

## **OAKVILLE - HAMILTON SECTION**

**ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION**

**HWY 60 ALRT                      DIST 4  
FAIRVIEW STREET EXTENSION  
BRIDGES**

**FOUNDATION INVESTIGATION  
AND DESIGN REPORT**

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### Record of Boreholes

## APPENDIX II

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            Silty Clay, some sand
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            Silty Clay, some sand

Drawing at Rear of Report  
Fairview Street Extension Bridges  
Borehole Locations and Soil Strata  
Station 19+909, 510

## 1.0 INTRODUCTION

Geocon Inc. has been retained by the GO ALRT programme to carry out a geotechnical investigation at the site of the proposed crossing of the future extension of Fairview Street. The above investigation has been carried out under the technical direction of Mr. K. G. Selby, (Senior) <sup>CHIEF</sup> Foundation Engineer, Ministry of Transportation and Communications. The work was carried out in accordance with our proposal, dated December 3rd, 1984.

The purpose of the investigation was to obtain subsurface information for use in design and construction of foundations for abutments and retaining walls required to support the proposed Fairview Street extension bridges.

Our mandate was to study the proposed crossing of the future street extension by the GO ALRT line plus a possible CN spur to the north of the GO ALRT line. The crossing of the future street extension by the existing CNR main line directly south of the GO ALRT line was excluded from our mandate.

## 2.0 PROCEDURE AND EQUIPMENT

The field work for this investigation was carried out between December 13th and 19th, 1984. A Bombardier mounted C.M.E. power auger drill, equipped with (standard) augers and BQ rock coring equipment, was used to put down a total of 6 boreholes. The boreholes were identified as Boreholes 1 to 6 inclusive, and ranged from 10.5 to 12.2 metres in depth.

Samples were recovered within the overburden, in conjunction with the Standard Penetration Test, at intervals of about 0.8 metres. Uncased dynamic cone penetration tests were driven to refusal (greater than 100 blows per 0.3 metres) adjacent to each borehole location.

The underlying bedrock was cored in BQ nominal size for depths ranging from 4.6 to 7.6 metres. The recovered core was examined to determine percent recovery, Rock Quality Designation (R.Q.D.) and bedrock condition.

Casagrande type piezometers were installed within the bedrock stratum near the base of all boreholes. Water levels were observed throughout the period of the field programme with a final set of water level readings taken on January 10th, 1985.

The recovered samples were transported to our Toronto Soil Mechanics Laboratory for detailed examination and testing.

### 3.0 SITE AND GEOLOGY

The proposed Fairview Street extension bridge structures are located within the City of Burlington about 1.0 km west of Burloak Drive. The site is located to the south of The Queen Elizabeth Way along the proposed GO ALRT alignment which runs parallel to, and immediately north of, the existing CNR tracks.

The proposed extension of Fairview Street will cross the GO ALRT alignment at about station 19+910 metres. The boreholes of this investigation were put down within an area extending from about 1 to 12 metres north of the centreline of the proposed GO ALRT line in the immediate vicinity of the future street extension.

The site is situated about 4 km north of Lake Ontario and is located to the south of the Iroquois Shoreline. The general area is described by Chapman and Putnam\* to consist of shale plains overlain by a thin layer of glacial till consisting to a large extent of material of local origin.

The ground surface at the site is generally flat. A shallow drainage ditch is present at the approximate location of the proposed centreline of the GO ALRT alignment. Two mounds of earth are present to the north of the proposed alignment, their location is shown on the attached drawing. As well, several small piles of debris and refuse have been dumped over the site.

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\* Chapman, L.J., and Putnam, D.F., "The Physiography of Southern Ontario", Second Edition, 1966.

#### 4.0 SUBSURFACE CONDITIONS

Reference should be made to the Records of Boreholes included in Appendix I of this Report as well as to the text of this section. Reference should also be made to the Drawing located at the rear of this Report upon which are shown the borehole locations and the subsurface conditions.

The site is generally covered with 1.3 to 1.8 metres of overburden overlying weathered shale and sound shale of the Queenston formation. At Borehole 2 about 1.1 metres of silty clay fill was encountered as the surficial stratum, overlying 1.4 metres of natural overburden.

The subsurface conditions are described in detail in the following sections.

##### 4.1 Fill - Silty Clay, some sand and organics

A surficial stratum of silty clay, some sand and organics fill was encountered to a depth of 1.1 metres in Borehole 2. The cohesive stratum, as inferred from measured Standard Penetration Test "N" values of 10 and 11, is described as being of a generally stiff consistency and is generally reddish brown in colour.

The results of a grain size distribution analysis of a single representative sample of the stratum are given in Figure 2 of Appendix II. The tests yielded results of 4 percent gravel, 24 percent sand, 58 percent silt and 14 percent clay. The result of a natural water content determination was 5.4 percent.

#### 4.2 Silty Clay, some sand and organics

A surficial stratum of silty clay, some sand and organics was encountered in Boreholes 1 and 3 to 6 and ranged in thickness from 1.3 to 1.8 metres. The stratum was also encountered underlying the fill stratum in Borehole 2. The stratum was typically reddish brown in colour.

Standard Penetration Tests carried out within the stratum yielded "N" values which ranged from 4 to in excess of 30, which indicate the soil is generally of a firm to hard consistency.

Grain size distribution analyses, carried out on three samples of the stratum, yielded the following results.

<u>Borehole</u>	<u>Sample</u>	<u>% Gravel</u>	<u>% Sand</u>	<u>% Silt</u>	<u>% Clay</u>
1	2	2	7	70	21
4	2	1	5	73	21
6	2	2	10	73	15

The results of these tests are plotted on Figure 3 Appendix II.

Grain size distribution varies throughout the stratum however in general the soil is described as a silty clay, some sand.

An Atterberg Limit Test carried out on the minus 425  $\mu$ m portion of a representative sample of the stratum yielded the following results:

<u>Borehole</u>	<u>Sample</u>	<u>Liquid Limit</u>	<u>Plastic Limit</u>	<u>Plasticity Index</u>	<u>Natural Moisture Content</u>
6	2	26.4%	19.0%	6.4%	8.7%

The results of these tests are plotted on the Plasticity Chart on Figure 1 in Appendix II. The soil is described as silty clay to clay of low plasticity. 2

#### 4.3 Weathered Shale

Weathered shale was encountered underlying the silty clay, some sand in all boreholes. The stratum ranged in thickness from 1.1 to 2.0 metres. The surface of the weathered shale was present between elevations 103.2 and 103.6 metres. Standard Penetration Tests carried out within the weathered shale yielded "N" values in excess of 100. 2

The weathered shale is generally reddish brown in colour. Greenish grey limestone beds are also occasionally present throughout the stratum.

#### 4.4 Sound Shale Bedrock

Sound shale was encountered, underlying the weathered shale, between elevations 101.4 and 102.3 metres. The boundary between sound and weathered bedrock, which is gradual, was inferred from observations made during advance of the augers. The sound shale was reddish brown in colour with occasional greenish grey shale and limestone beds. Occasional silty clay layers were also present in the stratum. The bedrock was generally sound and intact.

Bedrock recovery ranged from 74 to 100 percent. R.Q.D. values determined on BQ core for the sound shale generally ranged from 28 to 91 percent indicating poor to excellent bedrock conditions.

#### 4.5 Groundwater Conditions

Groundwater levels determined on January 10th, 1985, some 3 weeks after the end of the field work, ranged from elevation 98.8 to 100.5 metres. Groundwater levels are expected to vary somewhat seasonally.



## 5.0 GEOTECHNICAL DESIGN AND CONSTRUCTION RECOMMENDATIONS

It is understood that the GO ALRT crossing over the proposed Fairview Street extension in Burlington, Ontario will be constructed on a single or double span closed abutment rigid bridge structure. As well, a possible CN spur line located north of the GO ALRT line will be constructed over the proposed street extension on a double span closed abutment rigid bridge structure. Both bridge structures will be located immediately to the north of the existing CNR main line. The Fairview Street extension may only be constructed a number of years after the construction of the GO ALRT line. In this event, it is possible that the bridge structure for the CNR main line would be constructed at the time Fairview Street is extended. Therefore, the structure for the CNR main line may or may not be constructed when the GO ALRT and possible CN spur line bridge structures are built. The comments and recommendations given herein apply only to the GO ALRT and CN spur line bridge structures.

### 5.1 Proposed Bridge Structures

Spread footings may be used to support the proposed bridge structures.

Based on GO ALRT Drawing P-016, Revision 0, the elevation of the top of the asphalt of Fairview Street beneath the proposed GO ALRT LINE will be about 99.4. Based on the same drawing the elevation of the underside of the bridge structure foundations would be about 97.7. At this founding level sound shale bedrock will be the bearing stratum.

The spread foundations of the proposed structures should be designed using a bearing capacity at the U.L.S. of 1500 kPa on sound shale bedrock. The bearing capacity at S.L.S. Type II will not govern the design.

## 5.2 Proposed Bridge Retaining Walls

The proposed retaining walls which are integral to the bridge structures may also be supported on spread foundations. In view of the relatively shallow depth of overburden, such foundations would be carried on shale bedrock. Therefore surficial fill and silty clay, some sand and organics would be excavated at the locations of the proposed footings. Insofar as the founding level for the retaining walls will probably step up with increasing distance from the abutments, the spread foundations will be founded within both weathered and sound shale bedrock.

The spread foundations should be designed using a bearing capacity at the U.L.S. of 1000 kPa and 1500 kPa for weathered and sound shale bedrock, respectively. As for the bridge abutments the bearing capacity at S.L.S. Type II will not govern the design.

For inclined resultant loads the ultimate bearing capacity should be reduced as specified in the Ontario Highway Bridge Design Code (O.H.B.D.C.) 1983, Section 6.7.3.3.5.

## 5.3 Proposed CNR Retaining Wall

If the GO ALRT and CN spur line bridge structures are built prior to the CNR main line bridge structure, it will be necessary to maintain the integrity of the CNR main line during the GO ALRT construction campaign. Insofar as the distance between the existing north rail of the north track of the CNR main line and the south face of the proposed GO ALRT bridge structure is only slightly over 4 m, it is considered that a vertical retaining wall located between the CNR main line and the south face of the GO ALRT bridge structure will be the most practical shoring scheme to prevent loss of ground from beneath the CNR main line.

As it is not known how much time might elapse between the construction of the GO ALRT line and the CNR main line bridge structures, this retaining wall should be designed as a long term structure. Thus, the shale bed-rock formation, which weathers readily after exposure and through which the excavation for the GO ALRT bridge structure must be made, should not be allowed to stand unsupported without protection, i.e. the retaining wall should extend over the full height of the excavation. The wall should be installed prior to the mass excavation for the GO ALRT bridge structure foundations, assuming that the CNR main line remains in service during construction of the GO ALRT bridges.

Due to the limited space restrictions and the considerable amount of rock to be protected, it is considered that a vertical wall tied with rock anchors is the most feasible shoring scheme. One approach would be the slurry trench technique where the wall would be excavated and installed in sections, so as not to leave the entire unconcreted trench subject to loadings generated from train traffic. Churn drills would likely be necessary to excavate the rock formation. Alternatively, the retaining wall could be constructed out of concrete caissons drilled adjacent to one another. In order to minimize the risk of gaps between adjacent caissons, it is recommended that each caisson be drilled so as to overlap its neighbouring caisson. Thirdly, the wall may be designed as a soldier pile and lagging structure. The soldier piles, which could comprise steel H piles, would likely have to be pre-augered to achieve the necessary penetration. The wall should extend below the founding level and this depth would best be established during detailed design of the shoring system.

Rock anchors that will serve as part of the tie back system should be located at an angle of approximately 45 degrees and be designed with an ultimate grout to rock adhesion of not greater than 600 kPa.

Within the overburden and upper 1 m of weathered shale bedrock lateral earth/rock pressures should be computed assuming a unit weight of 24

kN/m<sup>3</sup> and an angle of internal friction of 30 degrees. Below the upper 1 m of the weathered bedrock formation the unit weight used in lateral pressure computations should be increased to 26 kN/m<sup>3</sup>. The wall should be designed incorporating the full static and dynamic loads from rail traffic on the CNR main line tracks.

If the retaining wall is installed with slurry wall or concrete caisson techniques, it should be assumed that no drainage behind the wall will occur. The current investigation indicated that the groundwater level was present between elevations 98.8 and 100.5. Thus it is recommended that the retaining wall be designed to accommodate hydrostatic pressures arising from a groundwater level at elevation 102.0 to allow for seasonal variation. A soldier pile and lagging retaining wall need not be designed for hydrostatic pressures.

If the proposed GO ALRT bridge structure is poured flush with the retaining wall, it is important that a construction joint extending the full height of the retaining wall be provided. In other words, the two structures must be structurally independent of one another, so as to facilitate the eventual removal of the retaining wall when the CNR main line bridge structure is constructed.

#### 5.4 General Design and Construction Recommendations

1. A minimum of 1.2 metres of earth cover should be provided above the base of all foundations for frost protection purposes.
2. A concrete working slab, of 150 mm minimum thickness, should be placed on the surface of the weathered shale and/or sound shale within 12 hours of excavation to the founding level to prevent disturbance of these bearing levels.

3. The silty clay, some sand and organics fill identified on the north side of the proposed GO ALRT line should be excavated prior to placement of approach fills. In addition all topsoil should be removed within the limits of the approach fills. Thus, the approach fills will be constructed over the firm to hard clayey <sup>2</sup> silt formation or granular backfill placed behind the bridge structures.
4. Backfill to the structures should be composed of free draining engineered granular fill (M.T.C. Granular B) in accordance with M.T.C. Standard Special Provision 121 October, 1983. Suitable positive drainage should be provided to prevent the build up of excess hydrostatic pressure behind the retaining walls of the bridge structures.
5. Computation of earth pressures should be carried out in accordance with Section 6.6.1.2 of the O.H.B.D.C. If M.T.C. Granular "A" backfill is to be used the following properties may be assumed for design purposes:  $\gamma = 22 \text{ kN/m}^3$ ,  $\phi = 35^\circ$ . For M.T.C. Granular "B" a wide range of values for  $\gamma$  and  $\phi$  exist. Unless the source of the material is known and soil tests are carried out prediction of the above values may be subject to error. In this case it will be necessary to compute earth pressures in accordance with Section 6.6.1.2.2. of the O.H.B.D.C. It should be noted that for earth pressure coefficients the at rest ( $K_0$ ) condition applies since the foundations are considered to be non yielding.
6. Lateral forces on the bridge structure retaining walls and abutments may be resisted by keying the footing into bedrock a minimum of 0.5 metres and designing the footing using a coefficient of friction of 0.47 ( $\tan 25^\circ$ ) between the footing and sound bedrock

and a coefficient of friction of 0.40 ( $\tan 22^\circ$ ) between the footing and weathered bedrock. Alternatively, lateral forces may be resisted by the installation of dowels a minimum of 1.5 metres into bedrock. Passive resistance in front of footings and retaining walls should be ignored except for that portion below the anticipated frost penetration of 1.2 m.

7. Temporary excavations in the overburden may be carried out in routine open cut with side slopes of 45 degrees. Temporary excavations in the bedrock formation may be carried out at slopes no steeper than 3 vertical : 1 horizontal. Care should be taken to scale off all loose pieces of rock prior to workmen entering the excavation.

Permanent slopes in both the overburden and bedrock should be no steeper than 2 horizontal : 1 vertical.

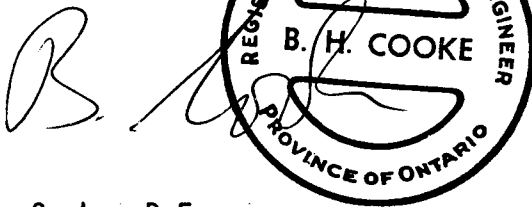
8. Construction of the bridge will involve excavation to a depth of about 7 m below ground surface and the excavation will therefore extend about 4 m below the surface of the sound shale. Care should be taken to avoid disturbance of the intact bedrock below founding level during rock excavation.
9. During the current investigation the groundwater level was observed to range between elevations 98.8 and 100.5. These levels are expected to fluctuate somewhat seasonally. In any event, some infiltration should be anticipated in all excavations below about elevation 102.0. This infiltration is not expected to be major and should be satisfactorily handled by filter equipped pumps located in sumps placed in the corners of the excavations.

## 6.0 CLOSURE

Field work for this investigation was carried out under the supervision of our Mr. P. Ho, P.Eng. This letter has been written by Mr. B. Cooke, P.Eng., with technical input from Mr. H.L. MacPhie, P.Eng.

We trust this report contains sufficient detail for your purposes. Please contact us should you require elaboration on any matter.

Yours very truly,  
GEOCON INC.,

A handwritten signature of Barry Cooke is written over a circular professional engineer stamp. The stamp contains the text "REGISTERED PROFESSIONAL ENGINEER", "B. H. COOKE", and "PROVINCE OF ONTARIO".

Barry Cooke, P.Eng.,  
Senior Project Engineer.

A handwritten signature of H.L. MacPhie.

H.L. MacPhie, P.Eng.,  
Vice President.

BC/pw

T10866/42704

APPENDIX I

Record of Boreholes



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

## METRIC

Fairview Street Extension  
Bridges

LOCATION 4 804 602.3N ; 283 905.4 E

ORIGINATED BY P.H.

DIST 4 HWY GO ALRT

BOREHOLE TYPE Standard Auger, Cone Test, BQ Core

COMPILED BY A.E.L.

DATUM Geodetic

DATE 1984 12 17-18

CHECKED BY BE

[illegible]

**+3, x5 : Numbers refer to Sensitivity**

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

## METRIC

Fairview St. Extension

W P Bridges LOCATION 4 804 616.5 N ; 283 916.5 E

ORIGINATED BY PH

DIST 4 HWY GO ALRT BOREHOLE TYPE Standard Auger, Cone Test, BQ Core

COMPILED BY AEL

DATUM Geodetic DATE 1984 12 13-14

CHECKED BY DB

[illegible]

# OFFICE REPORT ON' SOIL EXPLORATION

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

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



Ministry of  
Transportation and  
Communications  
Ontario

# RECORD OF BOREHOLE No 3

METRIC

Fairview St. Extension

W P Bridges LOCATION 4 804 629.3 ; 283 927.3

ORIGINATED BY PH

DIST 4 HWY GO ALRT BOREHOLE TYPE Standard Auger, Cone Test, BQ Core

COMPILED BY AEL

DATUM Geodetic DATE 1984 12 19

CHECKED BY Ble

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					
105.0	Ground Level																
0.0	Silty Clay some sand and organics Firm to hard		1	SS	6												
			2	SS	32		104										
103.2	Reddish Brown		3	SS	170/0		103										
1.8	Shale Bedrock Weathered		4	SS	50/0												
102.0	Reddish Brown		5	SS	100/0		102										
3.0							101										
							100										
			6	RC BQ	REC 92%		99										
							98										
	Shale Bedrock Sound		7	RC BQ	REC 100%		97										
							96										
	Reddish Brown		8	RC BQ	REC 93%		95										
							94										
			9	RC BQ	REC 93%		93										
			10	RC BQ	REC 100%												
92.8																	
12.2	END OF BOREHOLE																

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

OFFICE REPORT ON SOIL EXPLORATION

# RECORD OF BOREHOLE No 4

METRIC

Fairview St. Extension

W P Bridges

LOCATION 4 804 594.0 N ; 283 912.5 E

ORIGINATED BY PH

DIST 4 HWY GO ALRT

BOREHOLE TYPE Standard Auger, Cone Test, BQ Core

COMPILED BY AEL

DATUM \_\_ Geodetic

DATE 1984 12 18-19

CHECKED BY ABE

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

·  $+^3, \times^5$  : Numbers refer to Sensitivity

20  
15 - 5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE No 5

METRIC

Fairview St. Extension  
W P Bridges LOCATION 4 804 607.5 N ; 283 921.7 E ORIGINATED BY PH  
DIST 4 HWY GO ALRT BOREHOLE TYPE Standard Auger, Cone Test, BQ Core COMPILED BY AEL  
DATUM Geodetic DATE 1984 12 14-17 CHECKED BY Be

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					
105.0	Ground Level																
0.0	Silty Clay some sand and organics Firm to hard Reddish Brown		1	SS	7												
			2	SS	68		104										
103.5			3	SS	125												
1.5	Shale Bedrock Weathered Reddish Brown		4	SS	100/ 0.05 m												
102.0			5	SS	100/ 0.05 m												
3.0			6	SS	100/ 0.02 m												
	Shale Bedrock Sound Reddish Brown		7	SS	100/ 0.08 m												
			8	RC BQ	REC 82%												RQD 50%
			9	RC BQ	REC 100%												87%
			10	RC BQ	REC 100%												89%
94.2																	
10.8	END OF BOREHOLE																

+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



# RECORD OF BOREHOLE No 6

METRIC

Fairview St. Extension  
W P Bridges LOCATION 4 804 622.0 ; 283 932.8 E ORIGINATED BY PH  
DIST 4 HWY GO ALRT BOREHOLE TYPE Standard Auger, Cone Test, BQ Core COMPILED BY AEL  
DATUM Geodetic DATE 1984 12 18-19 CHECKED BY Be

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
104.7	Ground Level													
0.0	Silty Clay some sand and organics Firm to hard Reddish Brown		1	SS	3		104							2 10 73 15
103.4			2	SS	95		103							
1.3	Shale Bedrock Weathered Reddish Brown		3	SS	135/ 0.13 m		102							
102.3			4	SS	75/6		101							
2.4			5	SS	100/0		100							
			6	RC BQ	REC 100%		99							RQD 56%
	Shale Bedrock Sound Reddish Brown		7	RC BQ	REC 99%		98							38%
			8	RC BQ	REC 100%		97							73%
			9	RC BQ	REC 100%		95							91%
94.2														
10.5	END OF BOREHOLE													

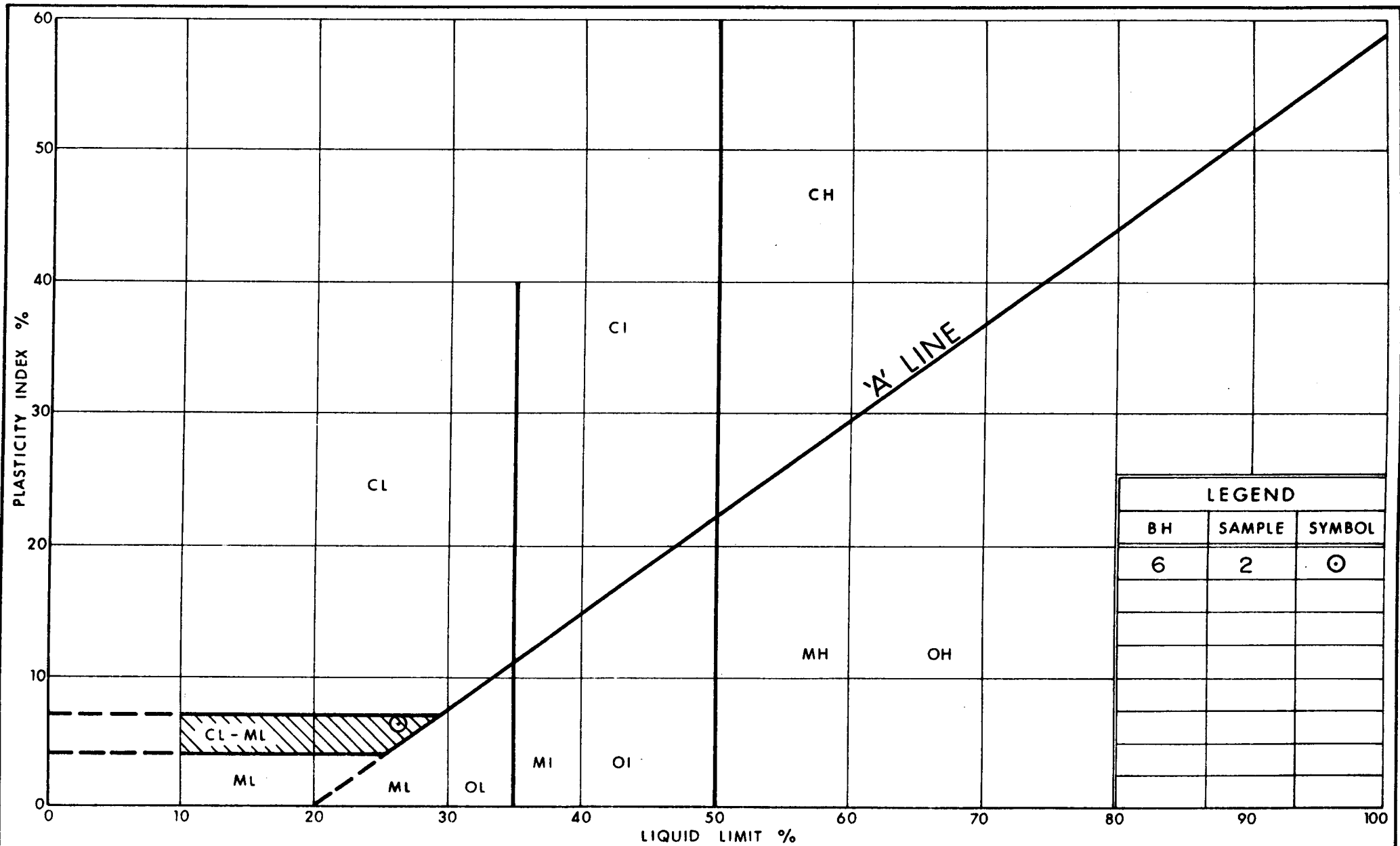
+3, x5: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

## APPENDIX II

- Figure 1    Plasticity Chart  
             Silty Clay, some sand and gravel
- Figure 2    Grain Size Distribution  
             Fill - Silty Clay, some sand
- Figure 3    Grain Size Distribution  
             Silty Clay, some sand





Ministry of  
Transportation and  
Communications

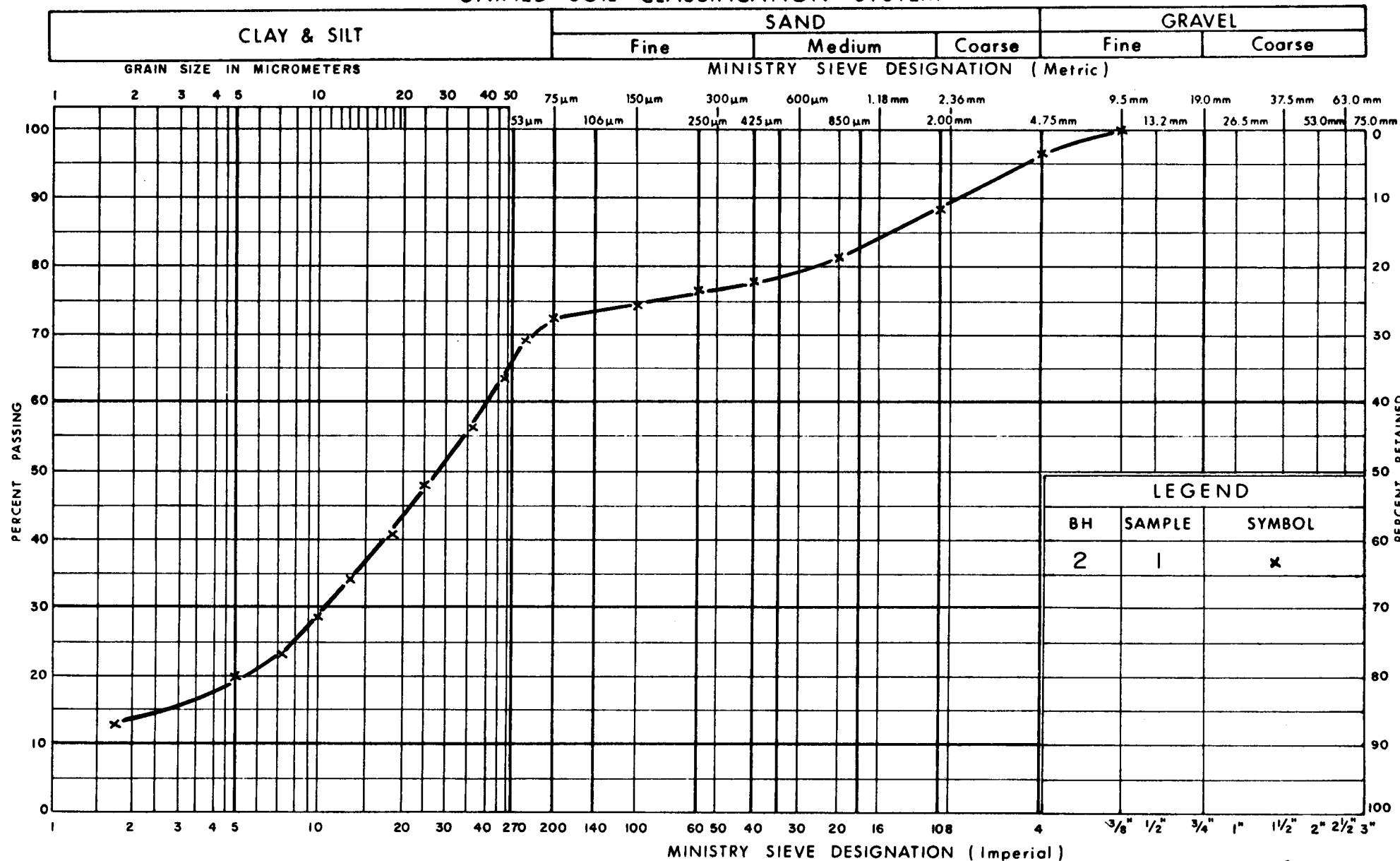
Ontario

# PLASTICITY CHART SILTY CLAY SOME SAND

FIG No 1

W P 2

## UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

 Ministry of  
Transportation and  
Communications

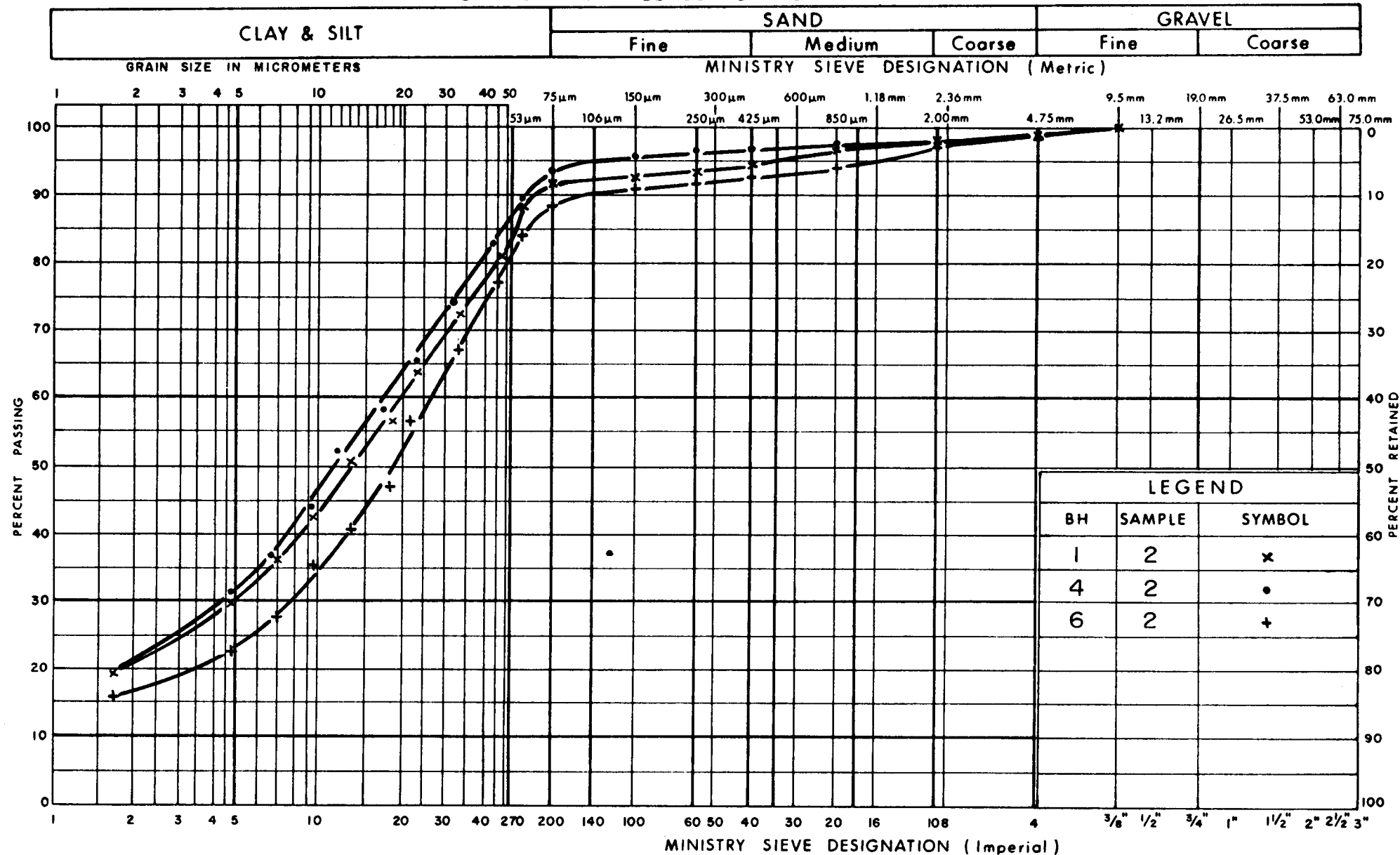
## GRAIN SIZE DISTRIBUTION

FILL SILTY CLAY SOME SAND

FIG No 2

W P 2

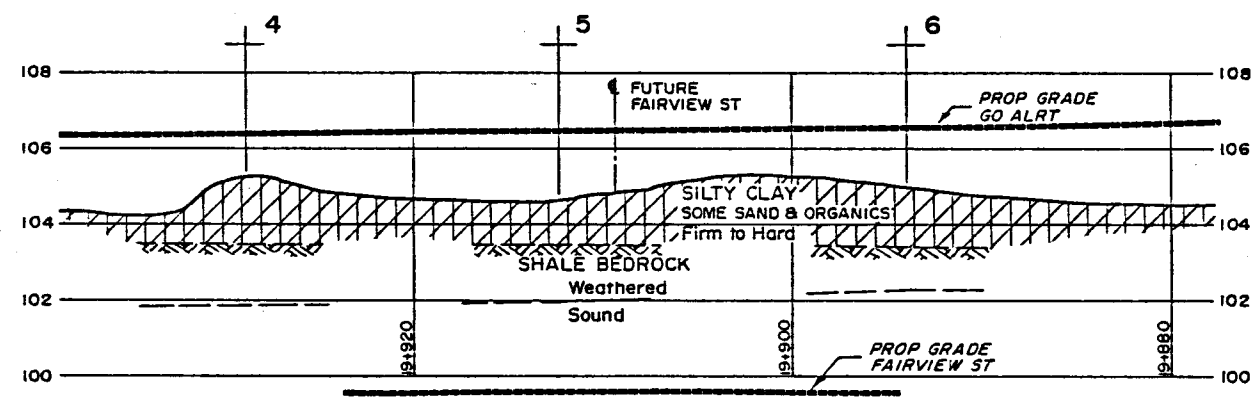
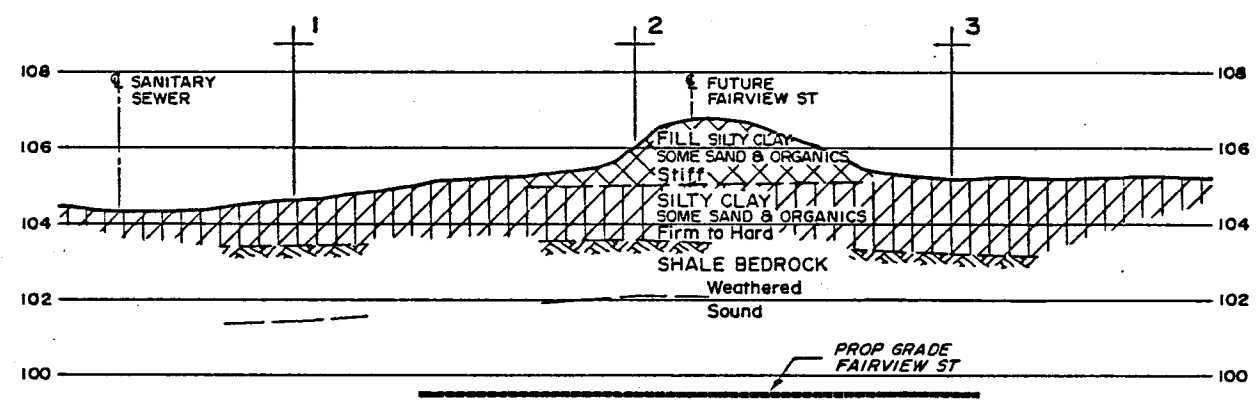
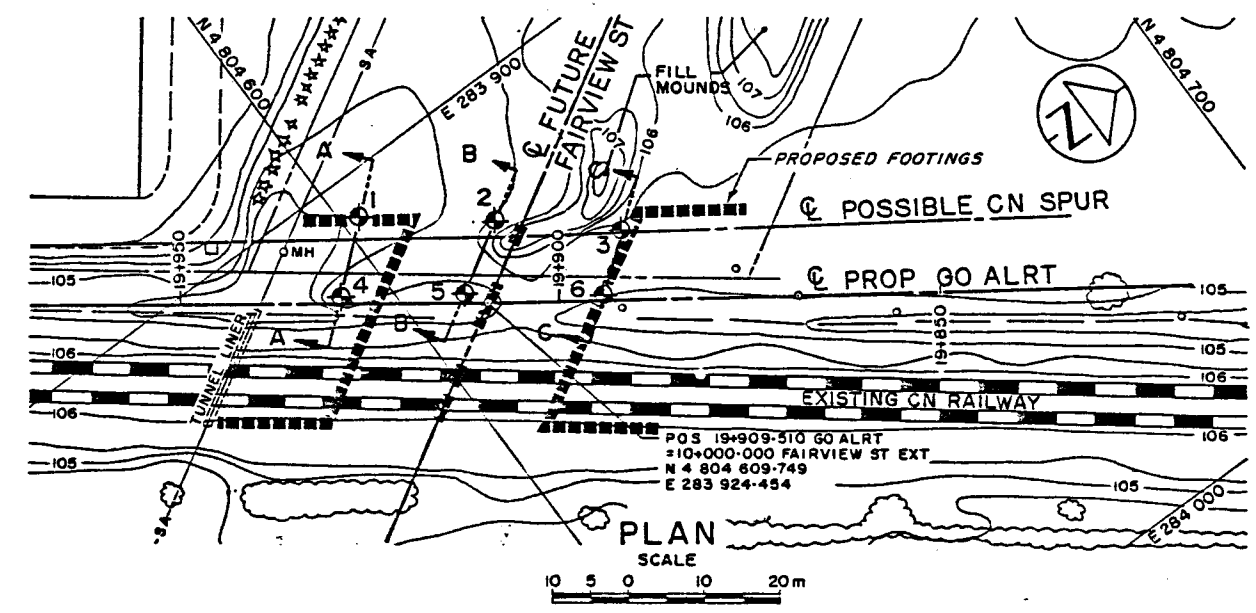
## UNIFIED SOIL CLASSIFICATION SYSTEM



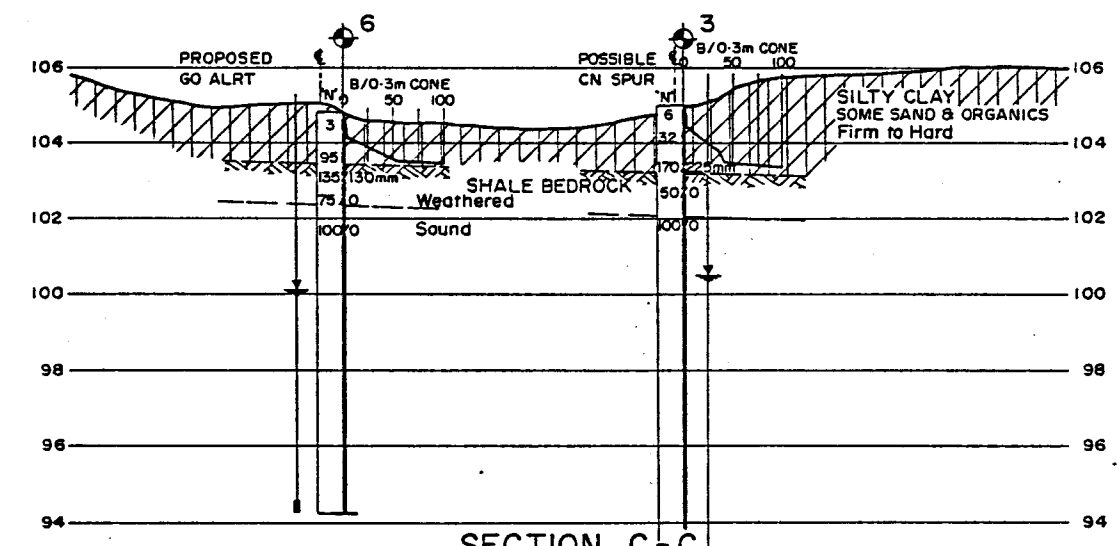
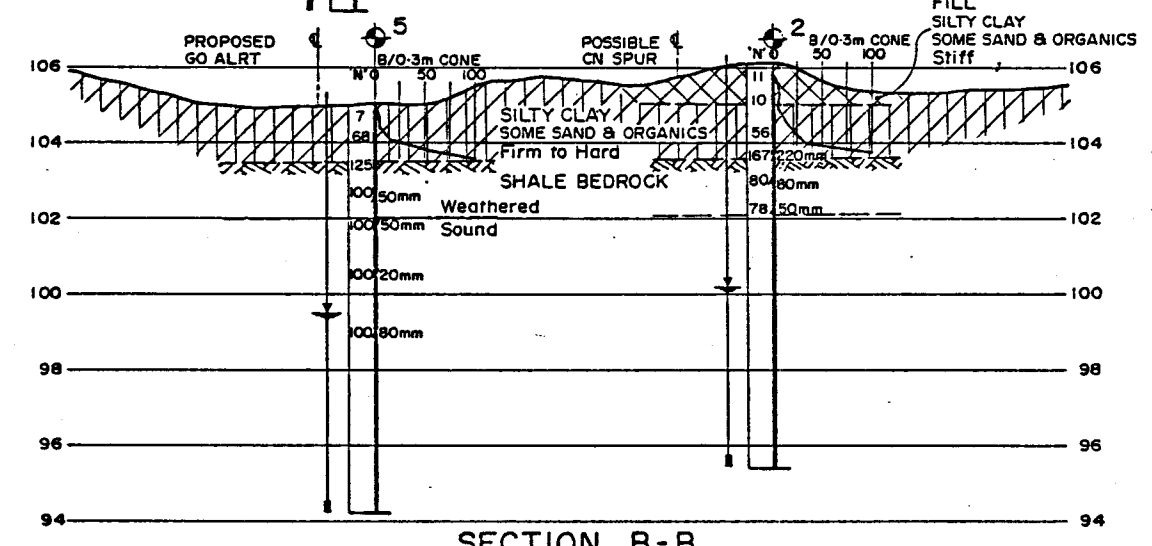
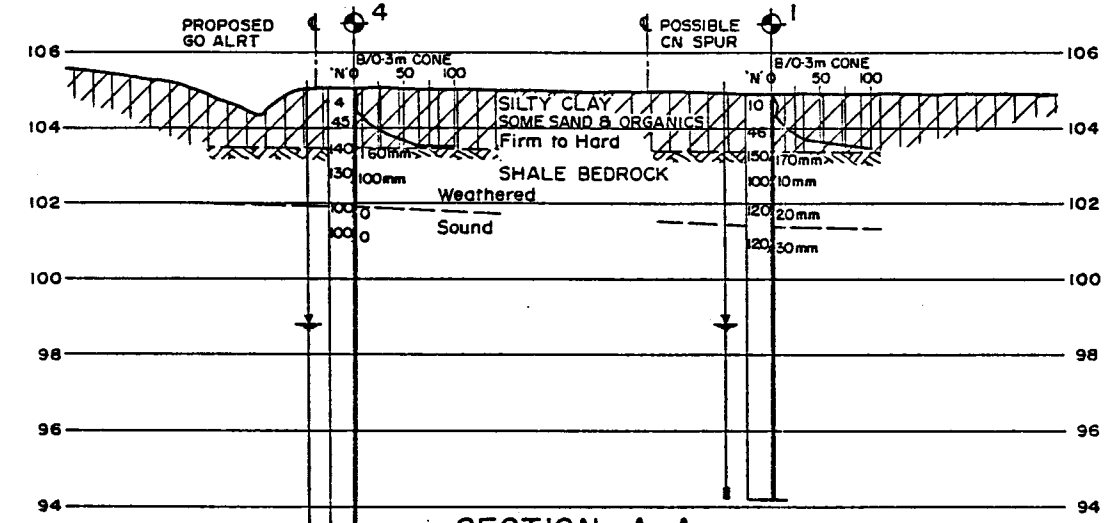
Ministry of  
Transportation and  
Communications

GRAIN SIZE DISTRIBUTION  
SILTY CLAY SOME SAND

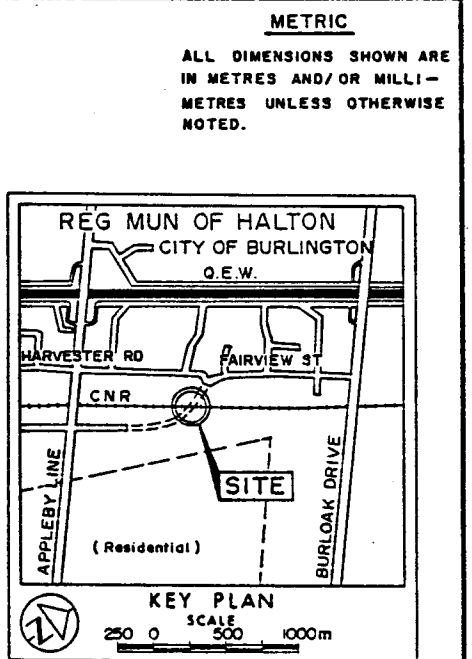
FIG No 3  
W P 2



SCALE FOR PROFILES  
HOR 4 2 0 4 8 12m  
VERT 2 1 0 2 4 6m



SCALE FOR SECTIONS  
2 1 0 2 4 6m



**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)
- WL at time of investigation 1985 01
- PIEZOMETER

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	104.9	4 804 602.3	283 905.4
2	106.1	4 804 616.5	283 916.5
3	105.0	4 804 629.3	283 927.3
4	105.0	4 804 594.0	283 912.5
5	105.0	4 804 607.5	283 921.7
6	104.7	4 804 622.0	283 932.8

Geocres No 3045-151  
=NOTE=  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

GO-ALRT REF PD2-300

REFERENCE DRAWINGS		REVISIONS		DRAWN BY: A. E. LOCKHART 85 01 15 CHK'D BY: DESIGNED BY: APPROVED BY: SCALE: FULL SIZE ONLY AS SHOWN		<b>GEOCON INC.</b> Ministry of Transportation and Communications OAKVILLE PROJECT - WEST EXTENSION	<b>HALTON REGION</b> FAIRVIEW ST EXTENSION BRIDGES BORE HOLE LOCATIONS & SOIL STRATA STA 19+909-510	CONTRACT NO DWG NO REV SHEET
P-016	PARKER CONSULTANTS							
REV 0	GO ALRT - HALTON REGION BRIDGE ON FAIRVIEW ST. EXT. GENERAL ARRANGEMENT I STA 19+909-510 DATED: 84 02 09							