Messrs. H. Szymanski, Engineering Materials Tech. and E. R. Magni, Geologist under overall supervision of Mr. M. Devata, Senior Foundations Engineer who also reviewed this report.

The equipment used was owned and operated by Marathon Drilling Co. Ltd. and Johnston Drilling Co. Ltd.

  
H. Szymanski  
Engineering Material Tech.

  
E. R. Magni  
Geologist


  
M. Devata  
Senior Foundations Engineer



TABLE 1

POINT LOAD TEST RESULTS

BOREHOLE #	Is <sub>50</sub> MPa Range (average)	Approximate equivalent uniaxial compressive strength MPa Range (average)
1	2.5 - 7.5 (4.0)	61 - 180 (107)
1A	3.9 - 5.4 (4.6)	94 - 130 (111)
2	1.7 - 7.2 (4.0)	40 - 173 ( 96)
3	0.7 - 5.9 (2.7)	17 - 142 ( 65)
5	2.2 - 7.0 (3.9)	53 - 167 ( 92)
8	4.1 - 7.6 (6.2)	98 - 182 (149)

A P P E N D I X





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# RECORD OF BOREHOLE No 1

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 027 028.0: E 364 292.3 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 to 82 04 06 CHECKED BY CP.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
74.9	Ground Surface																
0.0	Topsoil																
74.0	Silty Sand, Some Gravel Trace of Clay (Fill)						74									Om 15.2%	
73.5	Org. of High Plasticity		1	SS	12												
1.4	Silty Clay with Traces of Gravel, V. Soft		2	SS	8												5 45 32 18
2.5	Silty Sand with Traces of Clay Dense to V. Dense		3	SS	30		72										
70.8			4	SS	51												0 85 13 2
4.1	Silty Sand with Gravel & Trace of Clay Dense (Till)		5	SS	32		70										63 24 9 4
69.3	Bouldery Till		C.R.X R.Q.D.X														NXL CORE
67.9			49		28		68										
7.0	Limestone (95%) Grey; with randomly interbedded Shale (5%) partings black to dark grey, about 1 to 3 mm thick		100		43		66	WEATHERING		STRENGTH		JOINTING					
			100		83			Unweathered		High		Predominantly bedding, closely to moderately spaced, rough planar, tight					
			100		68		64										
			100		82												
62.6																	
12.3	End of Borehole																

+3, x5: Numbers refer to  
Sensitivity

20  
15 + 5 (%) STRAIN AT FAILURE  
10



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# RECORD OF BOREHOLE No 1A

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 027 019.3; E 364 272.4 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 to 82 04 07 CHECKED BY *CP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
75.0	Ground Surface																
0.0	Silty Sand some Gravel					*											
74.0	Trace of Clay (Fill)																
73.6	Org. of High Plasticity		1	SS	2		74									2.42	11 70 13 6
1.4	Silty Clay, Some Sand																
72.6	Trace of Gravel & Organics V. Soft		2	SS	3												
2.4	Silty Sand						72										7 82 6 5
	Some Gravel Compact		3	SS	11												
70.7			4	SS	26												
4.3	Silty Sand, Some Gravel Dense (Fill)		5	SS	38		70										21 50 25 4
69.5																	
5.5	Bouldery Till		C.R. % R.O.D. %														NXL CORE
68.5																	
6.5	Limestone, Grey		89		44		68	WEATHERING Slight	STRENGTH Medium	JOINTING Very Close							
67.7																	
7.3	Limestone (75%) Grey; with randomly interbedded Shaley (25%) zones and partings about 1 to 5 mm thick		96		70		66	Unweathered	High	Predominantly bedding, closely to moderately spaced, rough planar, tight							
			100		97		64										
62.9			100		83												
12.1	End of Borehole																
	*Water Level Not Established																

\*3, \*5: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10



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# RECORD OF BOREHOLE No 2

METRIC

W/P 174-80-02 LOCATION Co-ords. N 5 026 999.6; E 364 226.8 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 to 82 04 06 CHECKED BY *EP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
75.0	Ground Surface													
0.0	Silt													
74.1	Silty Sand, Some Gravel, Trace of Clay (Fill)		1	SS	8		74						On 13.12	
73.5	Org. of High Plasticity		2	SS	4									
1.5	Silty Clay, Some Sand Trace of Organics		3	SS	23		72							
72.6	Very Soft		4	SS	56									
2.4	Silty Sand with Gravel & Trace of Clay Dense to Very Dense (Till)		5	SS	257	3 cm	70							53 24 18 5
69.9	Limestone, Grey		C.R.Z	R.Q.D.										
69.1	Limestone (95%) Grey with randomly inter- bedded Shaley (5%) partings, about 1 to 3 mm thick		100		77		68							
5.9			93		83									
			100		50		66							
			100		65									
64.0														
11.0	End of Borehole													

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10

5 (%) STRAIN AT FAILURE



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# RECORD OF BOREHOLE No 3

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 026 986.8; E 364 209.7 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 06 to 82 04 07 CHECKED BY *CP*

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
75.4	Ground Surface															
0.0	Topsoil															
74.5	Silty Sand, Some Gravel															
73.9	Trace of Clay. (Fill)															
1.5	Org. of High Plasticity		1	SS	9											
1.5	Silty Clay with Some Sand and Trace of Gravel		2	SS	8											
72.0	V. Soft to Soft		3	SS	2											
3.4	Bouldery Till															
			C.R.Z	R.O.D.Z												
			82	31												
70.0			53	7												
5.4	Limestone, Grey with Occasional Shale Interbeds		100	53												
68.0			91	11												
7.4	Limestone (95%), Grey and Green; with randomly Interbedded Shaley (5%) partings about 1 to 3 mm thick		95	55												
			100	46												
64.7																
10.7	End of Borehole															

+3, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15  $\div$  5 (%) STRAIN AT FAILURE  
10



## METRIC

W P 174-80-02 LOCATION Co-ords. N 5 026 980.3: E 364 182.4 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 07 CHECKED BY [Signature]

[illegible]

+3, x5: Numbers refer to Sensitivity

15  $\phi$  5 (%) STRAIN AT FAILURE



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# RECORD OF BOREHOLE No 5

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 026 970.0; E 364 158.2 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 06 to 82 04 07 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
80.1	Ground Surface																
0.0	Silty Sand with Layers of Silty Clay (Fill) Compact to Very Dense		1	SS	23		80										0 83 10 7
			2	SS	22		78										
			3	SS	51	5 cm	76										0 66 25 9
			4	SS	41		74										
			5	SS	24		72										
74.1							70										
6.0	Silty Sand with Black Organics		6	SS	15		68										
73.1							66										
7.0	Silty Sand with Gravel Trace of Clay (Till) Very Dense		7	SS	120		64										
			8	SS	62		62										
69.6							60										
10.5	Limestone, buff to Grey		R.C.Z	R.O.D.Z			58										
67.9			100	16			56										
12.2	Limestone (80%) Grey, with Shaley (20%) zones randomly interbedded, about 1 to 5 mm thick		100	44			54										
			100	68			52										
64.8							50										
15.3	End of Borehole						48										

+3, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 6

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 026 951.7; E 364 115.7 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 07 CHECKED BY *ef*

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W <sub>p</sub>	W		
79.9	Ground Surface															
0.0																
74.4																
5.5	Bouldery Till (Boulders at about 5.5 to 5.8 m and 7.0 to 7.4 m)															NXL CORE
72.2																
7.7	Limestone, Grey		C.R.Z	R.O.D.Z												
71.2			100	63												
8.7	End of Borehole  *Water Level Not Established															



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# RECORD OF BOREHOLE No 7

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 026 944.9; E 364 099.8 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
79.6	Ground Surface															
0.0	Topsoil															
	Silty Sand Loose to Compact (Fill)		1	SS	4											0 72 25 3
			2	SS	4											
			3	SS	15											0 77 15 8
75.9			4	SS	32											
3.7	Silty Sand With Gravel		5	SS	39											39 40 14 7
74.5	Dense (Till)															
5.1	Refusal - Probably Bouldery Till *Water Level Not Established															

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
5 (%) STRAIN AT FAILURE





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# RECORD OF BOREHOLE No 8

METRIC

W P 174-80-02 LOCATION Co-ords. N 5 026 930.2; E 364 066.5 ORIGINATED BY HS & EM  
DIST 9 HWY 417 BOREHOLE TYPE Auger and NXL Rock Core COMPILED BY HS & EM  
DATUM Geodetic DATE 82 04 05 CHECKED BY *EP*

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>	
75.9	Ground Surface														
0.0	Topsoil														
74.4	Black Organics with Some Gravel		1	SS	16										Om 6.1%
1.5	Silty Sand with Gravel & Trace of Clay Dense (Till)		2	SS	24										16 43 30 11
			3	SS	48										
71.7			4	SS	23/										38 27 26 9
71.2	Bouldery Till		100		72										
4.7	Limestone (95%), Grey, with randomly inter- bedded Shale (5%) partings, black to dark gray, about 1 to 3 mm thick		R.C.Z	R.O.D.	Z										
			100	100											
			100	100											
			100	100											
			100	98											
65.2															
10.7	End of Borehole														

+3, x5: Numbers refer to  
Sensitivity

20  
15  
10  
5 (% STRAIN AT FAILURE)



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# RECORD OF BOREHOLE No 28

DONE BY  
GOLDER & ASSOC.

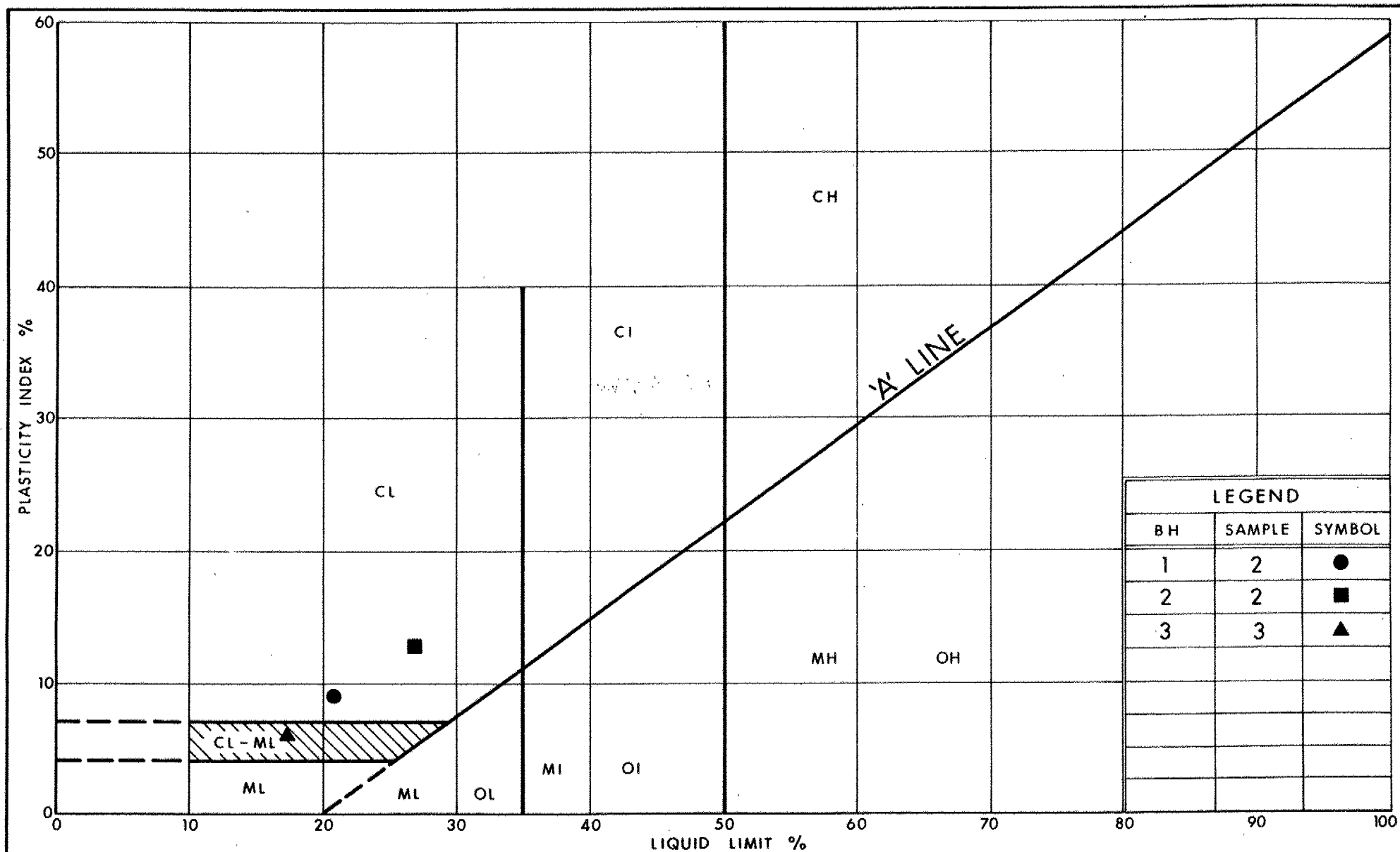
METRIC

W P 174-80-02 LOCATION Co-ords. N 5 027 051.0; E 364 301.0 ORIGINATED BY Golder  
DIST 9 HWY 417 BOREHOLE TYPE Wash Boring NX Casing, BX Rock Core COMPILED BY  
DATUM Geodetic DATE 75 03 31 to 75 04 01 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
74.8	Ground Surface																
0.0	Topsoil		1	SS	7		74										
	Silty Sand With Gravel Loose to Very Dense		2	SS	6												
			3	SS	11		72										
			4	SS	39												
70.5																	
4.3	Sandy Silt With Gravel																
69.6	Very Dense (Till)		5	SS	>100		70										
5.2	Limestone, Gray, With Some Interbedded Shale Layers, Black to Dark Gray		C.R.Z	R.O.D.Z													
			60		0												
			77		22		68										
67.1																	
7.7	Limestone, Gray, With Irregular Shale Bands And Some Dolomitic Layers		100		37		66										
			100		33												
			98		36		64										
			100		56		62										
			100		65		60										
			98		38		58										
			98		70												
56.2																	
18.6	End of Borehole																

\*3, \*5: Numbers refer to  
Sensitivity

20  
15  
10  
5  
0  
5  
10  
15  
20  
(%) STRAIN AT FAILURE



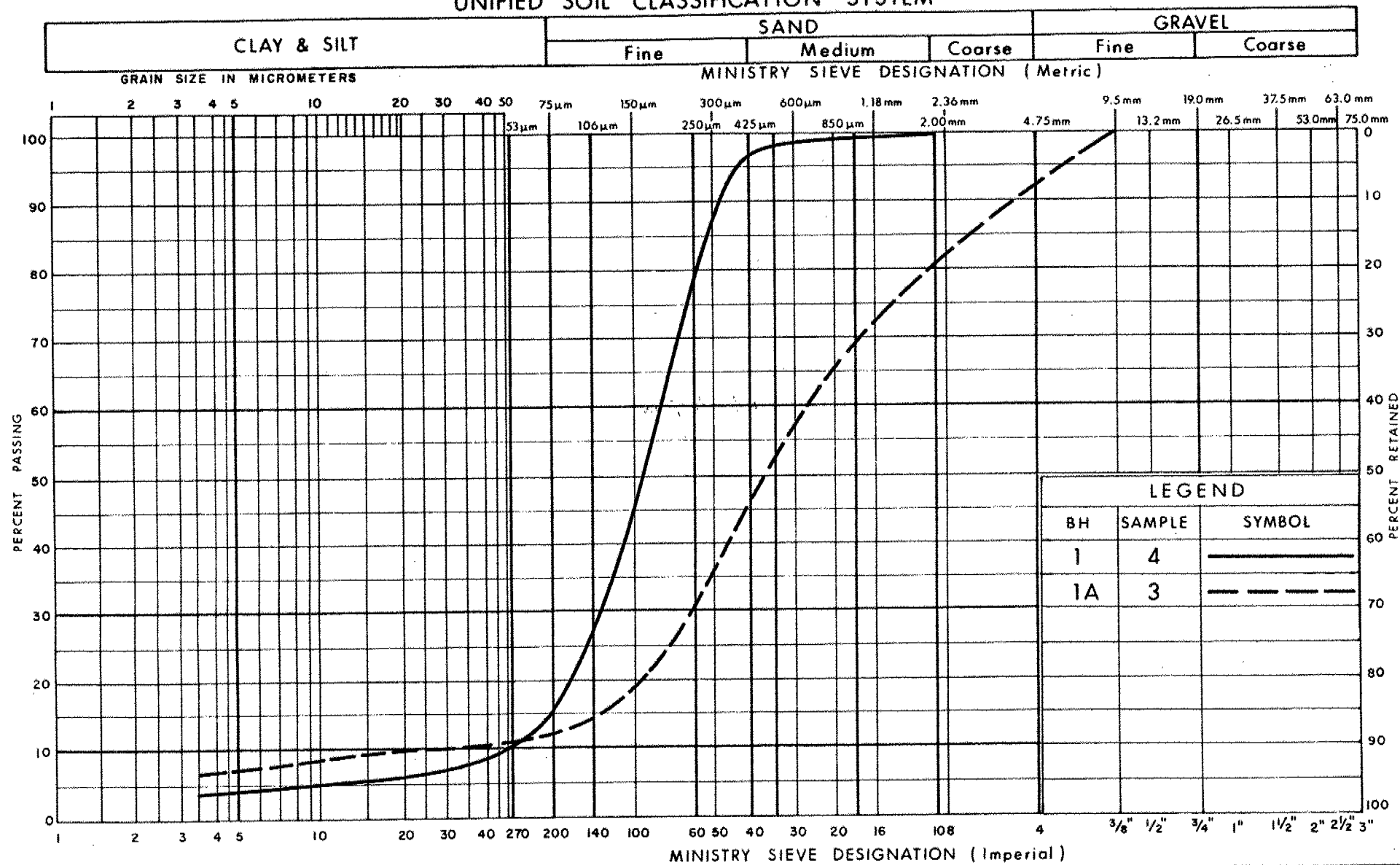
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# PLASTICITY CHART SILTY CLAY OF LOW PLASTICITY

FIG No 1

W P 174-80-02

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

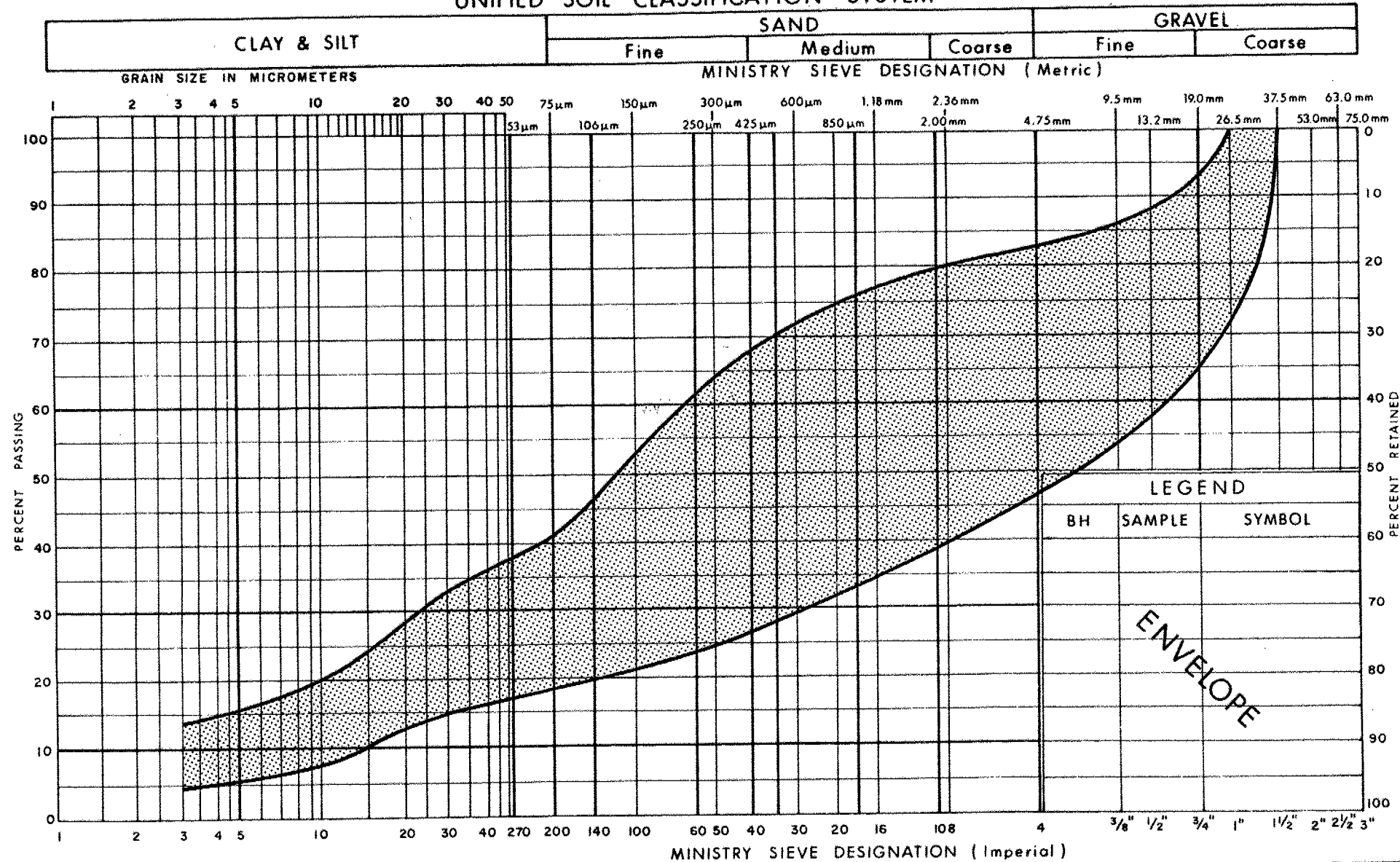
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GRAIN SIZE DISTRIBUTION  
SILTY SAND, TRACE OF GRAVEL & CLAY

FIG No 2

W P 174-80-02

## UNIFIED SOIL CLASSIFICATION SYSTEM



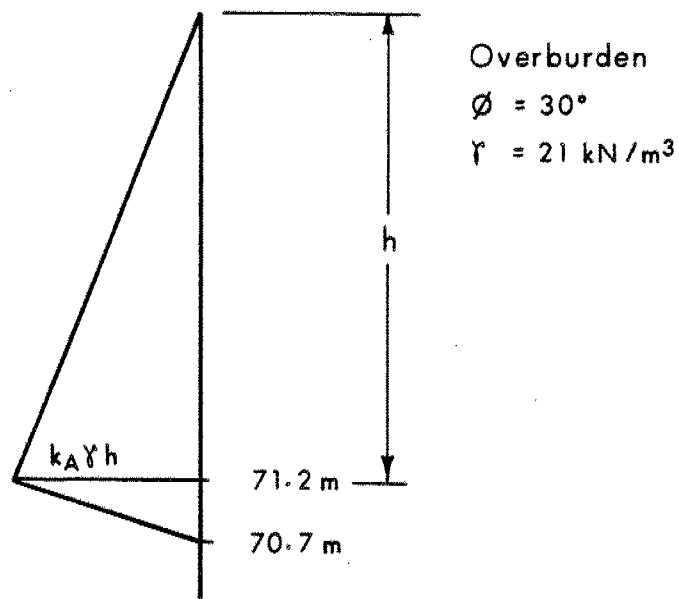
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**GRAIN SIZE DISTRIBUTION**  
**SILTY SAND, WITH GRAVEL & TRACE OF CLAY**  
 (Glacial Till)

FIG No 3

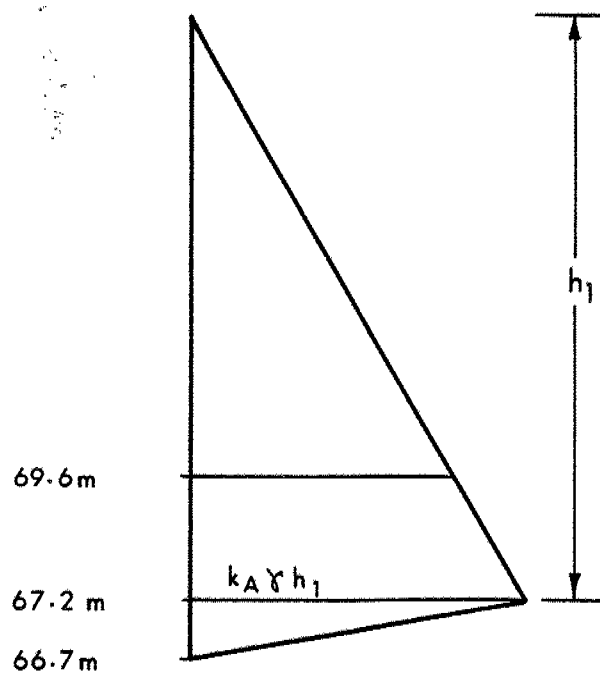
W P 174-80-02

# SHAFT AT STA 10+000.6 (B H 8)



# SHAFT AT STA. 10+263.5 (near B H 1 & 28)

Overburden  
 $\phi = 30^\circ$   
 $\gamma = 21 \text{ kN/m}^3$



Plus Water Pressures

N.T.S.

Fig. 4

## 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

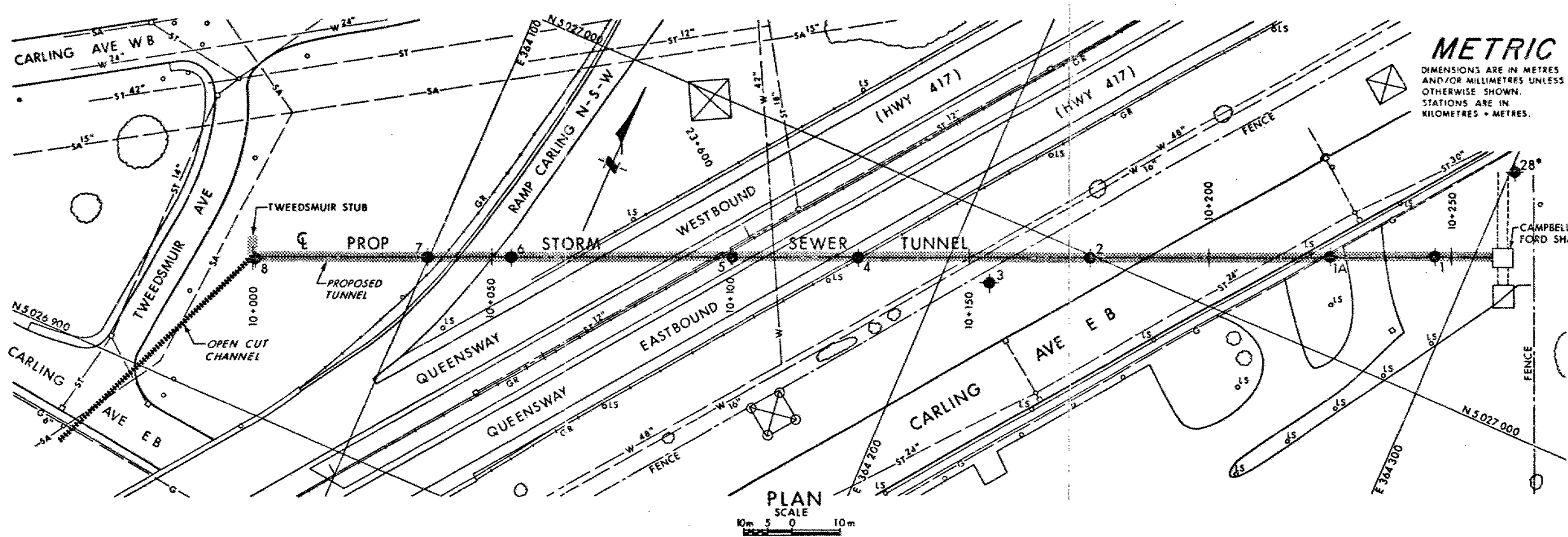
SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### MECHANICAL PROPERTIES OF SOIL

$u_w$	kPa	PORE WATER PRESSURE	$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$r_u$	1	PORE PRESSURE RATIO	$C_c$	1	COMPRESSION INDEX
$\sigma$	kPa	TOTAL NORMAL STRESS	$C_s$	1	SWELLING INDEX
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS	$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$\tau$	kPa	SHEAR STRESS	$c_v$	$\text{m}^2/\text{s}$	COEFFICIENT OF CONSOLIDATION
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES	H	m	DRAINAGE PATH
$\epsilon$	%	LINEAR STRAIN	$T_v$	1	TIME FACTOR
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS	U	%	DEGREE OF CONSOLIDATION
E	kPa	MODULUS OF LINEAR DEFORMATION	$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
G	kPa	MODULUS OF SHEAR DEFORMATION	$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\mu$	1	COEFFICIENT OF FRICTION	$\tau_f$	kPa	SHEAR STRENGTH
			$c'$	kPa	EFFECTIVE COHESION INTERCEPT
			$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
			$c_u$	kPa	APPARENT COHESION INTERCEPT
			$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
			$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
			$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
			$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{\min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
$\rho_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{\text{sat}}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{\text{sat}}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	$e_{\max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$\text{kN}/\text{m}^3$	SEEPAGE FORCE
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						

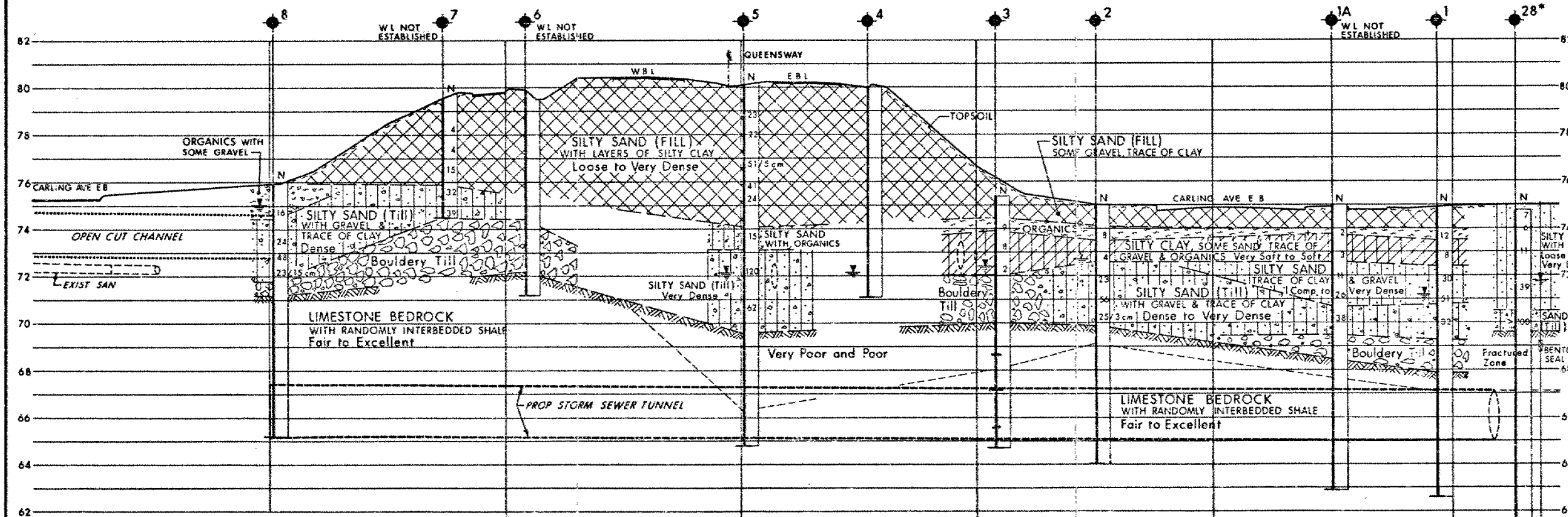
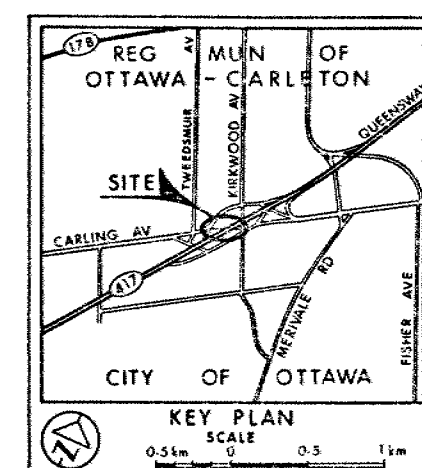


**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN.  
STATIONS ARE IN  
KILOMETRES + METRES.

CONT No  
WP No 174-80-02

PROP STORM SEWER TUNNEL  
(AT QUEENSWAY & CARLING AVE)  
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



- 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 1982 04
  - W.L. Not Established in Boreholes 1A, 6 & 7
  - W.L. in Standpipe 1975 04
  - Standpipe

No	ELEVATION	CO-ORDINATES NORTH	EAST
1	74.9	5 027 028.0	364 292.3
1A	75.0	5 027 019.3	364 272.4
2	75.0	5 026 999.6	364 226.8
3	75.4	5 026 986.8	364 209.7
4	80.0	5 026 980.3	364 182.4
5	80.1	5 026 970.0	364 158.2
6	79.9	5 026 951.7	364 115.7
7	79.6	5 026 944.9	364 099.8
8	75.9	5 026 930.2	364 066.5
28*	74.8	5 027 051.0	364 301.0

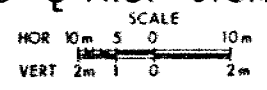
\* Borehole 28 done by Golder & Associates 1975 04

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

Rock mass classified according to RQD

RQD (%)	ROCK QUALITY
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor

PROFILE ALONG & PROP STORM SEWER TUNNEL



Revisions

DATE	BY	DESCRIPTION

Geocres No 31G5-135

HWY No 417	CHECKED	DATE 1982 06 04	SITE
SUBM'D HS	CHECKED	APPROVED	DWG 1748002-A



WP 174-80-02



**Golder Associates**  
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

WP 174-80-02  
CORPORATION OF THE CITY OF OTTAWA

SUBSURFACE INVESTIGATION  
HINTONBURGH WEST STORM COLLECTOR  
STAGE 6

OTTAWA

ONTARIO

Distribution:

12 copies - Corporation of the City of Ottawa,  
Ottawa, Ontario

2 copies - H.Q. Golder & Associates Ltd.,  
Ottawa, Ontario

August, 1975

752031

## TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	1-2
INTRODUCTION	3
DESCRIPTION OF PROJECT	3-4
PROCEDURE	4-5
SUBSURFACE CONDITIONS	5
A. CARLING AVENUE ALIGNMENT (ALTERNATE 1)	6-10
B. HAMPTON PARK ALIGNMENT (ALTERNATE 2)	10-13
DESIGN AND CONSTRUCTION CONSIDERATIONS	13-14
A. FULL ROCK FACE TUNNEL	14-17
B. SOFT GROUND & MIXED FACE TUNNEL	17-22
C. OPEN CUT SECTION	22-24
D. SHAFTS	24-25
TABLE 1 - SUMMARY OF COMPRESSIVE STRENGTH TESTS ON BEDROCK CORE	
TABLE 2 - SUMMARY OF PRESSURE PACKER TESTS ON BEDROCK	
ABBREVIATIONS AND SYMBOLS	
RECORD OF BOREHOLE SHEETS (W-24 to W-40, inclusive)	
FIGURES:	
1. Boring Plan - Carling Avenue Alignment (in pocket)	
2. Soil Stratigraphy - Carling Avenue Alignment (in pocket)	
3. Boring Plan - Hampton Park Alignment	
4. Soil Stratigraphy - Hampton Park Alignment	
5.-9. Grain Size Distribution Curves	
10. Shear Strength versus Elevation (Silty Clay) (Boreholes W-32 to W-36, & W-39)	

## ABSTRACT

The results of a subsurface investigation carried out along two proposed alignments for Stage 6 of the West Outlet of the Hintonburgh Area Storm Collector System are reported herein.

Based on the results of the boring program, the Carling Avenue alignment is shown to be underlain by shallow bedrock along much of its route. However, west of Merivale Road, the boreholes encountered a deep depression in the bedrock surface. The overburden in this area consists of shallow deposits of fill and silty clay underlain by some 50 feet of sandy silt till with gravel, cobble, and boulder size material. The till also contains lenses, layers, and pockets of silt, sand, and sand and gravel. The Hampton Park Alignment is underlain by shallow bedrock along about a third of its length. The overburden along the remainder of the route consists of surficial fills and silty sands underlain by a relatively extensive deposit of firm sensitive silty clay, the upper portion of which has been weathered to a stiff crust. The silty clay is in turn underlain by a glacial till consisting of silty sand to sandy silt with gravel, cobbles, and boulders. Towards the west end of this route near the Queensway, the tills are underlain by glacial sands and sand and gravel.

Depending on the alignment chosen, one or more types of construction will have to be employed. Full rock face tunnel sections, where the bedrock cover above the crown is greater than 1 tunnel diameter, should present no undue tunneling problems. The limestone bedrock may be expected to stand unsupported during construction. Recommendations on liner design are given in the report.

Soft ground and mixed face tunnel sections will present construction difficulties. Along Carling Avenue, it is anticipated that an extensive dewatering system would have to be installed to lower the groundwater level below tunnel level in order to limit seepage forces at the tunnel face. Seepage gradients at the tunnel face could, if allowed to develop, cause instability and loss of ground at the tunnel face. Dewatering will also have to be carried out in the Queensway area where the Hampton Park alignment will be in tunnel.

The open cut section proposed along the Hampton Park Alignment should be stable provided the clay is not unduly disturbed and provided surcharge loading due to stockpiled material is not allowed. Bottom heave of the trenches will be a consideration in this area and relief of the excess pressure in the underlying glacial till should be carried out prior to and during excavation.

Notes on the design and construction of the proposed shafts along the route are given in the report. Detailed earth pressure calculations could be provided once the location and sizing of these shafts is finalized.

## INTRODUCTION

H.Q. Golder & Associates Ltd. have been retained by the Corporation of the City of Ottawa, Physical Environment Department, to carry out a subsurface investigation for Stage 6 of the West Outlet for the Hintonburgh Area Storm Collector System in Ottawa, Ontario. The purpose of this investigation was to determine the subsoil, bedrock, and groundwater conditions in the area of the proposed sewer route. Based on this information, recommendations were to be made for the design and construction of the storm sewer from a geotechnical viewpoint.

## DESCRIPTION OF PROJECT

Stage 6 of the West Outlet for the Hintonburgh Area Storm Collector system (as originally planned) involved some 5,100 lineal feet of large diameter sewer running from Clarendon Avenue just north of the Queensway, southerly under the Queensway and Island Park Drive to a point about 200 feet south of Island Park Drive, thence westerly to Merivale Road. The proposed route then swings south along Merivale Road to Carling Avenue and west along Carling Avenue to Kirkwood Avenue (see Figure 1). The proposed invert of the sewer along this routing was at a depth of 50 to 60 feet below existing ground surface, necessitating the probable use of tunnel construction techniques. However, upon completion of the field borings (boreholes W-24 to W-31) in this area, it was concluded that this proposed alignment presented serious and costly tunneling considerations. It was therefore decided to investigate the subsurface conditions along an alternate alignment. Figure 3 (attached) shows this alternate alignment which runs westerly from Clarendon Avenue at the Queensway, parallels the Queensway across Island Park Drive, Merivale Road, and Hampton Park.

The proposed route then passes near the intersection of Carling Avenue (westbound) and Kirkwood Avenue and thence under the Queensway to Carling Avenue (eastbound). The proposed invert along this routing will be at a depth of 50 to 60 feet between Clarendon Avenue and Merivale Road. West of Merivale Road it is proposed to carry out construction in open cut with sewer invert levels in the order of 20 to 25 feet below existing ground surface. Tunneling techniques would be employed between Clarendon Avenue and Merivale Road and also where the sewer route crosses under the Ottawa Queensway.

#### PROCEDURE

The field work for this investigation was carried out between March 18 and April 16, 1975 (boreholes W-24 to W-31 inclusive), June 3 to 11, 1975 (boreholes W-32 to W-39 inclusive), and June 23 to 24, 1975 (borehole W-40). Boreholes W-24 to W-31 were put down using a trailer-mounted diamond drill rig while boreholes W-32 to W-40 were carried out using a bombardier mounted hollow stem auger machine. Both drill rigs were supplied and operated by the F.E. Johnston Drilling Co. Ltd., of Ottawa. All of the boreholes (with the exception of W-38) were taken to about 5 feet or more below the proposed sewer invert level. Standard drive open samples were taken throughout all the overburden strata and in-situ vane shear tests were carried out in the cohesive silty clay strata where encountered. Bedrock was cored in BX size to below proposed invert level in 11 of the 17 boreholes. In addition, pressure packer tests were carried out in a number of the boreholes to determine the coefficient of secondary permeability of the bedrock in the area. Standpipes or piezometers were installed in the majority of the boreholes to measure the stabilized groundwater level in the various substrata along the routes. The field work

was supervised throughout by members of our engineering staff who were responsible for borehole layout, drill rig supervision, and logging and care of the samples.

The soil and bedrock samples recovered from the boreholes were brought to our laboratory for detailed examination and representative classification testing. The results of the laboratory testing are shown on the Record of Borehole sheets and on Figures 5 to 9.

A detailed log of each borehole is given on the Record of Borehole sheets following the text of this report. The locations of the borings are shown on Figure 1 for Alternate 1 - Carling Avenue Alignment and Figure 3 for Alternate 2 - Hampton Park Alignment. A stratigraphic section showing the inferred subsurface conditions along Alternate 1 and 2 are shown on Figures 2 and 4 respectively.

The location of each boring was determined in the field by us with reference to existing features (buildings, curbs, etc.). It is understood that the boreholes are to be located with reference to survey baselines and alignments by City of Ottawa survey crews. The elevations of the ground surface at the borehole locations were given to us by the City of Ottawa survey personnel. It is understood that these elevations are referred to Geodetic datum.

#### SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each borehole is shown on the Record of Borehole sheets and is illustrated on the stratigraphic sections on Figures 2 and 4. Since this investigation was ultimately carried out along two completely differing alignments, the following summarized account of the subsurface conditions will be discussed separately for the two alignments.

---

A. CARLING AVENUE ALIGNMENT (Alternate 1)Surficial Fill

Beneath a 4 to 6 inch cover of either topsoil (off roadway areas) or asphalt (built up areas), all borings except W-28 and W-29 encountered a surficial fill deposit. This fill varied in thickness along the route from about 3 to 9 feet and was generally comprised of sand and gravel with some silt, cobbles, bricks and wood. Standard penetration tests, where they were carried out in the fill material, gave N values ranging from about 5 to 23 blows per foot indicating that the fill is generally loose to compact.

Silty Sand

Underlying the fill in boreholes W-26, W-27 and W-28 (western end of route) is a 1 to 13 foot thick (W-28) deposit of brown to grey silty sand with some gravel. Grading analyses carried out on samples of the silty sand recovered from borehole W-28 are shown on Figure 5. Standard penetration tests carried out on the silty sand gave N values of 4 to 39 blows per foot (increasing with depth) indicating a loose to very dense relative density.

Silty Clay

With the exception of boreholes W-24 and W-28 (extreme east and west ends of this routing) a deposit of silty clay was encountered in each boring put down along this alignment. The silty clay was found to be of limited thickness along this route, ranging from 2 feet in borehole W-26 to a maximum of 10 feet in borehole W-27. An atterberg limit test carried out on a sample of the silty clay from borehole W-25 gave a liquid limit value of 54 and a plastic limit value of 22.



The natural water content of the same sample was 68 percent, well above the liquid limit, typical of the sensitive clays of the Ottawa area.

In boreholes W-26, -27, -29, and -30 the silty clay was generally grey-brown in colour and N values of 2 to 6 blows per foot measured in the silty clay indicate a very stiff consistency. In boreholes W-25 and W-31 the clay was grey in colour and in-situ vane tests gave shear strength values of between 700 and 1,200 pounds per square foot, indicating a firm to stiff consistency for the silty clay in these boreholes.

#### Glacial Till

A relatively extensive deposit of glacial till was encountered, underlying the above fill, sand, and clay deposits, in each boring put down along this route. At the eastern end of the route, between Clarendon Avenue and Merivale Road, and also at the western limits of the route at Kirkwood Avenue, the till was found to consist of a thin (3 to 8 feet thick) mantle over relatively shallow bedrock. However, along Carling Avenue between Merivale Road and Kirkwood Avenue, the thickness of the glacial till increases significantly from about 15 feet in borehole W-3 (previous report 70794) to over 49 feet in borehole W-31. The thickness of the till is directly related to the depth to bedrock and defines a deep buried valley in this area.

Grain size analyses were carried out on a number of samples of the glacial till recovered from the boreholes and the results of these tests are shown on Figure 6. It is seen that although all the samples tested are well-graded, they do exhibit a wide variance in texture i.e. silt size and smaller varying from 12 to 32 percent. In general the glacial till may be classified as a sandy silt to silty sand

with gravel and a trace to some clay. It should be noted however, that the samples as tested were recovered in the standard 2 inch drive open sampler and so do not reflect the considerable cobble and boulder content of this glacial deposit. The cobble and boulder content was found to increase with depth in the till and, in every case, it was necessary to employ diamond drilling techniques to advance the borehole through the lower portion of the till deposit. Of significance are the results of boreholes W-26 and W-30 where the lower 2 to 6 feet of the glacial till was comprised almost entirely of cobbles and boulders. In addition to the variable texture of the till, numerous lenses, layers, and pockets of sand, silty sand, and sandy silt were encountered intermittently throughout the glacial till.

Standard penetration tests were carried out within the glacial till in each borehole. A wide range in N values was recorded, varying from a low of 11 blows per foot to greater than 100 blows per foot, but with an apparent majority of values being in the 20 to 50 blows per foot range. Based on the N values, the relative density of the glacial till is considered to range from compact to very dense, but to be generally in the compact to dense range.

#### Bedrock

Limestone bedrock was encountered in all the boreholes put down along this Carling Avenue alignment. The surface of the bedrock was encountered at depths ranging from 10 to 63 feet (elevation 241 to 180) below existing ground surface. The lowest bedrock surfaces were encountered in boreholes W-25 and W-31 and outline a deep depression in the bedrock surface along Carling Avenue to the west of Merivale Road.

The top 2 to 11 feet of the bedrock in boreholes W-24, -25, -28, and -30 was found to be fractured with core recoveries

in the order of 60 to 80 percent. Based on the core recoveries and R.Q.D.'s (Rock Quality Designation, which is a measure of the intact "sticks" of core longer than 4 inches expressed as a percentage of the total core drilled) the remainder of the bedrock appears to be relatively sound to sound (core recoveries generally greater than 95 percent).

The core samples recovered indicate the bedrock to be a grey, fine to medium grained limestone. However, the limestone also contains numerous irregular black shale partings and laminae and also zones of dolomitic limestone. In borehole W-31, the top 9 feet of core recovered was basically a shale bedrock with some limestone bands. Detailed descriptions of the core recovered are given on the individual Record of Borehole sheets.

Twelve unconfined compression tests were carried out on intact core samples of the limestone and dolomitic limestone. The results of these tests, in terms of maximum recorded compressive strength, are given in Table 1 following the text of this report. As shown, the compressive strength of the sound bedrock in this area was found to range from 5,000 to near 14,000 pounds per square inch. These values appear in order for a bedded sedimentary limestone bedrock.

Pressure packer tests were carried out within the fairly sound bedrock in boreholes W-28, -29, and -30. A summary of these tests is given in Table 2 following the text of this report. The coefficient of secondary permeability of the fairly sound limestone bedrock in this area was found to be in the order of  $1 \times 10^{-6}$  to  $5 \times 10^{-5}$  centimetres per second, indicating a bedrock mass with low permeability. Water takes in the fairly sound rock would be expected to occur along unhealed stringers, fractures,

and bedding planes. In addition, it would naturally be expected that significantly higher water inflows would be encountered within the fractured bedrock zones.

#### Groundwater

Standpipes or piezometers were installed in seven of the boreholes in order to allow measurement of the groundwater level after stabilization. Measurements taken in the sealed standpipes and piezometer (borehole W-31) indicate a water level in the till and bedrock at some 9 to 17 feet below existing ground surface. This would suggest that a slight excess water pressure exists in the glacial till and bedrock at depth.

#### B. HAMPTON PARK ALIGNMENT (Alternate 2)

##### Surficial Fill

Under a 4 to 13 inch cover of topsoil, the boreholes (with the exception of W-33 and W-39) along this routing encountered a 1 to 3 foot thick deposit of fill material. In general the fill is comprised of silty sand with gravel and occasional cobbles and boulders. Borehole W-40, put down through the Ottawa Queensway-Kirkwood Avenue embankment, encountered some 18 feet of fine to medium sand fill. Standard penetration tests in this embankment fill gave N values of 20, 26, and 36 blows per foot indicating a compact to dense relative density.

##### Silty Sand

Underlying the fill in all of the boreholes west of Merivale Road (Boreholes W-32 through W-37 and W-40) is a deposit of silty sand. This silty sand was found to vary

in thickness from 1 to 6.5 feet in the boreholes along the route. Standard penetration tests carried out within the silty sand stratum gave N values of between 3 to 14 blows per foot indicating a very loose to compact relative density.

#### Silty Clay

With the exception of boreholes W-24, W-28, W-38 and W-40, an extensive deposit of silty clay was encountered in the borings put down along this alignment. The silty clay was generally encountered below either the fill or silty sand material, except in borehole W-39 where the silty clay was encountered directly under the topsoil. The overall thickness of the silty clay was found to range from a minimum of 2.5 feet at borehole W-37 to an apparent maximum of almost 30 feet at borehole W-33.

The top 3 to 11 feet of the silty clay in all boreholes was found to be weathered to a very stiff grey-brown crust. Below this depth (except in W-37 where only 2.5 feet of crust was encountered), the color of the clay changes to grey and the consistency decreases to firm. Atterberg limit tests carried out on samples of the grey silty clay gave liquid limit values ranging from about 45 to 80 and plastic limit values from 18 to 26. The natural water content of the silty clay ranges from 40 to 60 percent in the weathered crust and from 45 to 85 in the underlying grey clay. In general, the natural water contents ranged from 4 to 14 percent in excess of the liquid limit value, typical of the plastic, sensitive Ottawa Valley clays.

In-situ vane tests were performed throughout the silty clay deposit. The in-situ shear strength of the silty clay was found to range from greater than 2,000 pounds per square foot in the weathered crust to between 700 and 1,000 pounds per square foot in the grey portion. A plot of shear strength

versus elevation is shown on Figure 10 for boreholes W-32 to W-36 and W-39.

### Glacial Till

Underlying the fill, sand, and silty clay deposits in all the boreholes put down along this alignment is a deposit of glacial till. Between Clarendon Avenue and Merivale Road (boreholes W-24, W-38 and W-39) and at Kirkwood Avenue (W-28) the till was found to consist of a relatively thin (1 to 9 feet thick) mantle over the bedrock. Between Merivale Road and Carling Avenue, boreholes W-32 to W-36 were terminated after penetrating some 5 to 17 feet into the till. In boreholes W-37 and W-40 the thickness of the glacial till was in the order of 7 to 8 feet.

Grading analyses carried out on three representative samples of the glacial till are shown on Figure 7 and indicate the till in this area to be essentially a silty sand to sandy silt with gravel and a trace to some clay. The glacial till also contains a fair percentage of cobble and boulder size material and, in many of the boreholes, lenses, pockets and layers of silty sand, sand, and gravel.

Standard penetration tests carried out in the glacial till gave N values ranging from 1 to 6 blows per foot in the upper portion of the till deposit, to between 26 and 93 blows per foot with depth. The relative density of the till is therefore considered to range from very loose to very dense.

### Bedrock

Limestone bedrock was encountered within the depth of exploration in six of the boreholes (W-24, W-28, and W-37 to W-40). The surface of the bedrock was encountered at depths of 12 to 29 feet (elevation 228 to 213) below existing ground surface.

The top 4 to 6 feet of the bedrock in boreholes W-24 and W-38 (east end of route) were found to be fractured with core recoveries in the order of 50 to 70 percent. The remainder of the core recovered appears to be relatively sound with core recoveries generally greater than 90 percent.

The core samples recovered indicate the bedrock to be a grey fine to medium grained limestone which also contains numerous irregular black shale partings and zones of dolomitic limestone. Detailed descriptions of the core recovered are given on the individual Record of Borehole sheets.

#### Groundwater

Standpipes or piezometers were installed in all but two of the boreholes along this route in order to allow measurement of the stabilized groundwater level. Measurements taken about 3 weeks after completion of the drilling indicate a water level at some 9 to 17 feet below existing ground surface.

#### DESIGN AND CONSTRUCTION CONSIDERATIONS

As noted under "Description of Project", two completely separate alignments have been investigated for this Stage 6 of the Hintonburgh West Outlet. The subsurface conditions along these two alignments have been discussed separately. However, since construction of the sewer along either alignment may involve one or more of the following techniques, the discussions which follow, in some cases, apply to both alignments. Based on the inferred stratigraphy along the alternate alignments, it is considered that one or more of the following types of construction will have to be employed:

- 1) Full Rock Face Tunnel

- areas where tunnel would be entirely within bedrock with more than one tunnel diameter of sound bedrock above tunnel crown

2) Soft Ground and Mixed Face Tunnel

- areas where tunnel construction would be carried out entirely within overburden or within a mixed face condition of part overburden and part bedrock; also includes areas where bedrock cover (above crown) is minimal at less than 1 tunnel diameter

3) Open Cut

- areas where open cut construction techniques could be employed.

The notes on design and construction which follow are given separately for each of the above construction techniques. The horizontal limits of each section type are approximate only, being based on the inferred stratigraphy between boreholes, and also being dependent on the finalized invert elevations along the sewer route.

A. Full Rock Face Tunnel

It is considered that the following sections could be carried out entirely within the bedrock with more than 1 tunnel diameter of sound bedrock above the tunnel crown.

1. Carling Avenue Alignment - Clarendon Avenue to Carling Avenue at Merivale Road
2. Carling Avenue Alignment - Archibald Street to Kirkwood Avenue (possibly, depending on invert level)
3. Hampton Park Alignment - Clarendon Avenue to Merivale Road (station 33+40 to 46+50 (approximately))



The boreholes put down along the above three sections indicate that the bedrock is at depths of some 7 to 35 feet below ground surface. Depending on finalized invert levels, these sections of the sewer should be in full rock face with more than 1 tunnel diameter of fairly sound bedrock cover. The bedrock at tunnel level along these sections is indicated to consist of fairly sound to sound grey limestone with some dolomitic bands and impure shaly limestone interbeds. A secondary jointing pattern consisting of near vertical hair-line cracks or fractures, infilled with calcite, was evident throughout the core recovered from the above bedrock formation. In summary, the percent core recovery and the quality of the core indicate that the limestone bedrock at the borehole locations along the full face rock tunnel sections of the route is generally sound at tunnel level.

The sound limestone should stand unsupported during construction without the need of temporary support such as ribs and lagging, liner plate, or the like. However, due to the bedded nature of this limestone formation and due to the secondary jointing pattern, small amounts of loose rock will probably develop on the tunnel roof in localized areas. It should be possible to remove the majority of this loose rock by scaling down the tunnel roof. However, if loose rock of significant area or thickness develops, provision will have to be made for installing rock bolts, wire mesh, or some other type of temporary support in these areas. The number, spacing, and length of rock bolts could best be determined in the field by the contractor.

The rock loads for temporary support along this "sound rock" portion of the tunnel will be essentially zero over the short term as the sound limestone should stand unsupported during construction. Liner plate or rib and lagging support should not be required along these full rock face portions

of the route. It should be pointed out however, that the bedding and jointing pattern of the bedrock, and to a larger extent the amount of sound rock cover, may be expected to vary over the length of the full rock face portion of the tunnel. Although it is not expected that roof support other than some rock bolting will be necessary, provision for the installation of roof support should be made in the contract. Roof support, where required for structural or safety reasons and as indicated by the Engineer, should be provided.

Based on the results of the borings, a major quantity of groundwater seepage into the tunnel is not anticipated in the full rock face sections. In view of the generally "tight" in situ nature of the limestone bedrock, as demonstrated by the drill water return and the pressure packer tests, there should only be minor quantities of seepage water and this will probably be local in nature. The inclined fractures and joints encountered in the core were generally healed and cemented with calcite. It must be anticipated, however, that at least some of these joints or fissures will be open. It is recommended that the water inflow from any open joints or fissures encountered during tunneling be treated on an individual basis and that such joints or fissures be packed and grouted as required. No inflow of soil is anticipated from open joints within the bedrock mass.

Although specific tests were not carried out, no indications of explosive or hazardous gases were noted during core drilling in the limestone formation. However, it should be noted that gases cannot always be detected by core drilling. Although it is not expected that gases will be encountered, a complete air monitoring program should be carried out during tunneling in order to provide for personnel safety and protection.

The design calls for a cast-in-place concrete lining with a good quality finish achieved by placing concrete against steel forms to produce a concrete liner. It is suggested that the following criteria be used for the design of the permanent tunnel liner under sound rock conditions. These criteria are based on the assumption that at least one tunnel diameter of sound bedrock exists above the tunnel crown.

(i) The permanent tunnel liner should be designed to withstand the following vertical rock pressure. Vertical rock pressure =  $0.25 \gamma (B+H)$  where:

$\gamma$  = total unit weight of rock

B = diameter of the driven tunnel

H = height of the driven tunnel

(ii) For a relatively rigid permanent concrete liner, the horizontal (lateral) rock pressure may be taken as 70 percent (0.7) of the vertical rock pressure value.

(iii) During tunneling and immediately following installation of the permanent liner, the hydrostatic pressure around the rock tunnel will be zero and the above loading conditions will apply. However, with time the hydrostatic pressure will build up behind the permanent liner. For long term conditions, the permanent liner should be designed to withstand an all round external hydrostatic pressure computed on the basis of a water level equal to the permanent or stabilized water level following construction. This water level should correspond to the water level measured prior to construction during our subsurface investigation.

#### B. Soft Ground and Mixed Face Tunnel

Based on the inferred stratigraphy shown on Figures 2 and 4 it is considered that the following section of the

sewer would be carried out in either full overburden tunnel or in a mixed face condition.

1. Carling Avenue Alignment - Merivale Road at Carling Avenue to Carling Avenue at Archibald Street (boreholes W-31, W-25, W-30)
2. Hampton Park Alignment - Ottawa Queensway Crossing (boreholes W-28, W-31, and W-40)

West of Merivale Road along Carling Avenue, the boreholes indicate a sharp depression in the bedrock, the bedrock surface falling off to about elevation 180 (64 foot depth) in borehole W-31. Based on proposed invert levels (elevation 180 to 190) the tunnel crown would break rock cover in this area for some 500 to 600 feet. In addition, a zone some 300 to 400 feet long will exist on either side of this where the rock cover above the crown will be minimal. The overburden through which the tunnel would pass consists essentially of silty sand to sandy silt till. However, as outlined under subsurface conditions, and from experience in similar till material on a previous section of this sewer, the glacial till may be expected to vary considerably in texture and composition over short horizontal and vertical distances. Zones of silt, sand, and sand and gravel may be expected. In addition, numerous cobble and boulder size material is indicated throughout the till. Previous experience also suggests that the bedrock surface may vary considerably over short horizontal distances. The stabilized groundwater level within the till is some 9 to 15 feet below ground surface which corresponds to a head of some 40 feet, as measured from tunnel invert. Unless adequate measures are taken to control this water pressure within the till stratum (and especially within the more granular zones

within the till), instability and loss of ground at the tunnel face will occur due to seepage gradients towards the tunnel face. Tunneling in this area may be expected to proceed only with great difficulty and danger unless the overburden materials are first transformed into firm and stable ground. In addition to soil inflow, a significant inflow of water must be anticipated from the pervious zones within the till and along the overburden bedrock contact.

Three special methods of controlling groundwater seepage (and thus limit soil inflow) into the tunnel face have been considered, namely, grouting of the overburden, tunneling under compressed air, and dewatering of the glacial till in the area of the tunnel.

The use of grout to transform the till material into a cohesive soil is not considered feasible as it would be difficult to ensure or even achieve complete injection of all the materials surrounding the tunnel. The danger exists that high seepage pressures may act through a small volume of ungrouted material of fine grain size and may cause an unexpected run at the tunnel face.

Although the use of compressed air would probably prove effective, it is considered that the cost of setting up for tunnelling under air pressure for such a short length of tunnel could make this method unattractive and possibly uneconomical. In addition, abnormally high air pressures would be required to control the water head within the sand and gravel.

Lowering of the groundwater in this area, to the rock surface if possible, will also prove difficult and costly. As a prerequisite, it will be imperative that any dewatering scheme used be designed, installed (if possible) and inspected

by a qualified contractor specializing in dewatering work only. Consultation with dewatering experts has suggested the following scheme.

1) installation of 4 to 5 deep wells south (upstream) of the sewer route to intercept, if possible, some of the recharge flow along the apparent overburden valley. These should be installed well in advance of tunneling.

2) installation of a closely spaced eductor well-point system on both sides of the tunnel as the work proceeds to provide localized drawdown in the immediate tunnel area. These well-points would be operated continuously as tunneling work proceeds.

Although the above combined system, if properly installed, would probably prove effective it will have some disadvantages. Due to the bouldery nature of the till, and due to the fact that at least some of the wellpoints will extend into the bedrock, it would be necessary to employ drilling techniques (as opposed to jetting) to install the wellpoints. The costs therefore would increase accordingly. In addition, this system would require substantial equipment at ground level. The resulting inconvenience in this developed area would tend to make this approach unattractive.

Should tunneling proceed in this area, provision will have to be made for the installation of a continuous full circle temporary support system throughout this mixed face and overburden tunnel section. The temporary support system will have to consist of a continuous system of liner plate, steel ribs and lagging, or the like, and should be designed to withstand full vertical overburden pressure. If the temporary liner is considered to be truly flexible, a lateral load equal to the vertical load may be used in design. It

will be imperative in this case to ensure immediate pressure grouting between the temporary lining and the tunnel walls so that ground movement is limited.

The permanent cast in place concrete liner in this area should be designed on the basis of a vertical load equal to the full weight of overburden above the tunnel and a horizontal (lateral) load equal to 0.7 times the total vertical pressure. For long term conditions, the liner should also be checked against an all round external hydrostatic pressure equal to the present stabilized groundwater level along the route.

The following bulk or total unit weights are suggested for the design of the tunnel along Carling Avenue.

- 1) Surface fills, silty sands, sands, etc.

$$\gamma_t = 120 \text{ lbs./cu. ft.}$$

- 2) Silty Clays

$$- \gamma_t = 100 \text{ lbs./cu. ft.}$$

- 3) Glacial Till, sand & gravel

$$- \gamma_t = 135 \text{ lbs./cu. ft.}$$

- 4) Bedrock

$$- \gamma_t = 170 \text{ lbs./cu. ft.}$$

Along the Hampton Park alignment the sewer will pass under the Ottawa-Queensway in soft ground tunnel. Based on an invert level at about elevation 230, the tunnel will be through deposits of silty sands, sand, sand and gravel, and sandy silt till. The stabilized groundwater level in this area is indicated to be some 4 to 7 feet above the proposed invert level.

The water level should be drawn down below invert level prior to tunneling operations to avoid loss of ground at the tunnel face, and possible subsequent subsidence of the roadway level above. In addition, full temporary support of the

tunnel will be required at all times. It is considered that deep wells installed into, and pumping from, the sand and gravel stratum (see Figure 9) may be the most practical means of lowering the groundwater level in this area. Again, it is recommended that a specialist dewatering contractor be specified for this dewatering project. The effectiveness of the groundwater lowering should be checked by standpipes or piezometers prior to and during tunneling operations.

The permanent concrete liner in this area should be designed on the basis of a vertical load equal to the full weight of overburden above the tunnel and a lateral load equal to 0.7 times the total vertical pressure. Total unit weights, as outlined above, should be used in the calculations.

#### C. Open Cut Section

It is understood that it is proposed to construct the section of the Hampton Park alignment between Merivale Road and the Queensway (station 46+50 to 69+00) in open cut. Based on proposed invert levels, the excavation will be in the order of 20 to 25 feet deep.

Much of the sewer excavation will be through sensitive silty clay of which only the upper few feet has been weathered to a stiff crust. The in-situ shear strength of the lower portions of the clay is in the order of 700 to 1,000 pounds per square foot only and the water content is generally well above the liquid limit value. It is considered that the sides of the open cut in the clay should in general be stable, provided that the clay is not unduly disturbed and provided that surcharge loading is not applied along the sides of the excavation. Stockpiling of excavated or backfill material along the sides of the excavation must not be allowed and the excavating machinery should operate at the end of the excavations only.



The silty clay is underlain along the route by sandy silt till. Towards Carling Avenue the excavations will extend into the till while towards Merivale Road the surface of the till will be some 5 to 6 feet below invert level. The hydrostatic water level within the till was found to rise some 12 to 20 feet above the proposed invert level. This excess pressure may cause bottom heave of the trench when the weight of clay left in place is not sufficient to overcome the water pressure. In order to ensure bottom stability, it is recommended that this excess hydrostatic pressure be relieved prior to excavating. Installation of relief wells extending some 10 feet into the till should be effective in bleeding off this excess pressure. These wells could consist of a wick of crushed stone or (alternatively) a wellpoint surrounded by a sand filter and installed, on a staggered arrangement at about 50 foot centres, along the excavation. The effectiveness of these relief wells should be monitored by piezometers as excavation proceeds to invert grade.

Where the sewer pipe will rest on glacial till, normal pipe bedding may be used. However, where founded on the sensitive clay, it is recommended that a minimum 12 inch thick layer of well-graded sand and gravel (M.T.C. granular "A" or equivalent) be placed on the clay in order to limit "punching in" of the bedding material into the clay subgrade.

The water content of the clay, especially the grey clay, is well above optimum and as such it would not be possible to adequately compact this clay as backfill to the trenches. The grey clay must not be used in areas of present or future roads and construction. Weather conditions permitting, it should however, be possible to use the grey-brown weathered crust of clay as trench backfill. All backfills should be

compacted to at least 95 percent of their respective Standard Proctor dry density value.

#### D. Shafts

Depending on the alignment chosen, some 3 to 4 temporary access and permanent shafts will be required. Following is a summarized account of the construction considerations which will have to be recognized in advancing the various shafts to depth.

- 1) Fills, Sands, Tills above the water table  
Conventional braced soldier piles and lagging would be an acceptable construction technique from the point of view of controlling "loss of ground".
- 2) Silty Sands, Sands, and Tills below the water table  
Control of the groundwater will be essential and may be achieved by either a suitable cut off or by lowering the groundwater level prior to excavating. Interlocking steel sheet piling driven to the surface of the bedrock would be a suitable method of forming a cut off. However, it may prove impractical to drive interlocking sheeting through some of the bouldery tills at this site without loosing the interlock and thus the groundwater control capability of the sheeting. In this case it will probably be necessary to lower the groundwater level and then carry the excavation to depth using conventional soldier pile and lagging support.
- 3) Silty Clays  
Either soldier piles and lagging or interlocking sheeting will be required in sinking shafts through the silty clay. In borehole W-3 (Merivale at Carling) for instance, the silty clay is stiff and it should be possible to carry out construction of the shaft using soldier piles and lagging.

However, at borehole W-39 (Hampton Park alignment) the clay at depth is only firm and basal instability of a shaft in this area may be a problem. In this case it will be necessary to install interlocking sheeting to control loss of ground and basal instability of the shaft.

4) Bedrock

Vertical shafts in the limestone bedrock should stand unsupported during construction. However, temporary support of the shaft walls will likely be required where the upper zone of the bedrock is fractured.

5) Earth Pressures

Earth pressures on the temporary braced excavations should be computed on the basis of an active earth pressure  $K_a$  of 0.3 when in granular soils. In clays, earth pressures may be calculated on the basis of an average pressure equivalent to  $\gamma H - 2C$ , where  $C$  is the average shear strength of the clay at the shaft location. Earth pressures thus calculated should be redistributed as a rectangle. The permanent shaft walls should be designed using an at rest earth pressure coefficient  $K_0 = 0.5$  (assuming granular backfill). The water pressure against the walls should also be added to the computed submerged lateral earth pressure.

Since the actual earth pressures on the shaft walls will be dependent not only on the soil conditions, but also on the geometry and size of the shafts, we would be pleased to provide you with detailed pressure calculations once the location and sizing of the various shafts has been finalized.

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TABLE 1

SUMMARY OF COMPRESSIVE STRENGTH  
TESTS ON BEDROCK CORE

<u>Borehole No.</u>	<u>Depth to Test (feet)</u>	<u>Unit Weight of core (lb./cu.ft.)</u>	<u>Compressive strength (lb./sq.in.)</u>
W-24	48.9 - 49.4	172.8	9214
	56.0 - 56.6	175.7	8677
W-27	49.0 - 49.7	171.3	12,183
	53.7 - 54.2	170.7	13,124
	57.2 - 57.7	173.8	10,669
W-28	40.0 - 40.4	173.4	13,780
	46.6 - 47.4	167.8	12,364
	57.2 - 57.7	175.0	7893
	58.3 - 58.7	173.1	5304
W-29	42.0 - 42.6	171.5	5092
	57.7 - 58.4	172.7	5867

TABLE 2

SUMMARY OF PRESSURE PACKER  
TESTS ON BEDROCK

<u>Borehole No.</u>	<u>Depth of Test (feet)</u>	<u>Maximum Pressure Head (ft. of water)</u>	<u>Water Take (gal./min.)</u>	<u>Coefficient of Permeability (cm./sec.)</u>
W-28	33.9 - 49.9	128.3	0.025	$1 \times 10^{-6}$
	49.9 - 60.9	128.3	0.015	$1 \times 10^{-6}$
W-29	20.5 - 40.5	127.3	0.25	$5 \times 10^{-6}$
	45.0 - 60.0	104.2	0.70	$3 \times 10^{-5}$
W-30	35.0 - 50.0	106.4	1.04	$5 \times 10^{-5}$

## LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

### I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample

### II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

*WH* sampler advanced by static weight—weight, hammer

*PH* sampler advanced by pressure—pressure, hydraulic

*PM* sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils

<i>Consistency</i>	<i>c<sub>u</sub>, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

### IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer <sup>1</sup>
<i>Q</i>	undrained triaxial <sup>2</sup>
<i>R</i>	consolidated undrained triaxial <sup>2</sup>
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test

#### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>Undrained triaxial tests in which pore pressures are measured are shown as  $\bar{Q}$  or  $\bar{R}$ .

## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 3.1416
$e$	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of $a$
$\log_{10} a$ or $\log a$	logarithm of $a$ to base 10
$t$	time
$g$	acceleration due to gravity
$V$	volume
$W$	weight
$M$	moment
$F$	factor of safety

### II. STRESS AND STRAIN

$u$	pore pressure
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	unit weight of submerged soil
$G_s$	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
$e$	void ratio
$n$	porosity
$w$	water content
$S_r$	degree of saturation

#### (b) Consistency

$w_L$	liquid limit
$w_P$	plastic limit
$I_P$	plasticity index
$w_S$	shrinkage limit
$I_L$	liquidity index = $(w - w_P) / I_P$
$I_C$	consistency index = $(w_L - w) / I_P$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$D_r$	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

#### (c) Permeability

$h$	hydraulic head or potential
$q$	rate of discharge
$v$	velocity of flow
$i$	hydraulic gradient
$k$	coefficient of permeability
$j$	seepage force per unit volume

#### (d) Consolidation (one-dimensional)

$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$c_v$	coefficient of consolidation
$T_v$	time factor = $c_v t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_r$	sensitivity

in terms of effective stress  
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress  
 $\tau_f = c_u + \sigma \tan \phi_u$

\*For the case of a saturated cohesive soil,  $\phi_u = 0$  and the undrained shear strength  $\tau_f = c_u$  is taken as half the undrained compressive strength.

# RECORD OF BOREHOLE W 24

LOCATION See Figure 1 & 2

BORING DATE MARCH 18 & 19, 1975

DATUM CLODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, K., CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.											
								20	40	60	80	1x10	1x10	1x10	1x10		
WASH BORING NX CASING EXCASING	234.3 0.0	GROUND SURFACE ASPHALT															
	232.8 1.5	LOOSE GREY TO BROWN SANDY SILT WITH CLAY AND GRAVEL (FILL)		1	DO	7											
	224.3 12.0	COMPACT TO DENSE BROWN TO GREY SANDY SILT WITH GRAVEL, TRACE CLAY, SOME BOULDERS (TILL)		2	DO	38											
	220.3 16.0	FRACTURED GREY LIMESTONE BEDROCK		4	RC	-	59	100	58								
				5	"	-	80		0								
				6	"	-	90		63								
				7	"	-	99		53								
				8	"	-	100		28								
				9	"	-	99		52								
				10	"	-			25								
				11	"	-			51								
				12	"	-			79								
				13	"	-			85								
				14	"	-			80								
	176.9 60.3	END OF HOLE															

GROUND SURFACE

BENTONITE SEAL

PLASTIC TUBING

NATIVE BACKFILL

STANDPIPE

W.L. IN STANDPIPE AT ELEV. 227.0 APRIL 27, 1975

NOTES ON CORE

BEDDING GENERALLY HORIZONTAL BUT LOCALLY DISTORTED WHERE HEAVILY INTERBEDDED WITH SHALE

FOSSILIFEROUS TO 28 FOOT DEPTH

FRACTURES IRREGULAR BUT MAINLY HORIZONTAL ALONG BEDDING PLANES

SANDY TEXTURE SHALE BEDS TO 1"

SOME DOLOMITE BANDS

CORE LENGTHS  
MIN. 1"  
MAX. 14"  
AVG. 8"

% CORE RECOVERY

EST. % DRILL WATER RETURN

ROCK QUALITY DESIGNATION (RQD)

15-10 Percent axial strain at failure

VERTICAL SCALE  
1 IN TO 5 FT.

Golder Associates

DRAWN H.R. ECF  
CHECKED RAM



DATUM GEODETIC

PENETRATION TEST HAMMER WEIGHT 140 LB. DROP 30 IN.

DRAWN H. REGT  
CHECKED BN

# RECORD OF BOREHOLE W 26

LOCATION See Figure 1

BORING DATE MARCH 24 & 25, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20		40		60		80			
								SHEAR STRENGTH $C_u$ , LB./SQ. FT.		NAT. V. - + Q. - ● REM.V. - ⊕ U. - ○		WATER CONTENT, PERCENT $W_p$ $W$ $W_L$					
WASH BORING NX CASING	245.7	GROUND SURFACE					250										
	0.0	TOP SOIL		1	DO	21	245										
	0.4	LOOSE BROWN FINE TO MEDIUM SAND WITH SILT (FILL)		2	"	8											
	238.7	LOOSE BROWN SILTY SAND WITH GRAVEL		3	"	6	240										
	235.3	FIRM GREY SILTY CLAY		4	"	2	235										
	12.3	LOOSE GREY SANDY SILT WITH CLAY AND GRAVEL (TILL)		5	"	6	230										
	228.2	VERY DENSE GREY SANDY SILT TO SILTY SAND WITH GRAVEL SOME CLAY COBBLES AND BOULDERS (TILL)		6	"	56	225										
	17.5	(NOTE: NUMEROUS COBBLES AND BOULDERS FROM 29' - 34.5' FOOT DEPTH)		7	"	54	220										
				8	"	440	215										
				9	BX RC	-		57	100	0							
ROTARY DRILLING BX CASING	210.8			10	"	-	210										
	34.9	SOUND GREY FINE TO MEDIUM GRAINED LIMESTONE BEDROCK SOME SHALY AND DOLOMITE BANDS		11	"	-	205										
				12	"	-	200										
				13	"	-	195										
				14	"	-	190										
				15	"	-	185										
BX CORE	186.2	END OF HOLE					185										

NOTES ON CORE

63 BEDDING NEAR HORIZONTAL FRACTURES MAINLY HORIZONTAL ALONG BEDDING PLANES

63

SOME VERTICAL CALCITE HEALED FRACTURES

62

FOSSILIFEROUS, WAVY SHALE BANDS

0 AT 39'-41' FINE GRAINED DOLOMITIC LIMESTONE

CORE LENGTHS

MIN - 1'

MAX - 8'

AVG - 3'

% CORE RECOVERY

99

99

100

94

97

EST. % DRILL WATER RETURN

100

25

0

0

ROCK QUALITY DESIGNATION (%)

63

62

0

36

15 0 5 Percent axial strain at failure

# RECORD OF BOREHOLE W 27

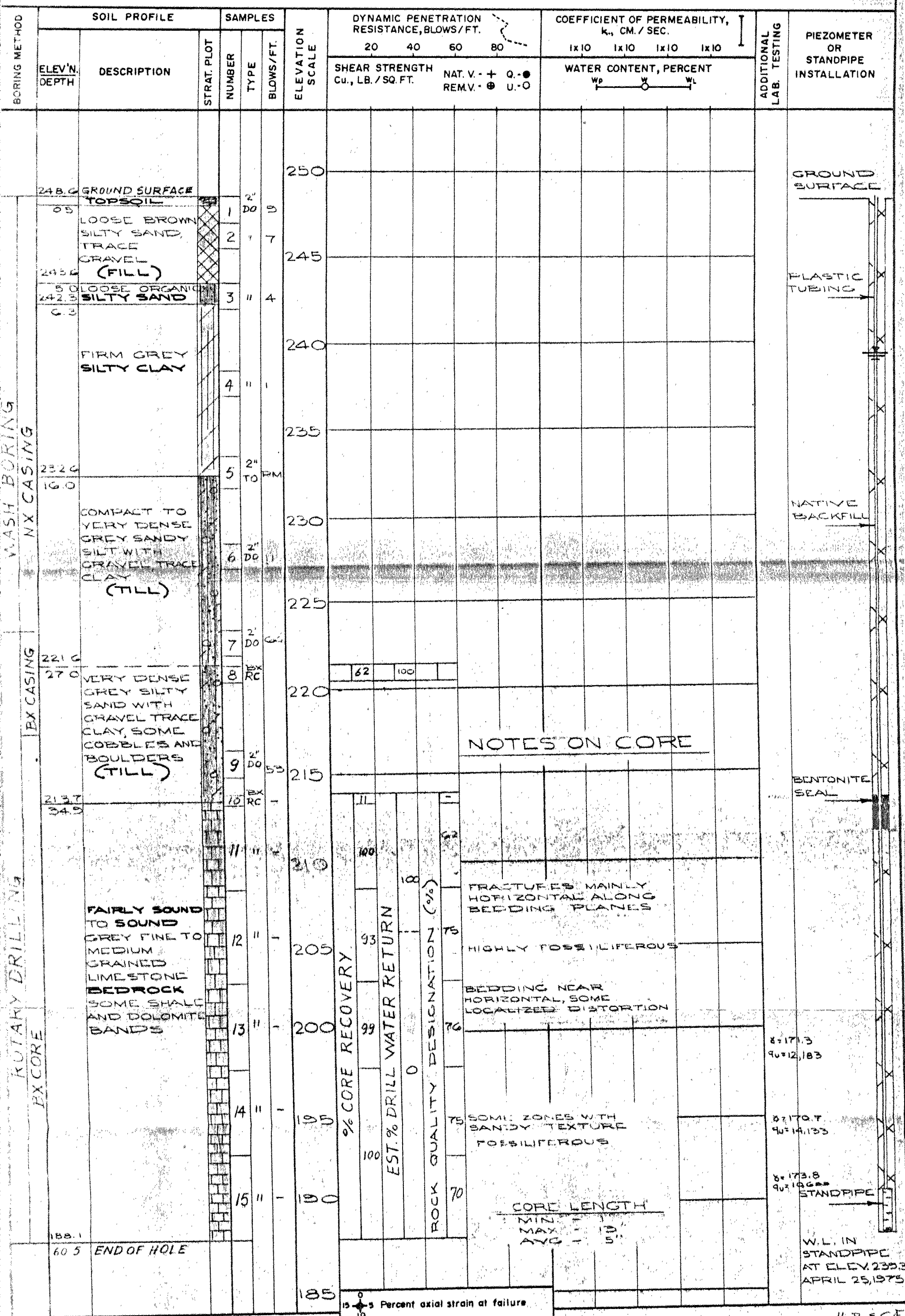
LOCATION See Figure 1

BORING DATE MARCH 20 & 21, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



VERTICAL SCALE

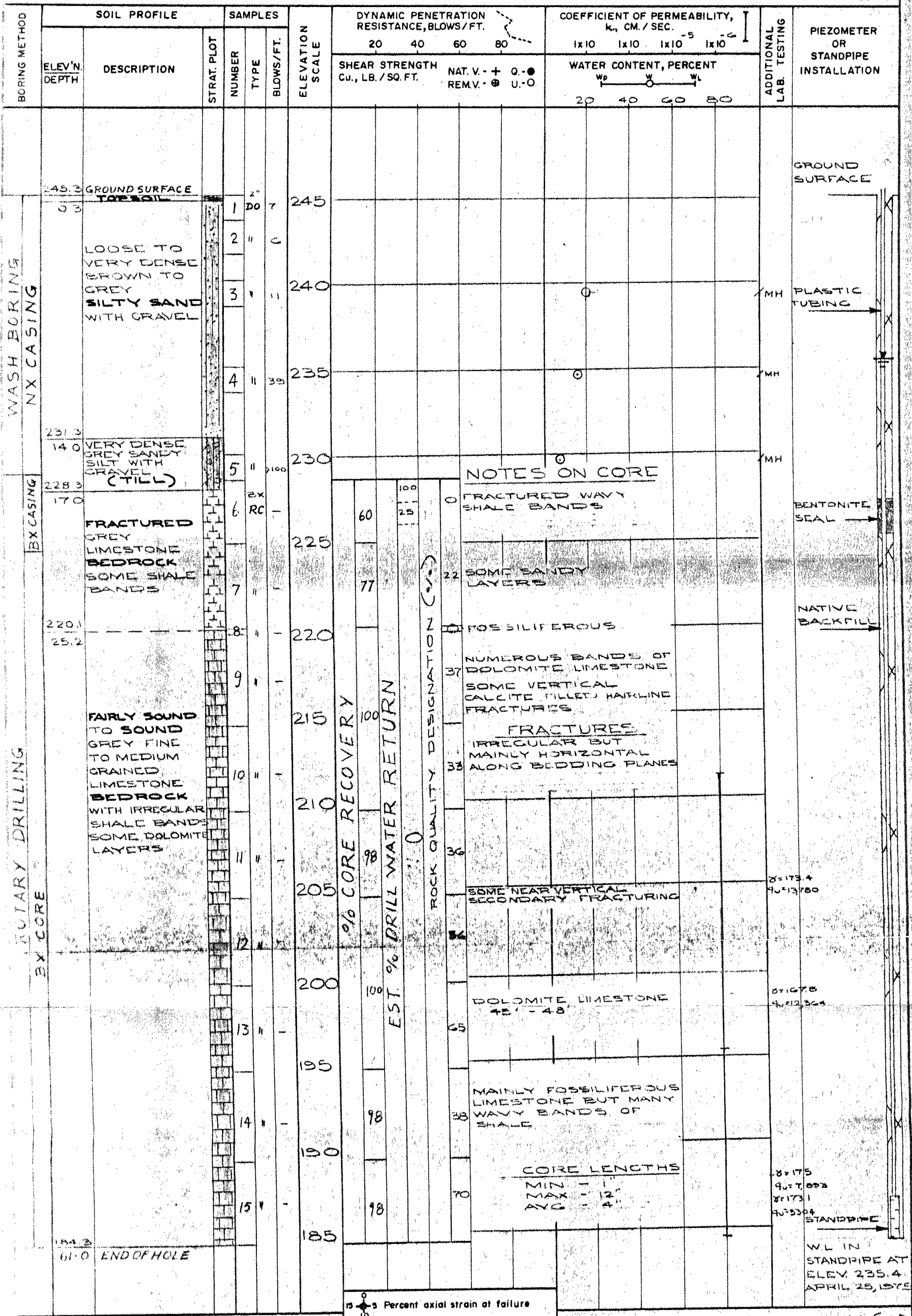
Golden Associates

DRAWN H.R. EGF  
CHECKED EGF



# RECORD OF BOREHOLE W 28

LOCATION See Figure 1 & 3 BORING DATE MARCH 31 & APRIL 1, 1975 DATUM GEODETIC  
 SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN. PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



# RECORD OF BOREHOLE W 29

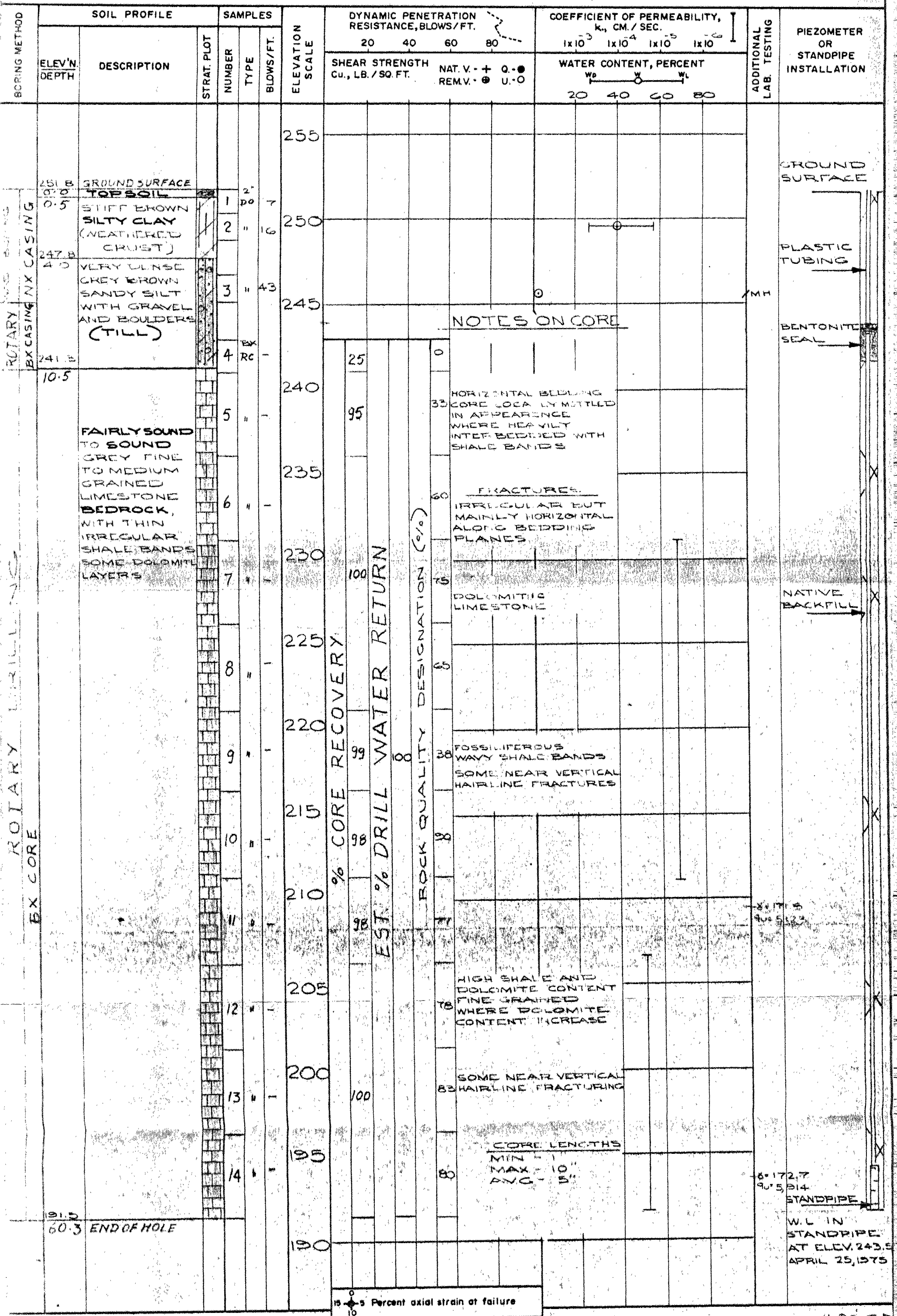
LOCATION See Figure

BORING DATE APRIL 23, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



15 5 Percent axial strain at failure

DRAWN H. RECA

VERTICAL SCALE

RECORD OF BOREHOLE W30

LOCATION See Figure

BORING DATE APRIL 25, 1975

DATUM COLUMBIAN TIDE

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20	40	60	80	$1 \times 10^{-5}$	$1 \times 10^{-4}$	$1 \times 10^{-3}$	$1 \times 10^{-2}$		
								SHEAR STRENGTH $C_u$ , LB./SQ. FT.				NAT. V. - + Q - • REM. V. - • U - O					
ROTARY DRILL EXCAVATION		ASPHALT					245									GROUND SURFACE  PLASTIC PIPE  NATIVE BACKFILL  STANDPIPE  W.L. IN STANDPIPE AT ELEV 232.4 APRIL 25, 1975	
		(FILL)		1	DC	12	240										
		CLAY		2	"	6	235										
				3	"	28	230										
				4	"	22	225										
				5	"	16	220										
				6	"	44	215										
				7	RC	-	210										
				8	"	-	205										
				9	"	-	200										
				10	"	-	195										
				11	"	-	190										
				12	"	-	185										

NOTES ON CORE

FRACTION IRREGULAR BUT MAINLY HORIZONTAL ALONG BEDDING PLANES

CORE IS GENERALLY MOIST OR WET IN APPEARANCE WHERE INTERBEDDED WITH SHALE SOME AREA IS HIGHLY FOSSETEROUS

CORE LENGTHS  
MAX - 4"  
MIN - 2"

15 5 Percent axial strain at failure

VERTICAL SCALE  
1 IN TO 5 FT.

Golder Associates

DRAWN H. K. ECF  
CHECKED E. S. H.

# OVERSIZE DRAWING

RECORD OF BOREHOLE W-32

LOCATION      See Figure

BORING DATE JUNE 3, 1972

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM./ SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV./N. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FT.		20 40 60 80				1x10 1x10 1x10 1x10					
								SHEAR STRENGTH $C_u$ , LB./SQ. FT.		NAT. V. - + 0 - ● REM. V. - ⊕ U - ○		WATER CONTENT, PERCENT					
								500 1000 1500 2000				$w_p$ — $w$ — $w_L$ 20 40 60 80					
	251.4	GROUND SURFACE					255								GROUND SURFACE		
	248.5	DARK BROWN SILTY SANDS TRACE GRAVEL ORGANICS (FILL)					250								PLASTIC TUBING		
	245.4	LOOSE BROWN SILTY SAND		1	2"	5	245										
	240.5	VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)		2	"	2	240								NATIVE BACKFILL		
	235.5	FIRM GREY SILTY CLAY OCCASIONAL THIN SAND SEAMS, SOME SHELLS		3	"	WH	235	⊕	+								
	230.5			4	"	WH	230	⊕	+						BENTONITE SEAL		
	227.5			5	"	25	225	⊕	+						PIEZOMETER		
	222.5	COMPACT TO VERY DENSE GREY SANDY SILT WITH SOME CLAY AND GRAVEL OCCASIONAL SAND AND GRAVEL POCKET (TILL)		6	"	81	220								PAVED IN MATERIAL		
	215.6	END OF HOLE		7	"	140	215								WL IN PIEZOMETER AT ELEV. 247.1 JULY 2 1975		
							210										

15 0 5 Percent axial strain at failure

### VERTICAL SCALE

**Golder Associates**

DRAWN           
CHECKED



# RECORD OF BOREHOLE W-33

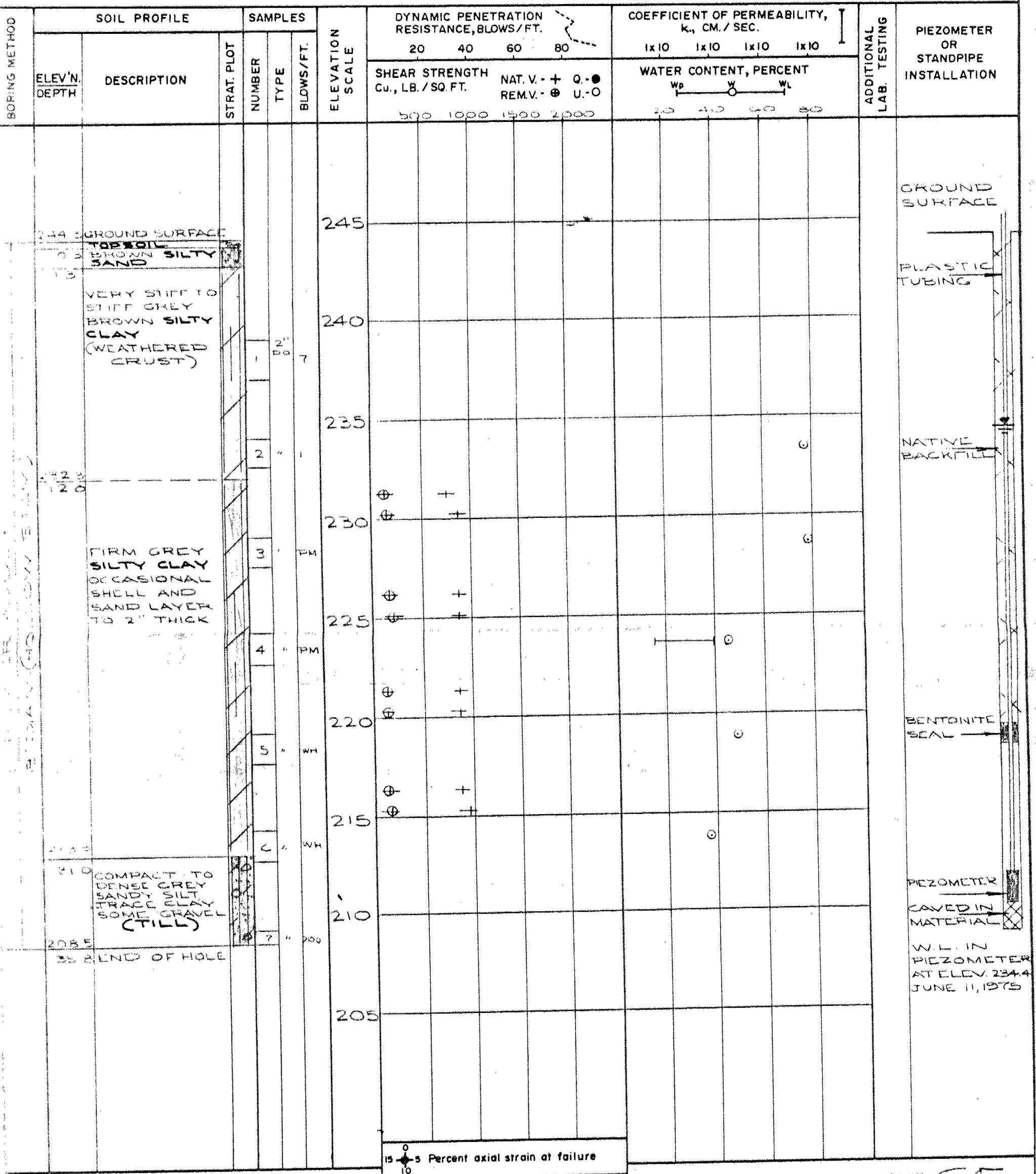
LOCATION See Figure 3

BORING DATE JUNE 3, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

DRAWN *SE*  
CHECKED *EPW*

## RECORD OF BOREHOLE W. 34

LOCATION	See Figure
1	1
2	2
3	3
4	4
5	5
6	6
7	7
8	8
9	9
10	10
11	11
12	12
13	13
14	14
15	15
16	16
17	17
18	18
19	19
20	20
21	21
22	22
23	23
24	24
25	25
26	26
27	27
28	28
29	29
30	30
31	31
32	32
33	33
34	34
35	35
36	36
37	37
38	38
39	39
40	40
41	41
42	42
43	43
44	44
45	45
46	46
47	47
48	48
49	49
50	50
51	51
52	52
53	53
54	54
55	55
56	56
57	57
58	58
59	59
60	60
61	61
62	62
63	63
64	64
65	65
66	66
67	67
68	68
69	69
70	70
71	71
72	72
73	73
74	74
75	75
76	76
77	77
78	78
79	79
80	80
81	81
82	82
83	83
84	84
85	85
86	86
87	87
88	88
89	89
90	90
91	91
92	92
93	93
94	94
95	95
96	96
97	97
98	98
99	99
100	100

BORING DATE JUNE 24 1975

DATUM 660617Z

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

[illegible]

VERTICAL SCALE  
1 IN. TO 1 FT.

## Goldier Associates

DRAWN CF  
CHECKED RFH

# RECORD OF BOREHOLE W-35

LOCATION See Figure 3

BORING DATE JUNE 4, 1975

DATUM GROUND SURFACE

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH $C_u$ , LB./SQ. FT.				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10	1x10		
	255.1	GROUND SURFACE															
	254.6	TOP SOIL															
	254.6	LOOSE BROWN SILTY SAND (FILL)															
	254.6	LOOSE BROWN SILTY SAND															
	247.1	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	244.1	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	240.0	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	235.0	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	230.0	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	225.0	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	220.0	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	217.1	STIFF GREY BROWN SILTY CLAY (WEATHERED)															
	215.0	STIFF GREY BROWN SILTY CLAY (WEATHERED)															

GROUND SURFACE

PLASTIC TUBING

NATIVE BACKFILL

BENTONITE SEAL

PIEZOMETER

W.L. IN PIEZOMETER AT ELEV. 236.6 JULY 2, 1975

0 5 10 Percent axial strain at failure

VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

DRAWN *G.F.*  
CHECKED *Ph*

# RECORD OF BOREHOLE W-36

LOCATION See Figure

BORING DATE JUN 5, 1965

DATUM C.T. 100.00

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.											
								20	40	60	80	1x10	1x10	1x10	1x10		
		TOP SOIL															
	247.5	GROUND SURFACE														GROUND SURFACE	
	247.5	LARGE BROWN LUMPY SAND (FILL)															
	250	1.5 TO 2.0 BROWN SILTY SAND		2	DO	7											
	245	STIFF GREY BROWN SILTY CLAY (SOME FIBERED CRUST)		2		1										NATIVE BACKFILL	
	240	FIRM GREY SILTY CLAY					+	+									
	235						+	+								PENTONITE SEAL	
	230	VERY LOOSE TO COMPACT GREY SANDY SILT TO SILTY SAND TRACE CLAY SOME GRAVEL COBBLES AND Boulders (TILL)															
	225															PIEZOMETER	
	220																
	215	END OF HOLE														WATER PIEZOMETER AT ELEV. 247.5 JULY 2, 1965	

0 5 10 Percent axial strain at failure

VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

DRAWN *GP*  
CHECKED *PH*

# RECORD OF BOREHOLE W-37

LOCATION See Figure

BORING DATE JUNE 24, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, K, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV./N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH Cu., LB./SQ.FT.				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10	1x10		
	249.5	GROUND SURFACE															
	248.5	TOP SOIL															
	247.5	VERY LOOSE BROWN SILTY SAND, SOME MEDIUM SAND LAYERS		1	DO	3											
	246.5	STIFF GREY BROWN SILTY CLAY		2	"	4											
	245.5	VERY LOOSE TO GREY SILTY SILT, SOME CLAY AND GRAVEL (TILL)		3	"	5											
	244.5	VERY DENSE SAND, GRAVEL BOULDERS WITH DEPTH		4	"	12											
	243.5			5	"	33											
	242.5			6	"	1											
	241.5	FAIRLY SOUND GREY LIMESTONE BEDROCK, SOME SHALE BANDS		7	"	1											
	240.5	END OF HOLE															

**NOTES ON CORE**

SHALY, POSSIBLY FOLDED, SOME NEAR VERTICAL FRACTURES

**CORE LENGTHS**

MIN - 1"

MAX - 2"

AVG - 4"

ROD (10%)

EST DRILL WATER RETURN (%)

CORE RECOVERY (%)

0  
15  
10

5 Percent axial strain at failure

GROUND SURFACE

PLASTIC TUBING

BENTONITE SEAL

NATIVE BACKFILL

PIEZOMETER

WELL IN PIEZOMETER REELLY 2361 JULY 2, 1975

VERTICAL SCALE  
IN TO FT.

Golder Associates

DRAWN *G.F.*  
CHECKED *RAH*

# RECORD OF BOREHOLE W-38

LOCATION See Figure 3

BORING DATE JUNE 5, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.				COEFFICIENT OF PERMEABILITY, $k_v$ , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLT.	NUMBER	TYPE	BLOWS/FT.											
								SHEAR STRENGTH $C_u$ , LB./SQ. FT.				WATER CONTENT, PERCENT					
	245	GROUND SURFACE															
	240	TOP SOIL DARK BROWN SANDY SILT WITH BOULDER DEBRIS (FILL)															
	235	CLAYEY SILT															
	230	TOO SOIL TO DETIDE BROWN SANDY SILT, SOME CLAY AND GRAVEL NUMEROUS BOULDERS (TILL)		1	EX												
	225	FRACTURED GREY LIMESTONE BEDROCK		2	RC												
	220	FAIRLY SOUND GREY LIMESTONE BEDROCK		3													
	215	END OF HOLE		4													
	210			5													
				6													
				7													
				8													
				9													
				10													

**NOTES ON CORE**

BEDDING  
LOCALLY DISTORTED  
FRACTURES  
IRREGULAR  
FOSSILIFEROUS  
WITH WAVY  
SHALE BANDS

**CORE LENGTHS**

MIN - 1"  
MAX - 5"  
AVG - 2"

R.O.T. (0%)  
EST. DRILL WATER RETURN (0%)  
CORE RECOVERY (0%)

15 0 5 Percent axial strain at failure

 VERTICAL SCALE  
1 IN. TO 25 FT.

Golder Associates

 DRAWN *GF*  
CHECKED *PRJ*

# OVERSIZE DRAWING

# RECORD OF BOREHOLE W - 40

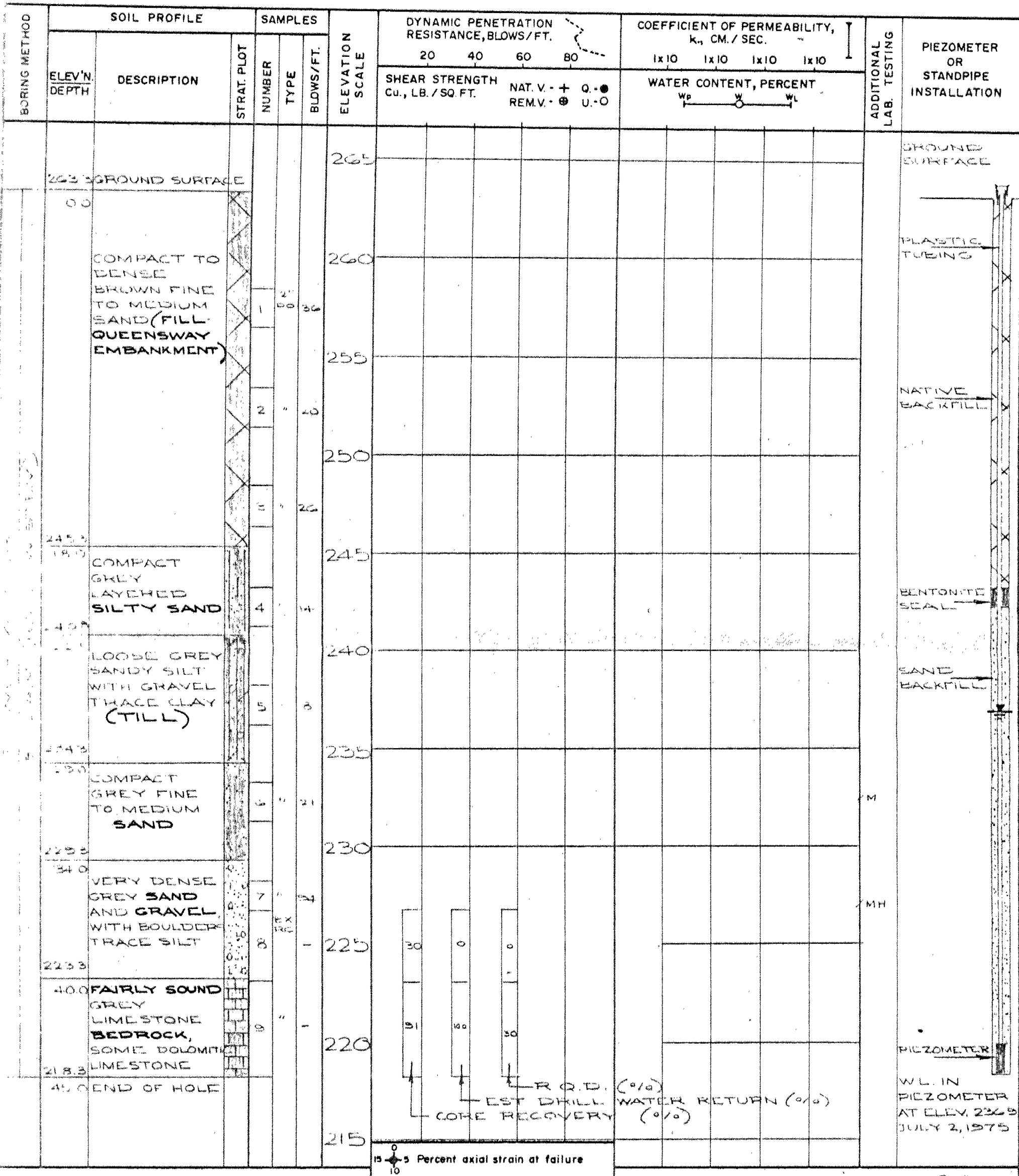
LOCATION See Figure 3

BORING DATE JUNE 23, 1975

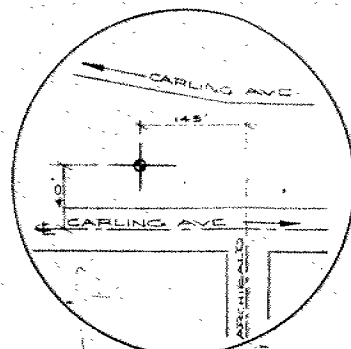
DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

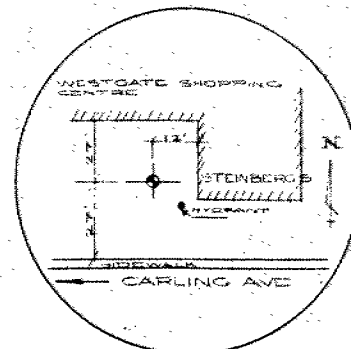
PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.



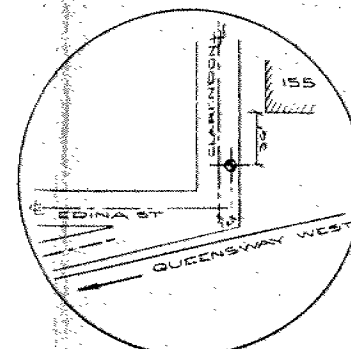




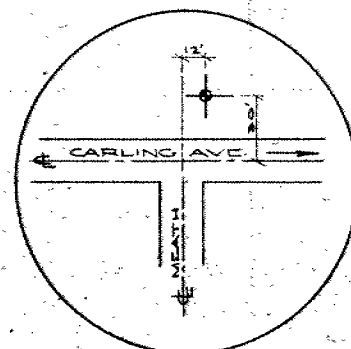
LOCATION OF B.H. W-26  
N.T.S.



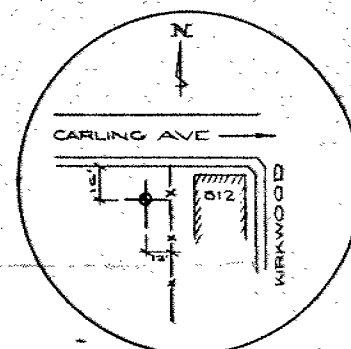
LOCATION OF B.H. W-25  
N.T.S.



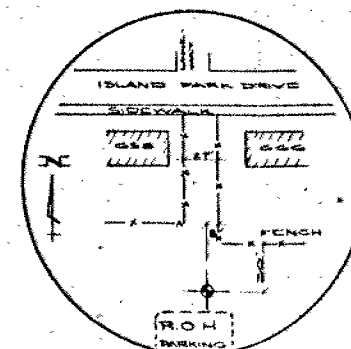
LOCATION OF B.H. W-24  
N.T.S.



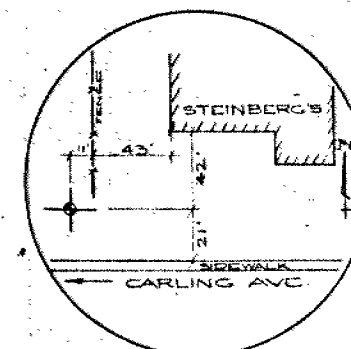
LOCATION OF B.H. W-27  
N.T.S.



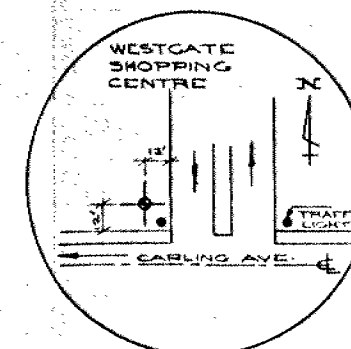
LOCATION OF B.H. W-28  
N.T.S.



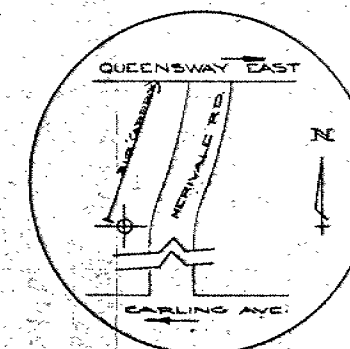
LOCATION OF B.H. W-29  
N.T.S.



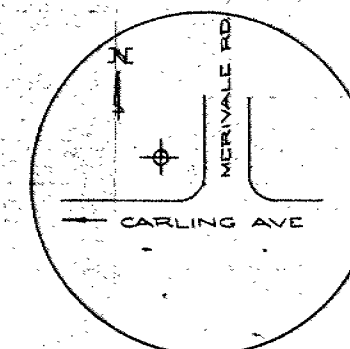
LOCATION OF B.H. W-30  
N.T.S.



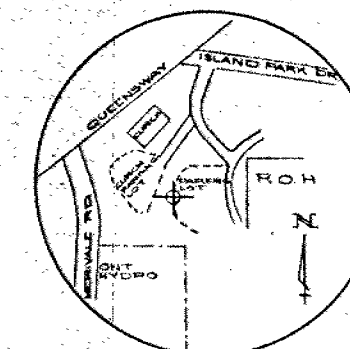
LOCATION OF B.H. W-31  
N.T.S.



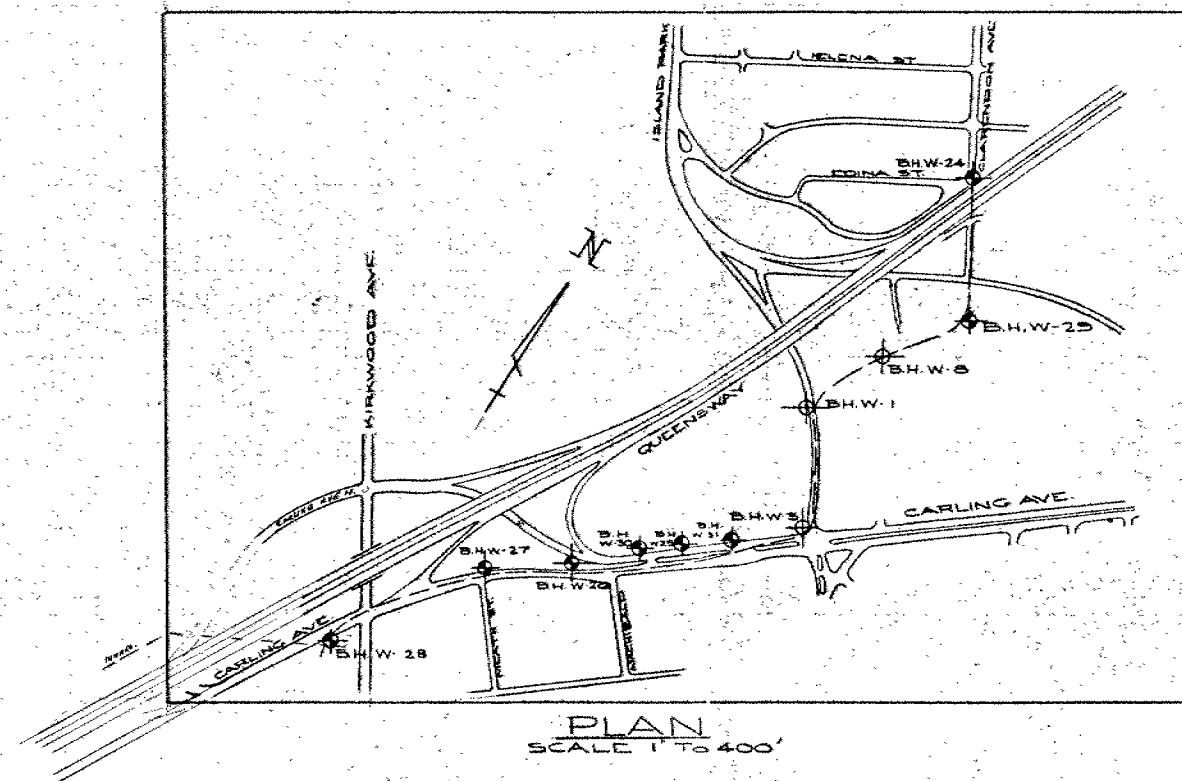
LOCATION OF B.H. W-1  
N.T.S.



LOCATION OF B.H. W-2  
N.T.S.



LOCATION OF B.H. W-3  
N.T.S.



**LEGEND**

- BORING IN PLAN  
PRESENT INVESTIGATION
- BORING IN PLAN  
PREVIOUS INVESTIGATION  
REPORT #70794

**REFERENCE:**  
CITY OF OTTAWA,  
CENTRAL REGISTRY,  
UNDERGROUND SERVICES  
PLANS SUPPLIED BY  
CITY OF OTTAWA  
ENGINEERING DEPARTMENT

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

# OVERSIZE DRAWING

**DeLCan**

DE LEUW CATHER, CANADA LTD.

CONSULTING ENGINEERS AND PLANNERS

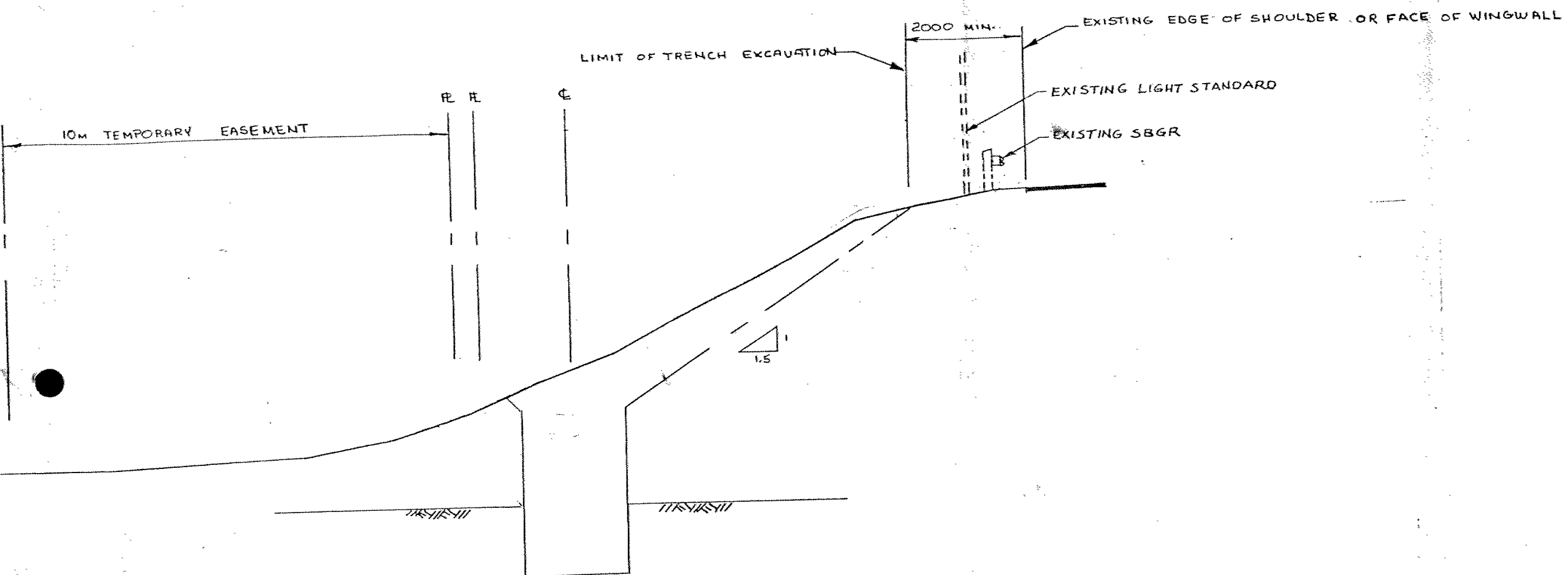
- OTTAWA

REG MOLARO

PROPOSED METHOD OF  
EXCAVATION FOR SEWER ALONG  
NORTH R.O.W. LIMIT WEST  
OF CARLING AVE. FOR YOUR  
COMMENTS & APPROVAL.

---

From: Paul G. Sharpe  
(613) 733-4160  
Telex: 053-4152

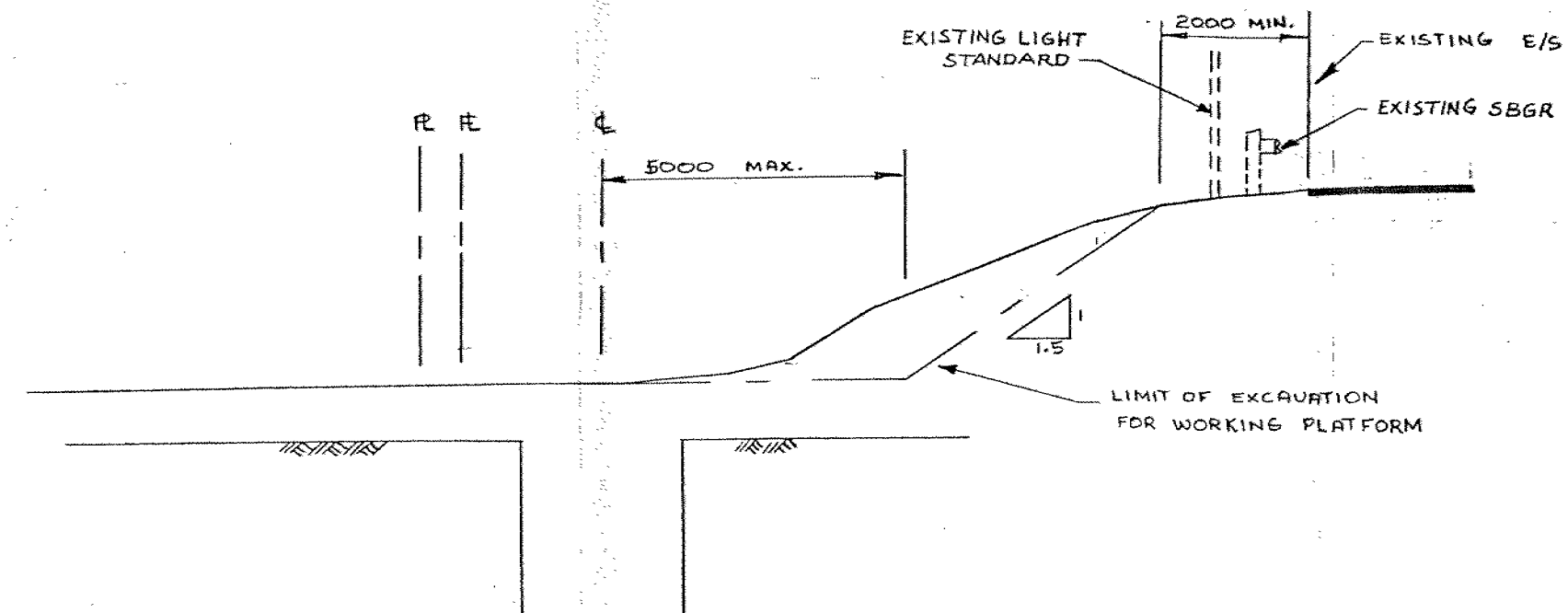


TYPICAL SECTION - TRUNK SEWER - WITH EASEMENT

STA. 22+655 TO STA. 22+745

STA. 22+767 TO STA. 22+925

STA. 23+310 TO STA. 23+455



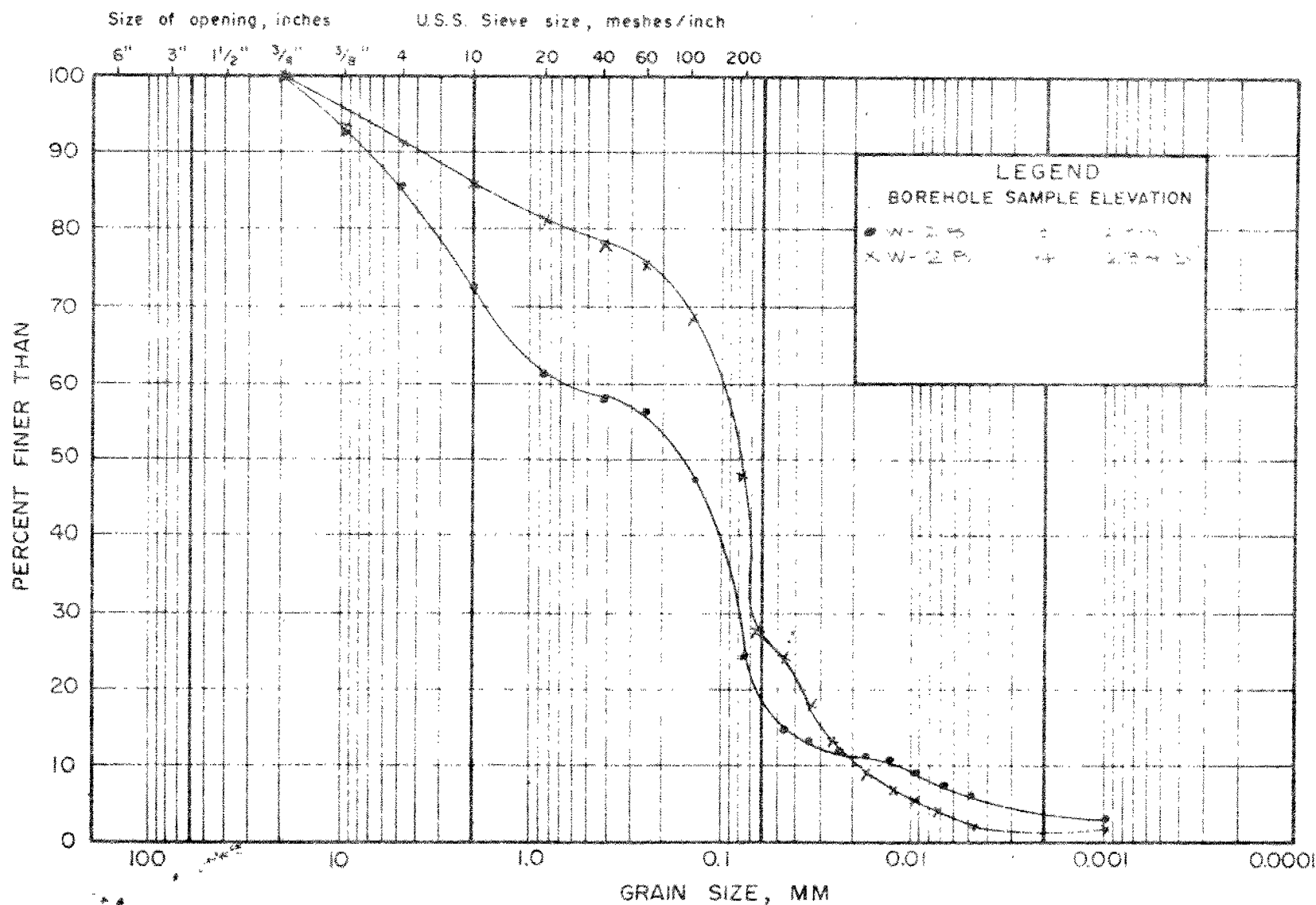
22+925- 23+000 E.C. - 19  
E.C.S. - 9

23+250 - 23+310 E.C. - 10  
E.C.S. - 1

TYPICAL SECTION - TRUNK SEWER. WITHOUT EASEMENT

STA. 22+925 TO STA. 23+000  
STA. 23+250 TO STA. 23+310

M.I.T. GRAIN SIZE SCALE

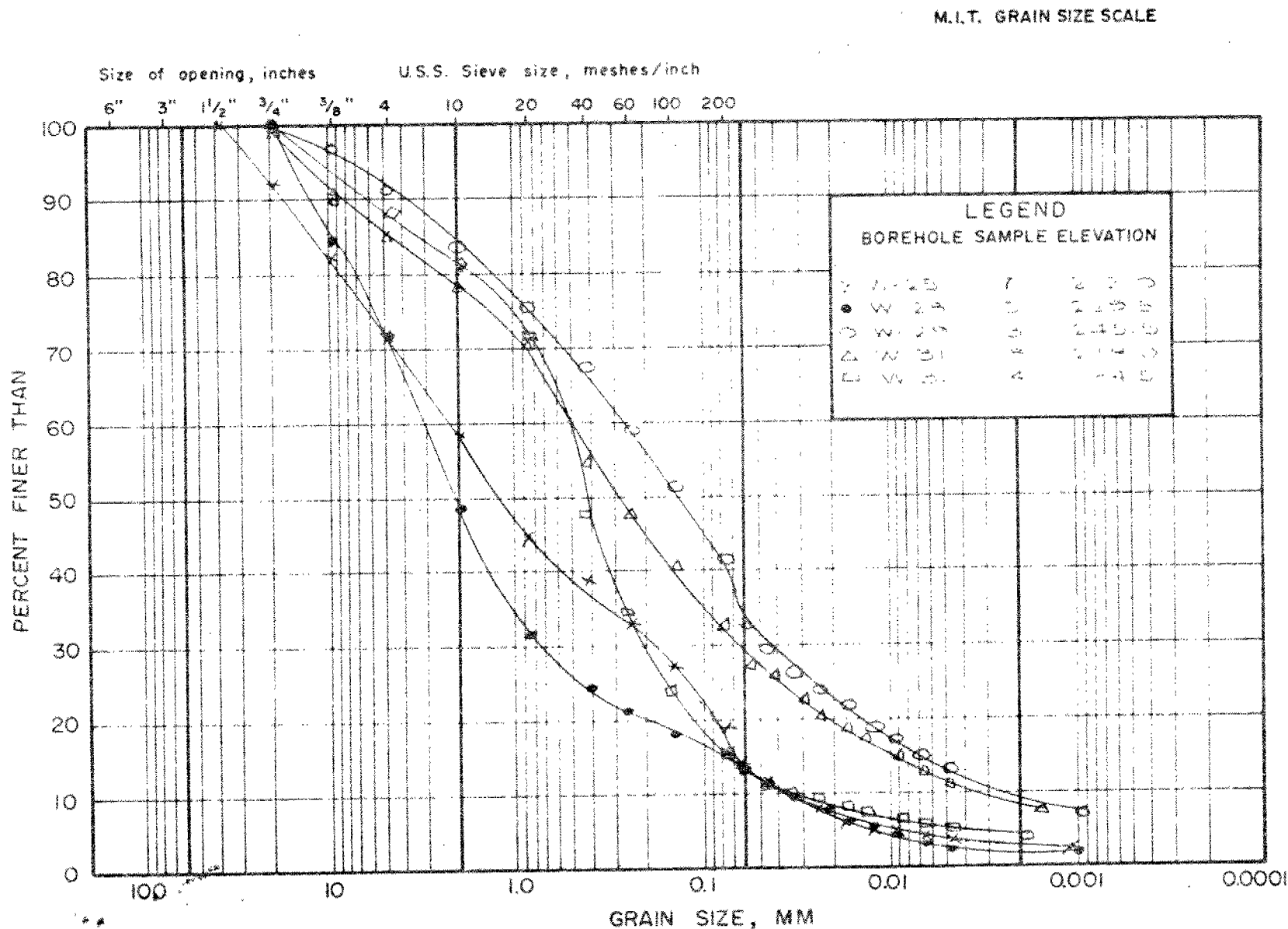


COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

SILTY SANDS WITH GRAVEL

GRAIN SIZE DISTRIBUTION

FIGURE 10



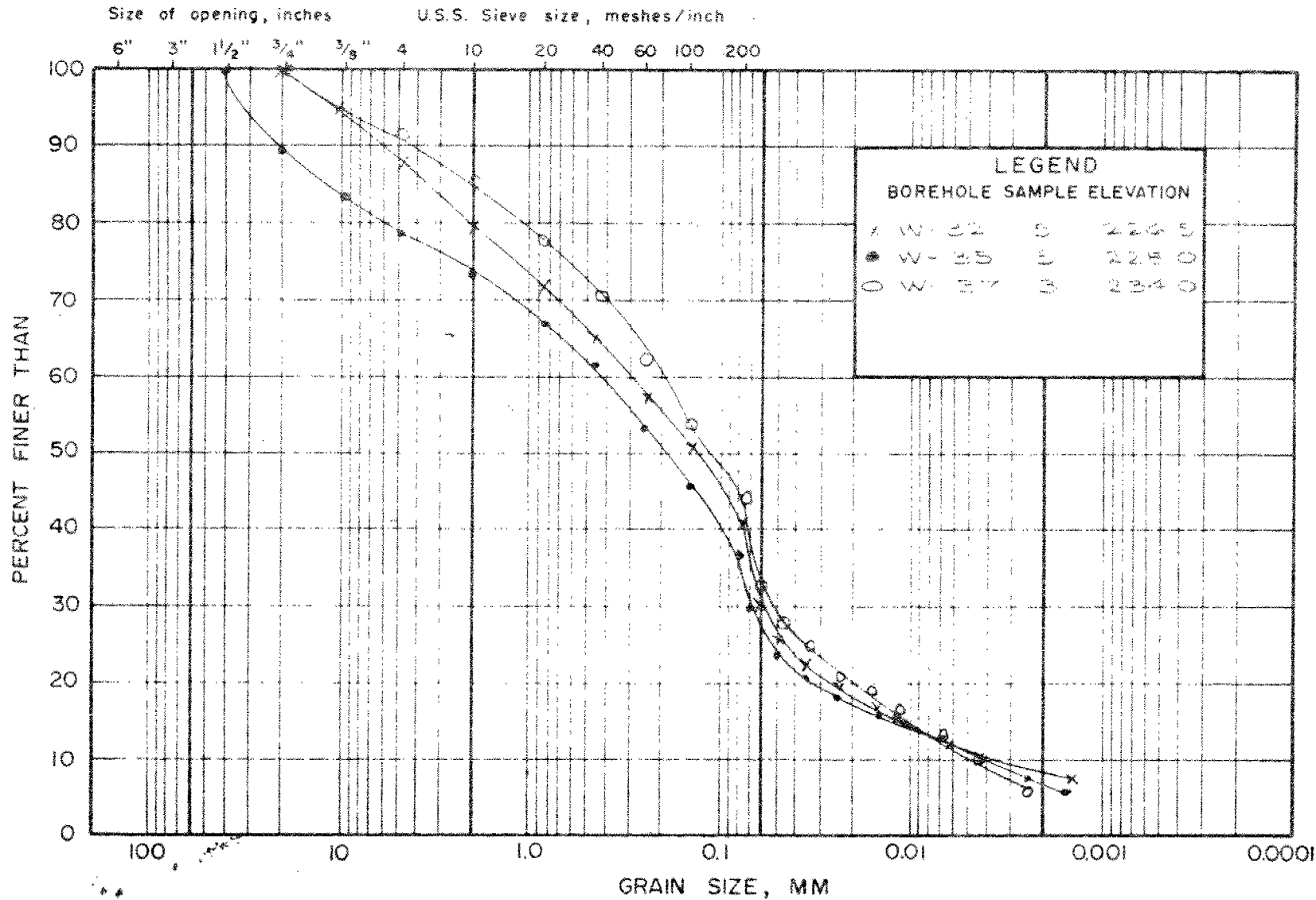
GLACIAL TILL

GRAIN SIZE DISTRIBUTION

FIGURE 6

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

M.I.T. GRAIN SIZE SCALE



GLACIAL TILL

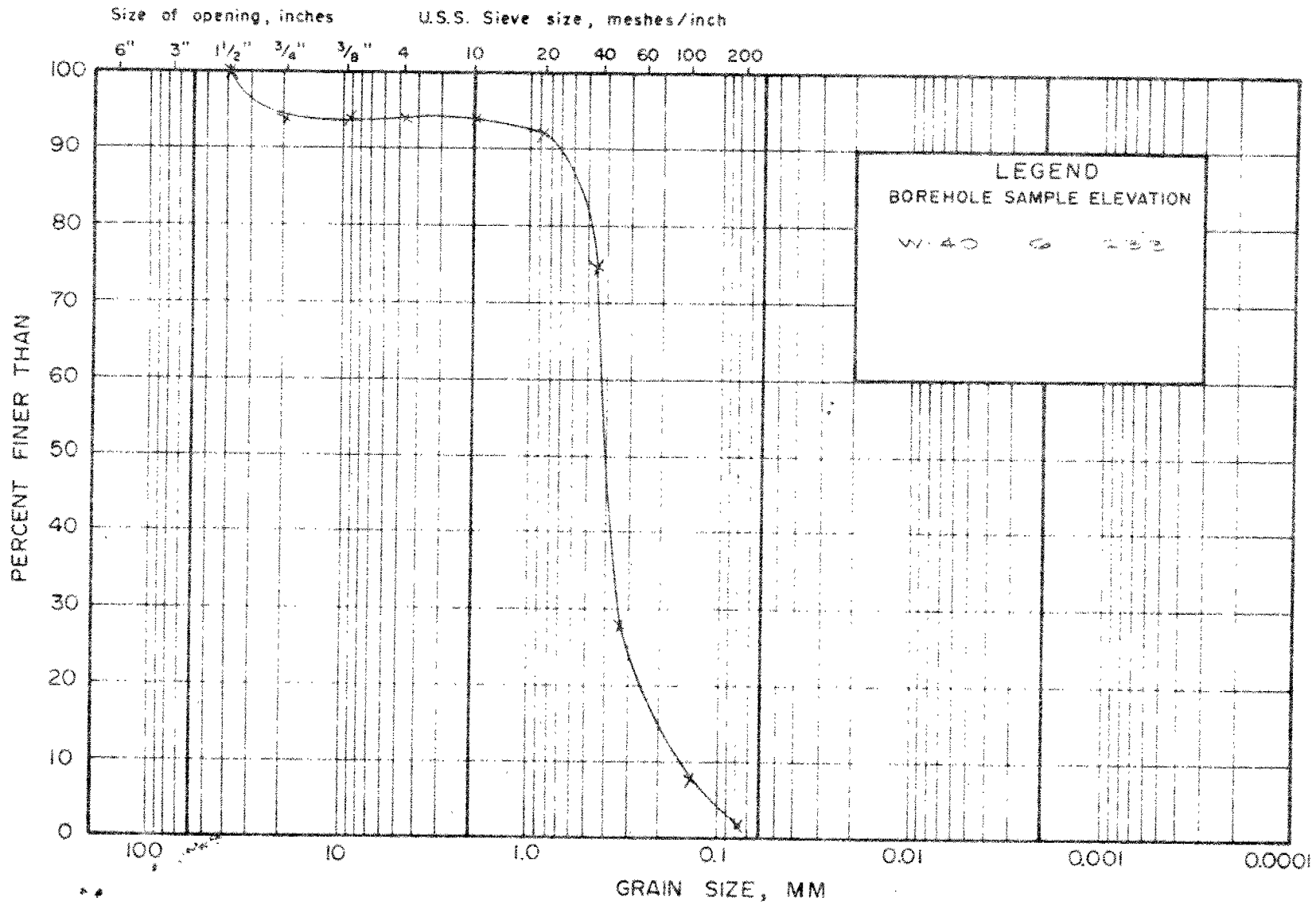
GRAIN SIZE DISTRIBUTION

FIGURE 7

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			



M.I.T. GRAIN SIZE SCALE



FINE TO MEDIUM SAND

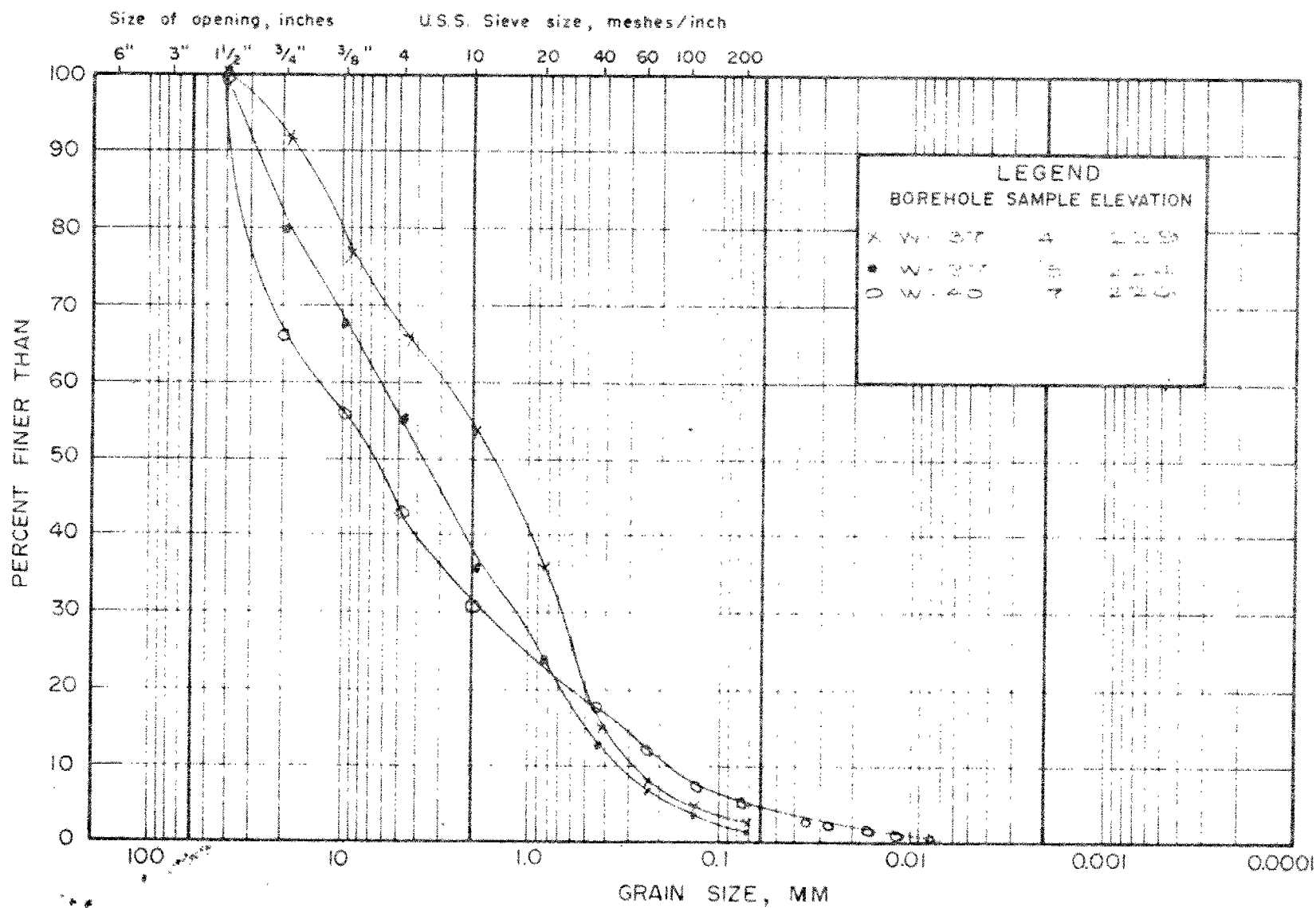
GRAIN SIZE DISTRIBUTION

FIGURE 3

Golder Associates

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

M.I.T. GRAIN SIZE SCALE



SAND AND GRAVEL

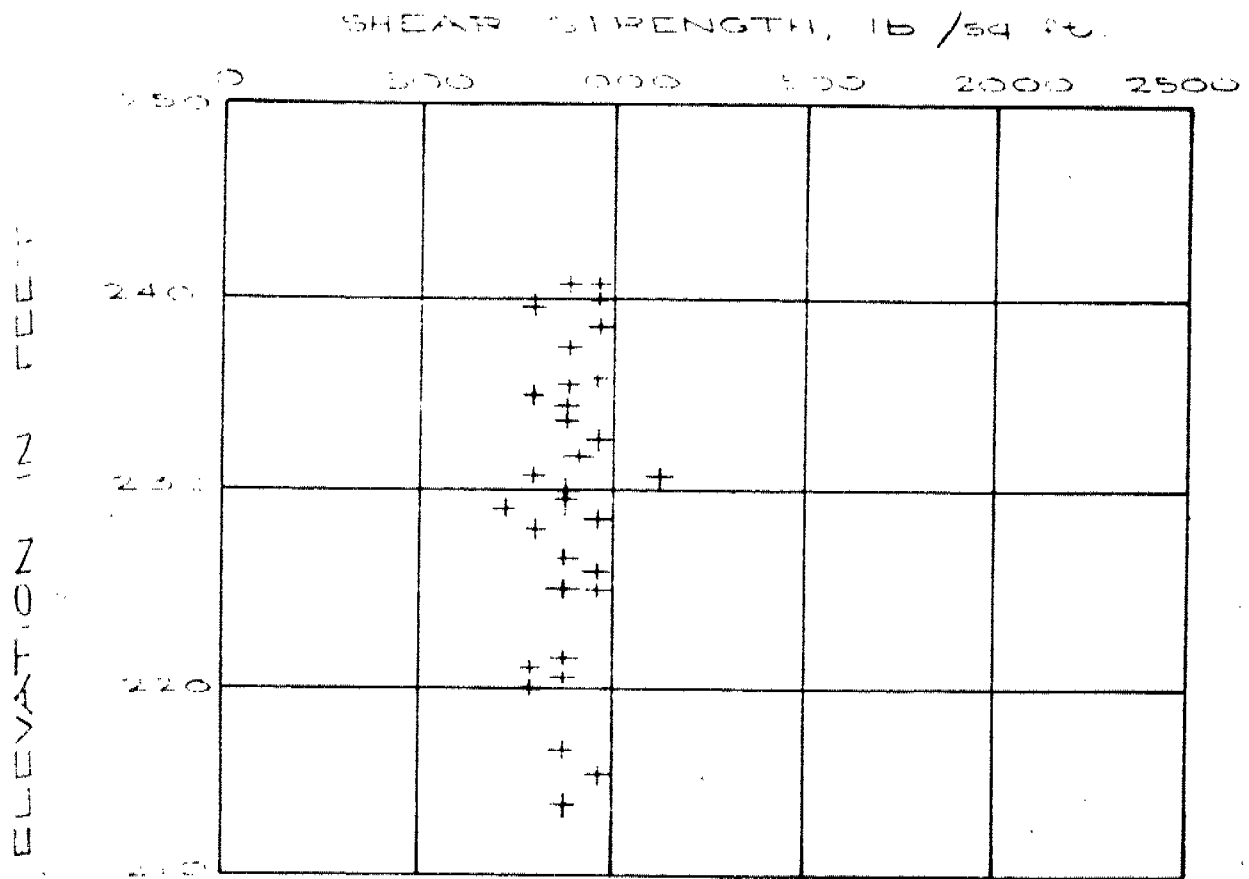
GRAIN SIZE DISTRIBUTION

FIGURE 9

# SHEAR STRENGTH VERSUS ELEVATION (SILTY CLAY)

FIGURE 10

BOREHOLES W-32 TO W-36 & W-39



## LEGEND

+ INSITU FIELD VANE

Date JULY 22, 1975

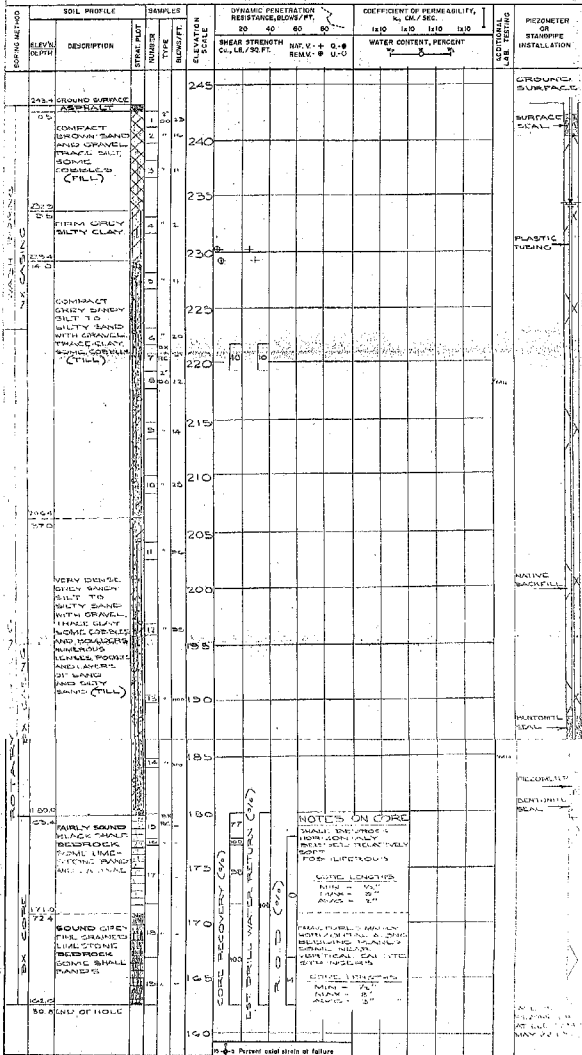
Golder Associates

Drawn GF  
Chkd. RAA  
Appd. RAA



DAFUM GEDRETTET

PENETRATION TEST HAMMER WEIGHT 140 LB. DROP 30 IN.



VERTICAL SCALE  
IN. TO 5 FT.

**Golder Associates**

DRAWN 2/1  
CHECKED AM

# RECORD OF BOREHOLE W-39

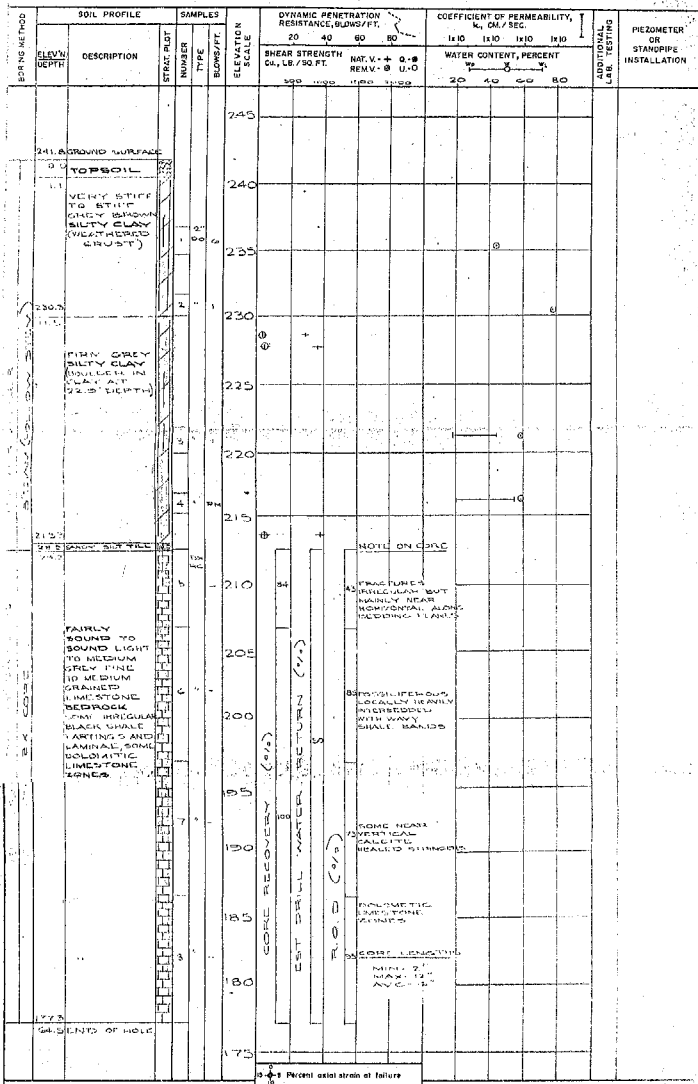
LOCATION See Figure 3

BORING DATE JUNE 11, 1975

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.





SCALE	HORIZ	1" = 100'
	VERT	1" = 10'

1000 ASPHALT  
 1000 CRUSHED STONE  
 1000 TOPSOIL  
 1000 LOTS TO CONVERT BRUSH, SAND AND GRAVEL  
 1000 LOOSE SOIL, COLLECTED, STONES AND VEGG (FILE)  
 1000 LARGE TO VERY BONE BRUSH, TO VERY SILTY  
 1000 SAND WITH GRAVEL  
 1000 100% TO 90% CLAY, TO BROWN SILTY CLAY  
 1000 (CONTAMINATED) CLAY  
 1000 ATTY GRAY SILTY CLAY  
 1000 COMPACT TO VERY DENSE SANDY CLT TO SILTY  
 1000 SAND WITH GRAVEL, TO SILTY CLAY, SOME SANDS  
 1000 WITH FRAGMENTS (FILE)  
 1000 FRACTURED SILTY CLAY AND BEDROCK  
 1000 FAIRLY SOUND BLASTED BEDROCK, BEDROCK TOILE  
 1000 100% SAND, SILTY  
 1000 FAIRLY SOUND TO SOUND SILTY CLAY TO SILTY  
 1000 (FAIRLY) BEDROCK, BEDROCK, SAND, SAND  
 1000

WATER LEVEL IN ELEVATION AT TIME OF  
INVESTIGATION

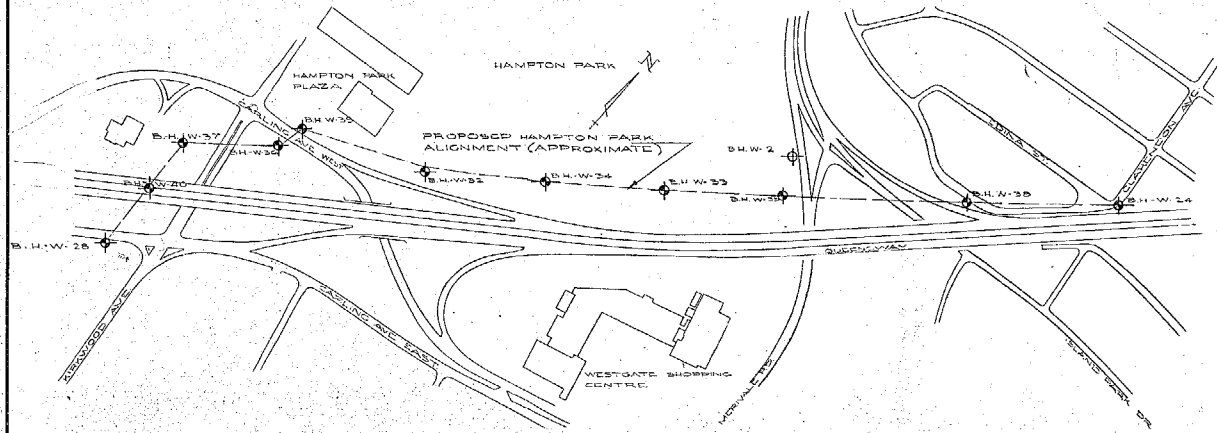
BENCHMARK ELEVATION PREVIOUS PRESENT OR  
A BENCH MARK TOTAL

Small (classified) and medium (unclassified) size groups of fish were sampled in the following way: 100 m<sup>2</sup> and 200 m<sup>2</sup> hauls, respectively, for each of the stations and each station was sampled three times in the same month of the year. The samples are being processed in the studies of the authors (2003).

1970-1971, 1972-1973, 1974-1975, 1976-1977, 1978-1979, 1980-1981, 1982-1983, 1984-1985, 1986-1987, 1988-1989, 1990-1991, 1992-1993, 1994-1995, 1996-1997, 1998-1999, 2000-2001, 2002-2003, 2004-2005, 2006-2007, 2008-2009, 2010-2011, 2012-2013, 2014-2015, 2016-2017, 2018-2019, 2020-2021, 2022-2023, 2024-2025, 2026-2027, 2028-2029, 2030-2031, 2032-2033, 2034-2035, 2036-2037, 2038-2039, 2040-2041, 2042-2043, 2044-2045, 2046-2047, 2048-2049, 2050-2051, 2052-2053, 2054-2055, 2056-2057, 2058-2059, 2060-2061, 2062-2063, 2064-2065, 2066-2067, 2068-2069, 2070-2071, 2072-2073, 2074-2075, 2076-2077, 2078-2079, 2080-2081, 2082-2083, 2084-2085, 2086-2087, 2088-2089, 2090-2091, 2092-2093, 2094-2095, 2096-2097, 2098-2099, 2100-2101, 2102-2103, 2104-2105, 2106-2107, 2108-2109, 2110-2111, 2112-2113, 2114-2115, 2116-2117, 2118-2119, 2120-2121, 2122-2123, 2124-2125, 2126-2127, 2128-2129, 2130-2131, 2132-2133, 2134-2135, 2136-2137, 2138-2139, 2140-2141, 2142-2143, 2144-2145, 2146-2147, 2148-2149, 2150-2151, 2152-2153, 2154-2155, 2156-2157, 2158-2159, 2160-2161, 2162-2163, 2164-2165, 2166-2167, 2168-2169, 2170-2171, 2172-2173, 2174-2175, 2176-2177, 2178-2179, 2180-2181, 2182-2183, 2184-2185, 2186-2187, 2188-2189, 2190-2191, 2192-2193, 2194-2195, 2196-2197, 2198-2199, 2200-2201, 2202-2203, 2204-2205, 2206-2207, 2208-2209, 2210-2211, 2212-2213, 2214-2215, 2216-2217, 2218-2219, 2220-2221, 2222-2223, 2224-2225, 2226-2227, 2228-2229, 2230-2231, 2232-2233, 2234-2235, 2236-2237, 2238-2239, 2240-2241, 2242-2243, 2244-2245, 2246-2247, 2248-2249, 2250-2251, 2252-2253, 2254-2255, 2256-2257, 2258-2259, 2260-2261, 2262-2263, 2264-2265, 2266-2267, 2268-2269, 2270-2271, 2272-2273, 2274-2275, 2276-2277, 2278-2279, 2280-2281, 2282-2283, 2284-2285, 2286-2287, 2288-2289, 2290-2291, 2292-2293, 2294-2295, 2296-2297, 2298-2299, 2300-2301, 2302-2303, 2304-2305, 2306-2307, 2308-2309, 2310-2311, 2312-2313, 2314-2315, 2316-2317, 2318-2319, 2320-2321, 2322-2323, 2324-2325, 2326-2327, 2328-2329, 2330-2331, 2332-2333, 2334-2335, 2336-2337, 2338-2339, 2340-2341, 2342-2343, 2344-2345, 2346-2347, 2348-2349, 2350-2351, 2352-2353, 2354-2355, 2356-2357, 2358-2359, 2360-2361, 2362-2363, 2364-2365, 2366-2367, 2368-2369, 2370-2371, 2372-2373, 2374-2375, 2376-2377, 2378-2379, 2380-2381, 2382-2383, 2384-2385, 2386-2387, 2388-2389, 2390-2391, 2392-2393, 2394-2395, 2396-2397, 2398-2399, 2400-2401, 2402-2403, 2404-2405, 2406-2407, 2408-2409, 2410-2411, 2412-2413, 2414-2415, 2416-2417, 2418-2419, 2420-2421, 2422-2423, 2424-2425, 2426-2427, 2428-2429, 2430-2431, 2432-2433, 2434-2435, 2436-2437, 2438-2439, 2440-2441, 2442-2443, 2444-2445, 2446-2447, 2448-2449, 2450-2451, 2452-2453, 2454-2455, 2456-2457, 2458-2459, 2460-2461, 2462-2463, 2464-2465, 2466-2467, 2468-2469, 2470-2471, 2472-2473, 2474-2475, 2476-2477, 2478-2479, 2480-2481, 2482-2483, 2484-2485, 2486-2487, 2488-2489, 2490-2491, 2492-2493, 2494-2495, 2496-2497, 2498-2499, 2500-2501, 2502-2503, 2504-2505, 2506-2507, 2508-2509, 2510-2511, 2512-2513, 2514-2515, 2516-2517, 2518-2519, 2520-2521, 2522-2523, 2524-2525, 2526-2527, 2528-2529, 2530-2531, 2532-2533, 2534-2535, 2536-2537, 2538-2539, 2540-2541, 2542-2543, 2544-2545, 2546-2547, 2548-2549, 2550-2551, 2552-2553, 2554-2555, 2556-2557, 2558-2559, 2560-2561, 2562-2563, 2564-2565, 2566-2567, 2568-2569, 2570-2571, 2572-2573, 2574-2575, 2576-2577, 2578-2579, 2580-2581, 2582-2583, 2584-2585, 2586-2587, 2588-2589, 2590-2591, 2592-2593, 2594-2595, 2596-2597, 2598-2599, 2600-2601, 2602-2603, 2604-2605, 2606-2607, 2608-2609, 2610-2611, 2612-2613, 2614-2615, 2616-2617, 2618-2619, 2620-2621, 2622-2623, 2624-2625, 2626-2627, 2628-2629, 2630-2631, 2632-2633, 2634-2635, 2636-2637, 2638-2639, 2640-2641, 2642-2643, 2644-2645, 2646-2647, 2648-2649, 2650-2651, 2652-2653, 2654-2655, 2656-2657, 2658-2659, 2660-2661, 2662-2663, 2664-2665, 2666-2667, 2668-2669, 2670-2671, 2672-2673, 2674-2675, 2676-2677, 2678-2679, 2680-2681, 2682-2683, 2684-2685, 2686-2687, 2688-2689, 2690-2691, 2692-2693, 2694-2695, 2696-2697, 2698-2699, 2700-2701, 2702-2703, 2704-2705, 2706-2707, 2708-2709, 2710-2711, 2712-2713, 27

# **BORING PLAN** (HAMPTON PARK ALIGNMENT) (BORING LOCATIONS W-24, W-28, W-32 TO W-40)

FIGURE 3



## **LEGEND**

- BORING IN PLAN PRESENT INVESTIGATION
- BORING IN PLAN PREVIOUS INVESTIGATION REPORT NO. 70734

REFERENCE  
 CITY OF OTTAWA CENTRAL  
 REGISTRY UNDERGROUND  
 SERVICES PLANS SUPPLIED BY  
 CITY OF OTTAWA ENGINEERING  
 DEPARTMENT

## **SPECIAL NOTE**

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

## **NOTE**

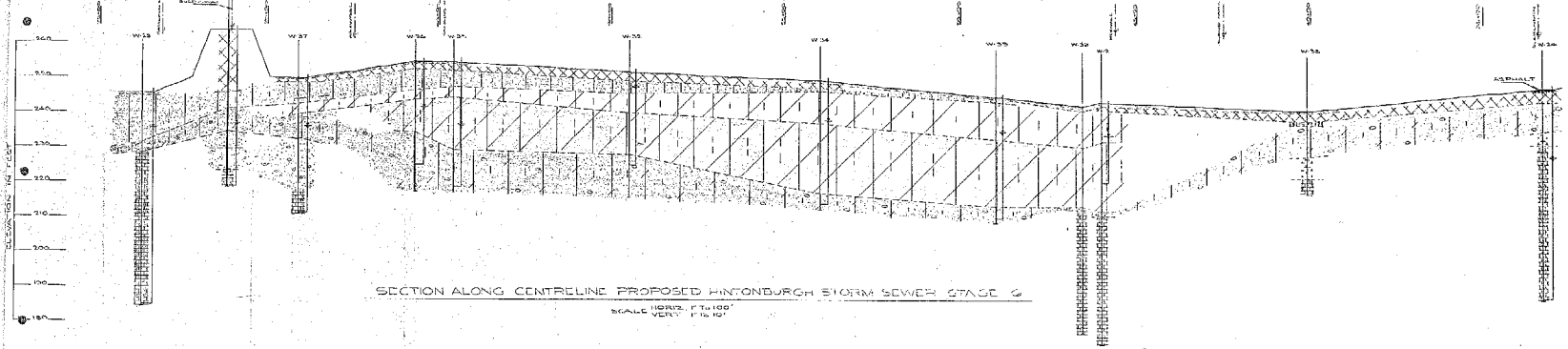
BORING LOCATIONS SHOWN ON  
 THIS DRAWING ARE APPROXIMATE  
 ONLY FOR EXACT LOCATIONS, SEE  
 SURVEY NOTES BY CITY OF  
 OTTAWA SURVEY DEPARTMENT

PLAN  
 SCALE 1" TO 200'

DATE: JULY 15, 1973

Golder Associates

Drawn by  
 C.A.D. *AM*  
 App. *AM*



- STRATIGRAPHY**
- TOPSOIL
  - DARK BROWN SILTY SAND TO SILTY SAND WITH BOULDER, TRACE CLAY AND GRAVEL (FILL)
  - VERY LOOSE TO COMPACT BROWN SILTY SAND
  - VERY SILTY CLAY BROWN SILTY CLAY (WEATHERED CRUST)
  - FIRM GREY SILTY CLAY OCCASIONAL THIN SAND LENSES
  - COMPACT TO VERY DENSE GREY SILTY SAND TO SANDY SILT WITH GRAVEL, SHELLS AND BOULDER, TRACE CLAY (FILL)
  - COMPACT TO DENSE SAND AND GRAVEL
  - VERY DENSE GREY FINE TO COARSE SAND AND GRAVEL SOME BOULDER
  - FRACTURED GREY LIMESTONE BEDROCK
  - FAIRLY SOUND TO SOUND GREY LIMESTONE BEDROCK
- Notes: 1. This diagram is a schematic representation of the soil profile. It is not a photograph. 2. The soil profile is shown in cross-section. 3. The soil profile is shown in cross-section. 4. The soil profile is shown in cross-section. 5. The soil profile is shown in cross-section.