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STR. SITE No. 38S-137

HWY. No. 638

LOCATION Hwy 638 & Coopers Creek

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.           

REMARKS:



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## **FOUNDATION DESIGN SECTION**

# **foundation investigation and design report**

ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION

WP 351-87-01 DIST 18  
HWY 638 STR SITE 38S-137  
Cooper's Creek Culvert

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# FOUNDATION INVESTIGATION REPORT

For

Hwy. 638/Cooper's Creek Culvert

W.P. 351-87-01, Site 38S-137

District 18, Sault Ste. Marie

## INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. A culvert structure has been proposed immediately south of the existing Bailey bridge at the site. A temporary Bailey bridge will be required to detour traffic during the construction of the new culvert. This report includes the subsurface conditions encountered at the site and recommendations pertaining to structure foundations and related earthworks.

## SITE DESCRIPTION AND GEOLOGY

The site is located at the existing Cooper's Creek-Hwy. 638 crossing situated within Dunns Valley in the Township of Galbraith, District of Algoma. The unpaved Hwy. 638 ends approximately 100 m north of the site at the existing Spruce Street and changes to Moose Street north of the Hwy. 638/Spruce Street intersection.

A single lane Bailey Bridge presently exists at the Hwy 638/Cooper's Creek crossing. The Bailey bridge is supported on timber crib abutments which reveal no signs of rotational or translational movements. A two storey residential dwelling is located immediately north of Cooper's Creek and west of Hwy. 638 at the site.

Cooper's creek flows in a westerly direction and meanders in a valley that has a floor width ranging from 3 m to 13 m width and a valley crest width ranging from approximately 10 m to 15 m width. The creek bank slopes are generally very steep and in the order of  $\frac{1}{2}H:1V$ . The slopes are partially exposed and partially covered by low lying shrubs, grassland and trees. There appears to be no visible signs of any slope instabilities. Rip rap exists on the creek bed immediately upstream and downstream of the bridge for scouring protection purposes. The water level in the creek at the time of the investigation was approximately 1 m in depth.

The terrain surrounding the site is generally flat and consists of grassland, shrubland and forestland.

Physiographically, the site is located in the geological domain known as the Thessalon Area. The deposition of overburden in the area is related to the last major advance and retreat of the Wisconsin ice sheet of the Pleistocene Epoch. The glacial deposits are underlain by bedrock of the Precambrian and Paleozoic age.

The overburden deposits are the product of Glacial Lake Algonquin, formed as meltwater between the retreating ice and higher ground to the south. The glacial lake submerged the entire area and deposition was the result of isostatic rebound.

Till consisting of a bouldery silty sand is the oldest surficial sediment in the area. Glaciolacustrine silts and clays are also present in the area. Some of the clay has a reddish colour which is the result of iron staining. Organic deposits have also developed in depressions in the area.

The bedrock is exposed throughout the area producing a "rock knob" and "rock ridge" physiography. This topography is largely the result of differential weathering between the various rock types as well as glacial ice erosion as the glacier moved over the area. The bedrock consists of a complex array of metasedimentary and intrusive rocks.

#### INVESTIGATION PROCEDURES

Soil data and inherent properties were obtained by in situ and laboratory testing conducted. The procedures employed are discussed below.

#### Field Investigation

The fieldwork for the investigation was carried out between 90 06 05 and 90 06 07 and consisted of 4 sampled boreholes advanced to depths ranging from 12.6 to 15.7 m. A dynamic cone penetration test was also advanced to a depth of 29.9 m at BH 1.

Track mounted CME 55 equipment employing hollow stem augering techniques was used to advance the boreholes in the overburden.

In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 6.6 m and at 1.5 m intervals thereafter. Disturbed subsoil samples were retrieved by a split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). Relatively undisturbed samples were also randomly retrieved using a Shelby tube sampler in accordance with standard practice (ASTM D1587). In situ vane tests were also conducted in the cohesive soils, generally at 1.5 m intervals and sequenced within the aforementioned sampling intervals, to determine the undisturbed and remoulded undrained shear strengths of the soil. The test was conducted employing the standard MTO 'N' vane in accordance with ASTM D2573.

All samples were identified in the field and then returned to the laboratory for applicable testing.

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

Survey information related to the location and elevation of boreholes was provided by Northwestern Region Surveys and Plans.

#### Laboratory Analyses

To identify the behaviour, gradation and pertinent properties and characteristics of the soil, various laboratory tests were performed. These tests included:

- 1) Atterberg Limit Tests
- 2) Natural Moisture Contents
- 3) Grain Size Distributions
- 4) Unit Weights
- 5) Consolidation Tests
- 6) Undrained Shear Strength Tests (Unconfined Compression, Undrained Unconsolidated (Quick Triaxial))

Laboratory test results have been summarized in the subsequent section of this report entitled "Subsurface Conditions", and are illustrated on corresponding figures and boreholes included in the attached Appendix.

#### SUBSURFACE CONDITIONS

The soil stratigraphy at the site is generally uniform and consists of a surficial deposit composed of a cohesionless silty sand to a sandy silt. The deposit also contains random interbeds of silt and traces of organics are also present in the deposit. The deposit, which has a very loose denseness, has a thickness that ranges from 3.5 m to 4.4 m extending to an elevation of 228.6 m to 229.5 m. The ground surface beyond the Cooper's Creek valley is generally flat and at an elevation of approximately 233.0 m.

The surficial cohesionless deposit is underlain by an extense deposit of a cohesive clayey silt that also contains random interbeds of plastic silt. The deposit was explored for a maximum thickness of 11.3 m to an elevation of 217.3 m. Although, this deposit was not sampled beyond this depth, it is inferred, based on a dynamic cone penetration test advanced, that this deposit probably extends to a minimum thickness of 25.5 m or elevation 203.1 m.

This cohesive deposit is characterized by liquidity indices exceeding unity. Its consistency is generally firm to stiff.

The boundaries between the various soil types, in situ and laboratory test results as well as groundwater levels established at the time of investigation are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwg. 3518701-A.

A detailed description of the subsurface conditions encountered is given below.

#### Silty Sand to Sandy Silt

The surficial deposit at the site consists of a cohesionless material ranging in composition from a silty sand to a sandy silt. Random interbeds of non-plastic



silt are also present within the deposit ranging in thickness from 50 mm to 0.3 m. Inclusions of organics are also present within the deposit.

A grain size distribution envelope as determined by mechanical sieve analysis illustrating the gradation of this deposit is illustrated in Figure 1 in the Appendix. The envelope reveals that sand and silt percentages range from 37 to 65% and 35 to 63% respectively.

The deposit has been oxidized to a brown colour for the surficial 1.5 to 2.3 m, beyond which the soil is unoxidized and grey. The thickness of the deposit ranges from 3.5 m to 4.4 m extending to an elevation of 228.6 m to 229.5 m.

Standard Penetration tests carried out in the deposit revealed 'N' values ranging from 1 blow/0.3 m to 5 blows/0.3 m indicating a very loose state of denseness.

#### Clayey Silt with random interbeds of Silt

The surficial cohesionless deposit is underlain by a deposit consisting of a grey cohesive clayey silt. The deposit was explored to a maximum thickness of 11.3 m (El. 217.3 m) and random interbeds of plastic silt ranging in thickness from 50 mm to 100 mm were also found to exist within the deposit. A grain size distribution envelope as determined by mechanical sieve and hydrometer analyses, illustrating the composition of this deposit is provided in Figure 2 in the Appendix of the report.

Cohesive deposits are categorized according to their behaviour and consequently, Atterberg Limit tests were carried out to define the plasticity of the soil. The results are plotted in Figure 3 in the Appendix and summarized in Table 1 below.

Table 1 - Clayey Silt

	<u>Range</u>	<u># of Tests</u>
Natural Moisture Content (w%)	20-27	6
Liquid Limit ( $w_L$ %)	4-9	6
Plasticity Index ( $I_p$ )	26-30	6
Liquidity Index ( $\frac{w-w_p}{I_p}$ )	1.4-2.5	6
Unit Weight ( $kN/m^3$ )	18.9-19.1	4
Undrained Shear Strength ( $c_u$ )(kPa)		
- Field Vane	28-92	26
- Lab	29-45	3
Sensitivity	3-5	26

The test results reveal that the deposit is of low plasticity and hence can be categorized as a clayey silt with random interbeds of plastic silt. Natural moisture contents generally exceed the liquid limit of the soil and hence the soil has a liquid index that exceeds unity.

Undrained shear strength measurements ( $c_u$ ) of the soil were obtained by conducting in situ vane tests and laboratory tests, namely undrained unconsolidated (quick triaxial) and unconfined compression tests. The results reveal that the deposit within the investigated depth, has undrained shear strength values ranging from 16 to 92 kPa and hence the soil has an undrained shear strength ranging from soft to stiff. In general, however the soil can be categorized as having a firm to stiff consistency. A shear strength ( $c_u$ ) vs. elevation (m) profile is provided in Figure 4 in the Appendix.

The sensitivity of the soil as defined by the ratio of the undrained strength in the undisturbed state to the undrained strength, at the same water content, in the remoulded state was also determined by the field vane tests and the results are tabulated in Table 1 and identified on the Record of Borehole sheets. Sensitivity values range from 3 to 5 indicating that the soil has a low sensitivity.

Standard Penetration tests carried out in this stratum revealed 'N' values ranging from 1 blow/0.3 m to 5 blows/0.3 m confirming the relatively weak consistency of the soil.

The results ( $e$ -log  $p$  curves) of consolidation tests on representative samples of the clayey silt material that illustrate the compressibility characteristics of the soil are shown in Figures 5 and 6 in the Appendix. The results reveal that the soil ranges from an underconsolidated state to a slightly preconsolidated state. It appears that with increasing depth within the deposit, the soil has been preconsolidated. At El. 223.5 m (8.5 m depth), test results reveal that the soil has been preconsolidated to approximately 45 kPa in excess of the existing overburden pressure. However, for the thickness of the deposit above this elevation, the deposit is generally in an underconsolidated to normally consolidated state. This signifies that the present effective overburden

pressures are the maximum pressures to which the deposit has ever been consolidated at any time in its history. In fact, for underconsolidated deposits, the deposit may not be fully consolidated under the present effective overburden pressure. The preconsolidation pressure is generally within a range of 30-35 kPa below the effective overburden pressure.

The pertinent properties of the subsoil samples tested for consolidation are provided in the Figures 5 and 6 previously mentioned. The results reveal that the initial void ratio of the soil ranges from 0.84 to 0.86 and the compression indices range from 0.1 to 0.13.

The coefficient of consolidation ( $c_v$ ) used to determine the time rate of consolidation settlement was computed using Taylor's Method (1948). The value ranges from 0.03  $m^2/day$  to 0.05  $m^2/day$ .

#### GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes. Groundwater levels determined at the time of investigation were approximately 3.0 m below the natural ground surface (El. 230 m). These water levels correlate approximately to the water level of the flowing Cooper's Creek.

Groundwater levels, in general, are subject to seasonal fluctuations and hence can vary from the values given in this report.

## DISCUSSION AND RECOMMENDATIONS

It is proposed to replace the existing Bailey bridge at the site with a new concrete box culvert. The proposed culvert is to be located immediately south of the existing Bailey bridge and will necessitate some realignment of the existing Cooper's Creek.

Either a cast-in-place or precast concrete culvert has been proposed. The length, width and height of the culvert are in the order of 30 m, 7 m and 3 m respectively. The proposed invert elevation is 230.1 m whilst the proposed grade elevation is 234.1 m. Consequently, a culvert roof cover of approximately 1 m and approach fills in the order of 4 m will be required.

A temporary detour has been planned approximately 26 m east of the existing Hwy. 638. A Bailey bridge is proposed at the detour-creek crossing. A plan illustrating the proposed new concrete box culvert and detour location is shown on Dwg. 3518701-B in the Appendix.

Recommendations pertaining to the following foundation and geotechnical considerations are included in the purview of this report.

- 1) Structure Foundations
- 2) Approach Fills
- 3) Channel Realignment
- 4) Construction Considerations

- 1) Structure Foundations

In view of the weak, normally consolidated nature of the native subsoil at the site and the substantial soil deformation anticipated as a result of applied vertical loading, it is recommended that the proposed precast or cast-in-place concrete culvert be replaced with a steel pipe arch culvert. The steel pipe arch culvert is structured to eliminate any vertical loading at the foundation. The steel pipe arch culvert is also a more flexible structure and hence can tolerate greater differential soil displacements than a rigid concrete culvert structure. In addition, rigid culvert structures attract additional vertical shear forces caused by settlement of the fill adjacent to the structure. Recommendations for the foundation design of the steel pipe arch culvert are given below.

### Structural Pipe Culvert

The foundation for the structural pipe culvert can be founded at the proposed invert elevation (230.1 m) provided it is constructed in accordance with current MTO bedding and backfilling requirements as specified in OPSS 421 and OPSD 802 series. The major items of consideration are summarized below.

- 1) The bedding should consist of a granular pad (Granular 'A') with a minimum thickness of 300 mm. The excavation for the bedding shall extend to a width of a minimum 1.5 m on either side of the culvert.
- 2) All loosened, disturbed material created during construction of the structure foundation and any deleterious or organic material present at the foundation founding elevation shall be removed and replaced with a granular material.
- 3) For the width of the area under the bottom radius of the pipe arch the bed should be levelled and left uncompacted for a depth of 300 mm below the invert level.
- 4) The culvert pipe bed is to be carefully shaped to receive the lowest segment of pipe formed by the bottom radius.
- 5) The area adjacent to the haunches of the pipe and under the portion of the sloping invert should be compacted by means of hand tamping.
- 6) The minimum depth of cover shall be the span of the pipe culvert divided by 6 or 300 mm, whichever is greater.
- 7) The pipe culvert shall be designed with a camber of 100 mm located at mid-distance between the inlet and outlet to account for anticipated settlements (see "Approach Fills-Settlement" in subsequent section of report).
- 8) Scour protection at the culvert inlet and outlet shall be provided to protect the culvert foundation. The design of the scour protection shall be made in conjunction with applicable hydrological parameters.

### Backfill to Structure

Backfill for the plate pipe culvert shall be designed and constructed according to OPSD 803 series. The following items of consideration are hereby reinforced.

- 1) The frost penetration depth at the site is 2.2 m and the frost taper should be designed accordingly. Adequate frost protection shall be provided for footings subject to frost penetration as for instance during winter construction.
- 2) The backfill material should be machine compacted on both sides of the pipe simultaneously in equal lifts in accordance with OPSS 501.08.02.
- 3) To prevent piping around the culvert, a 1 m thick blanket of approved impermeable material (refer to OPSS 1205) should be placed at the culvert inlet as a sealer behind a 600 mm layer of rip-rap. This blanket should extend to the high water level. Around the culvert outlet, a 1 m thick blanket of Granular 'A' material should be placed as a filter behind a 600 mm layer of rip-rap.
- 4) The passive resistance of the granular backfill and any other applicable earth pressure can be calculated using the properties tabulated in Table 2.

### Approach Fills

#### Materials

Approach fills in the order of magnitude of 4 m will be required immediately adjacent to the culvert walls. It is recommended that this material consist of a free draining material such as Granular 'A' or Granular 'B' to prevent hydrostatic pressure build-up on the culvert walls. Design parameters of the soil are given in Table 2 below. The angles of internal friction given are unfactored.

Table 2 - Backfill Properties

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction ( $\phi$ )	35°	30°
Unit Weight (kN/m <sup>3</sup> )	22.8	21.2
*Coefficient of Active Earth Pressure (Ka)		
- S.L.S.	0.27	0.33
- U.L.S.	0.33	0.4
*Coefficient of Earth Pressure at Rest (Ko)		
- S.L.S.	0.43	0.5
- U.L.S.	0.5	0.58

\*Horizontal surface backfill only. Appropriate consideration must be given to sloping surface backfill.

For a rigid and unyielding structure, the earth pressure coefficient at rest is to be used in computing lateral earth pressure.

The backfill beyond the granular wedge as illustrated on OPSD 803 series can consist of acceptable borrow material as defined in OPSS 212.05.

### Stability

Stability computations were carried out to determine both the overall (global) stability and internal stability of the fills in the longitudinal direction of the culvert. The analysis was carried out in terms of total stress or the undrained (short term) condition applying Bishop's Modified Method digitized on an in-house mainframe program. The analysis was conducted employing a factor of safety of 1.3 and circular slip surfaces. Properties of the fill material and subsoil parameters used in the analysis are summarized in Table 3 below. Should the approach fill material differ significantly from the material properties indicated, this office should be contacted for further examination and evaluation.

Table 3 - Slope Stability Soil Parameters

<u>Soil</u>	<u>El.</u> <u>(m)</u>	<u>Unit Weight*</u> <u>(kN/m<sup>3</sup>)</u>	<u>Shear Strength Parameters</u>	
			<u>(cu) (kPa)</u>	<u>(<math>\phi</math>) (°)</u>
Fill	234-230	20	-	30
Clayey Silt	230-227	19	30	-
	227-222	19	40	-
	<222	19	70	-

\*Buoyant unit weights were applied for soils submerged below groundwater table.

The results of the analysis reveal that no internal nor external instabilities are anticipated for embankment fill heights up to 6 m. All slopes should be protected against surface erosion. This can be accomplished by conventional sodding application.

#### Settlement

The prediction of the magnitude of settlement induced by the placement of the embankment fill consisted of the computation of:

- a) Elastic (immediate) settlement induced in the clayey silt.
- b) Elastic settlements induced within the fill itself.
- c) Primary consolidation settlement of the clayey silt deposit.

A unit weight of fill of  $20 \text{ kN/m}^3$  was employed for all settlement calculations.

The elastic settlement within the native clayey silt deposit was determined using Hooke's law assuming a homogeneous, isotropic mass whereas the elastic settlement within the fill itself was deduced as a percentage (0.5%) of the fill height. It is anticipated that the sum of the elastic settlements will be in the order of 50 mm. These settlements are expected to be realized during or immediately following construction.

The primary consolidation settlement of the clayey silt stratum was predicted using one dimensional consolidation theory. The load-deformation curves obtained by laboratory testing were applied in the computation. The stress distribution and the increase in vertical stress in the native soil as a result of the applied fill loading was determined by assuming an equivalent uniformly loaded rectangular area (after Steinbrenner, 1936).

The total primary consolidation settlement predicted is in the order of magnitude of 100 mm. It is expected that approximately 40% of the settlement is



to be realized within two years subsequent to construction. The remainder of the settlement is time dependent with 90% degree of consolidation expected within 13 years.

It is recommended that because the proposed highway realignment is a secondary highway, the post construction settlements be repaired by conventional routine resurfacing methods.

#### Channel Realignment

The realignment of the Cooper's Creek will necessitate excavation cuts in the order of magnitude of 4 m primarily in the surficial silty sand to sandy silt with random interbeds of silt deposit. The excavated slopes shall be no steeper than 2H:1V. The slopes shall be protected against scour in compliance with the hydrological specifications at the site.

#### CONSTRUCTION CONSIDERATIONS

##### Temporary Diversion

To facilitate the construction of the culvert whilst maintaining traffic along Hwy. 638, a detour has been proposed approximately 26 m east of the existing Hwy. 638. A Bailey bridge is proposed to span Cooper's Creek along this alignment. The Bailey bridge can be supported on conventional timber crib abutments founded in the native surficial silty sand to sandy silt deposit at a depth below the frost penetration (2.2 m). The following bearing capacities are provided for the design of the Bailey bridge foundations.

Table 4 - Bailey Bridge Timber Crib Foundation

Bearing Capacity at S.L.S. Type II	50 kPa
Factored Capacity at U.L.S.	75 kPa

The structure foundations shall be located a minimum 3.0 m beyond the crest of the Cooper's Creek slope.

Settlements of the timber crib foundations are anticipated as a result of the elastic compression of the founding soil and also as a result of primary consolidation of the clayey silt deposit at the site. For the tabulated applied pressures, settlements in the order of magnitude of 50 mm can be anticipated, assuming a footing width of 4 m. It is recommended that the Bailey bridge be periodically monitored for the development of these settlements and the structure adjusted as required.

#### Dewatering

To facilitate the construction of the culvert foundations, a dewatering scheme will be required. The excavation for the culvert will be advanced in the surficial silty sand to sandy silt deposit, soils that typically slough and boil under conditions of unbalanced hydrostatic head. Although part of the excavation may be above the prevailing groundwater table excavation will also be required beneath the groundwater table. To control these unfavourable conditions during culvert foundation and backfilling construction, it is recommended that an oversized excavation composed of perimeter ditches and a sump pumping discharge system be used to drain accumulated water. Soil migration during the dewatering process must be adequately controlled to prevent potential undermining of the footing bed and inevitable resulting settlements. A properly designed filter fabric placed on the excavated slopes can be used to achieve the retention of soil migration.

#### Founding Soil Preservation

To protect the founding soil from disturbance caused by construction activity and weathering, it is recommended that a granular working slab be placed on the founding soil.

#### MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, and M. Iampietro, Student Engineer, utilizing equipment owned and operated by Master Soils Investigation.

The project was carried out by T. Sangiuliano under the general supervision of Dr. B. Iyer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed and approved by Dr. B. Iyer.



A handwritten signature in cursive script, appearing to read "T. Sangiuliano".

T. Sangiuliano, P.Eng.  
Foundation Engineer

A handwritten signature in cursive script, appearing to read "B. Iyer".

for

M.S. Devata, P.Eng.  
Chief Foundation Engineer

## **APPENDIX**

## EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINTING AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### 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

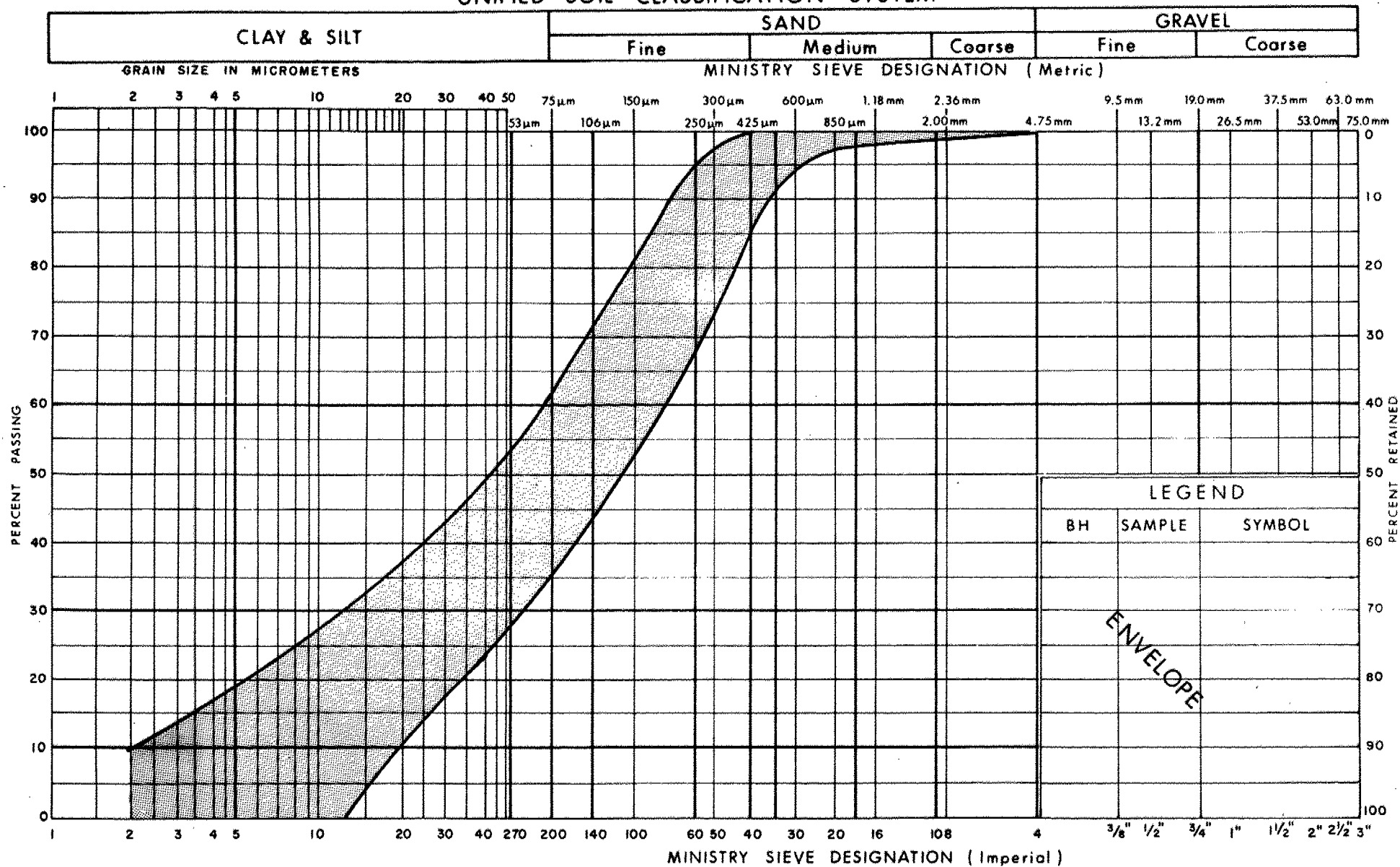
### MECHANICAL PROPERTIES OF SOIL

$m_v$	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 = $w_L - w_p$	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						

## UNIFIED SOIL CLASSIFICATION SYSTEM



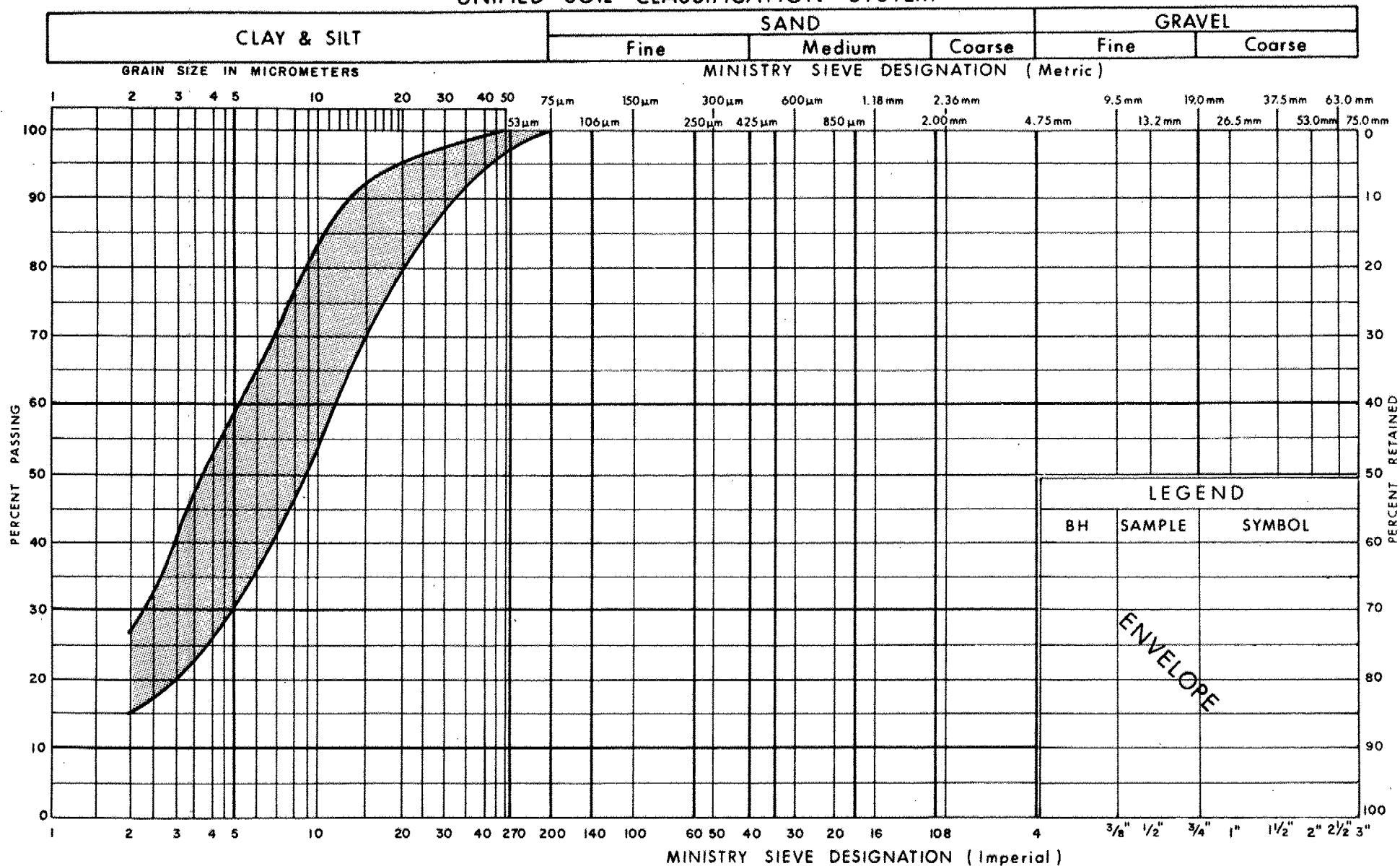
Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
**SILTY SAND TO SANDY SILT**  
**WITH RANDOM INTERBEDS OF SILT**

FIG No 1

W P 351-87-01

## UNIFIED SOIL CLASSIFICATION SYSTEM

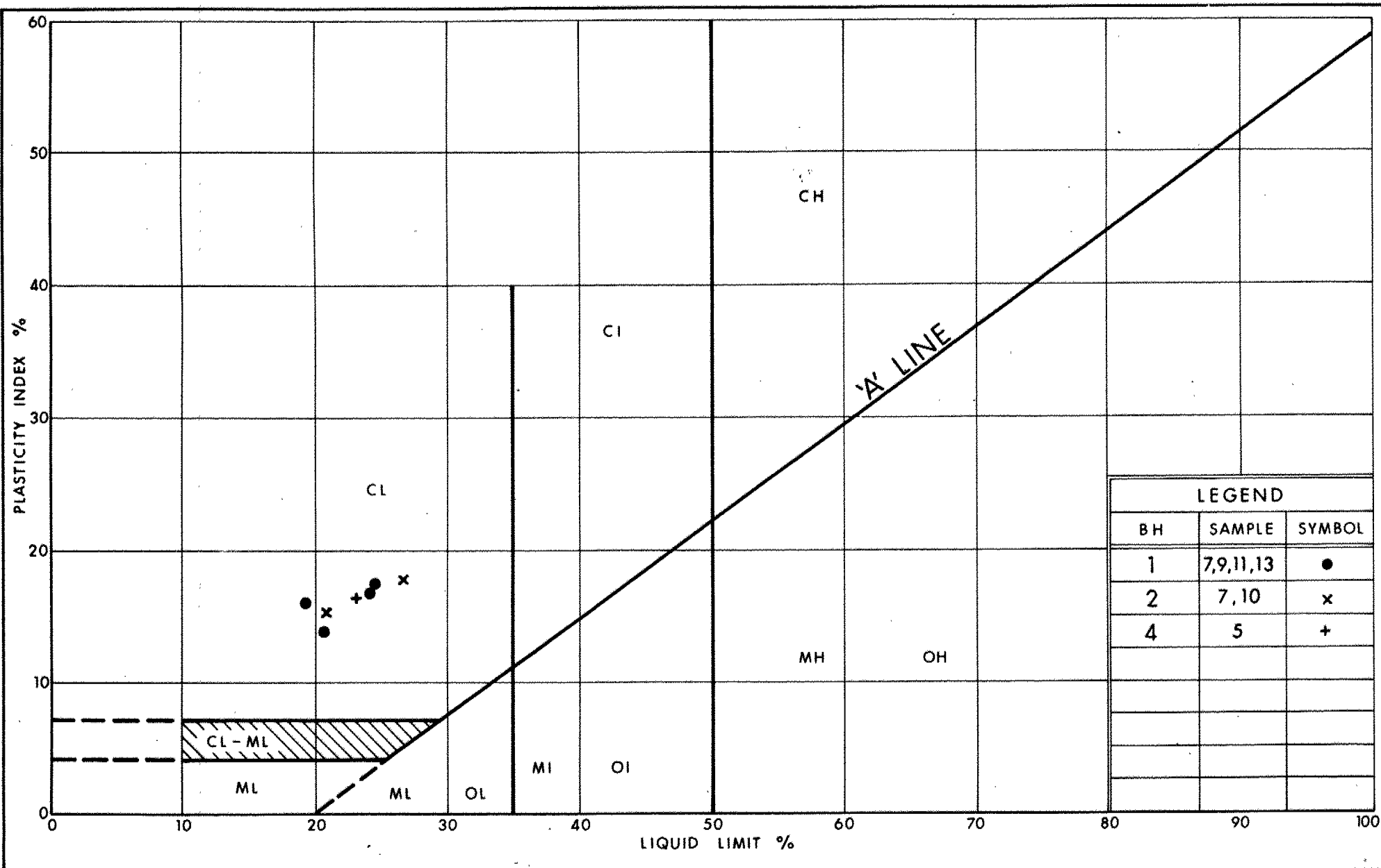


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
CLAYEY SILT WITH RANDOM INTERBEDS OF SILT

FIG No 2

W P 351-87-01



Ministry of  
Transportation

Ontario

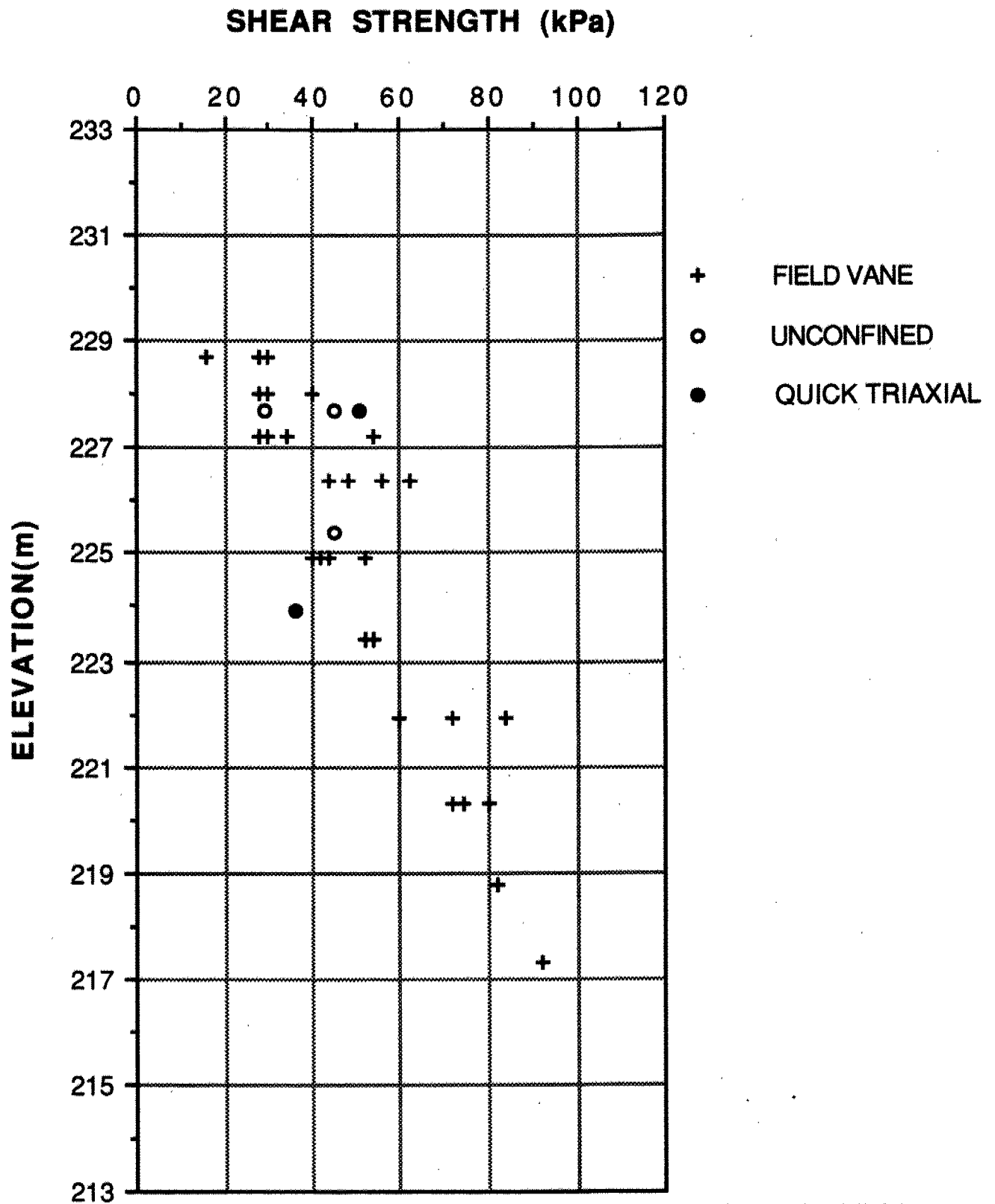
# PLASTICITY CHART CLAYEY SILT WITH RANDOM INTERBEDS OF SILT

FIG No 3

W P 351-87-01



**FIGURE 4**  
**SHEAR STRENGTH(CU) Vs ELEVATION**



# VOID RATIO - PRESSURE CURVES

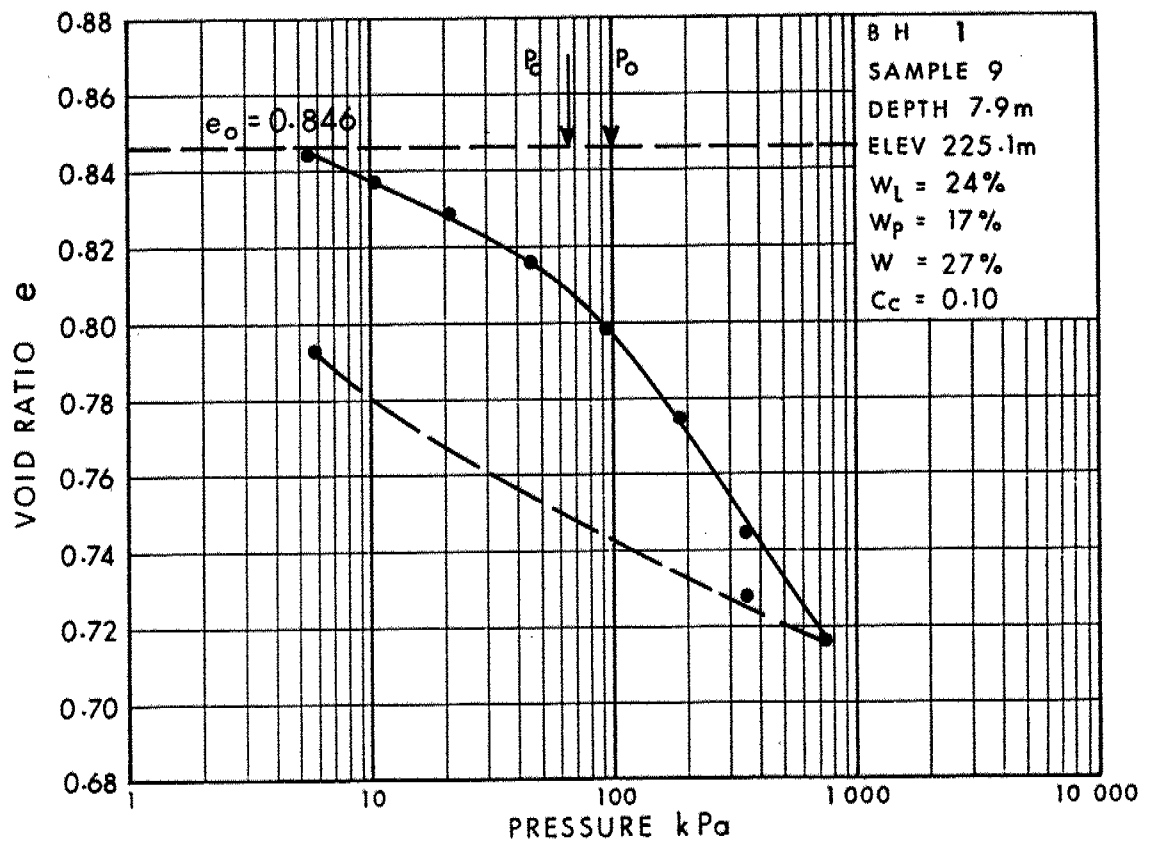
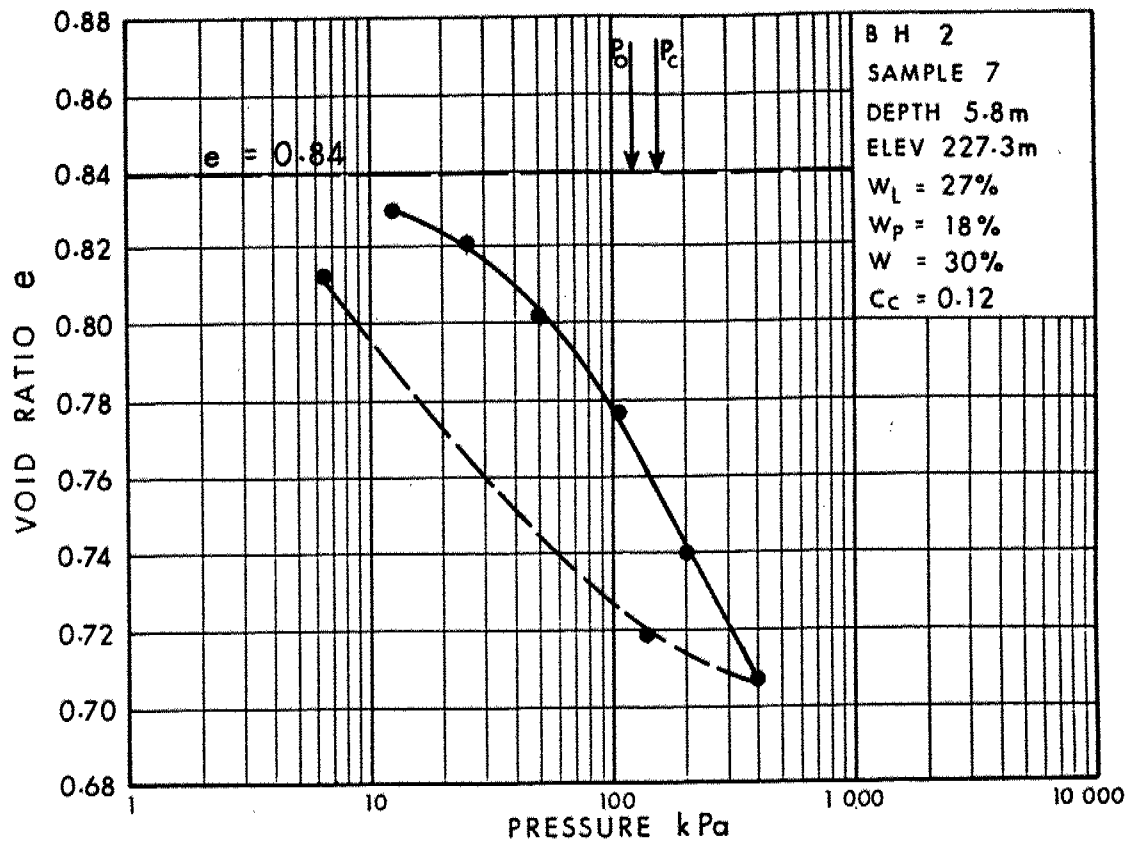


Fig 5

WP 351-87-01

# VOID RATIO - PRESSURE CURVES

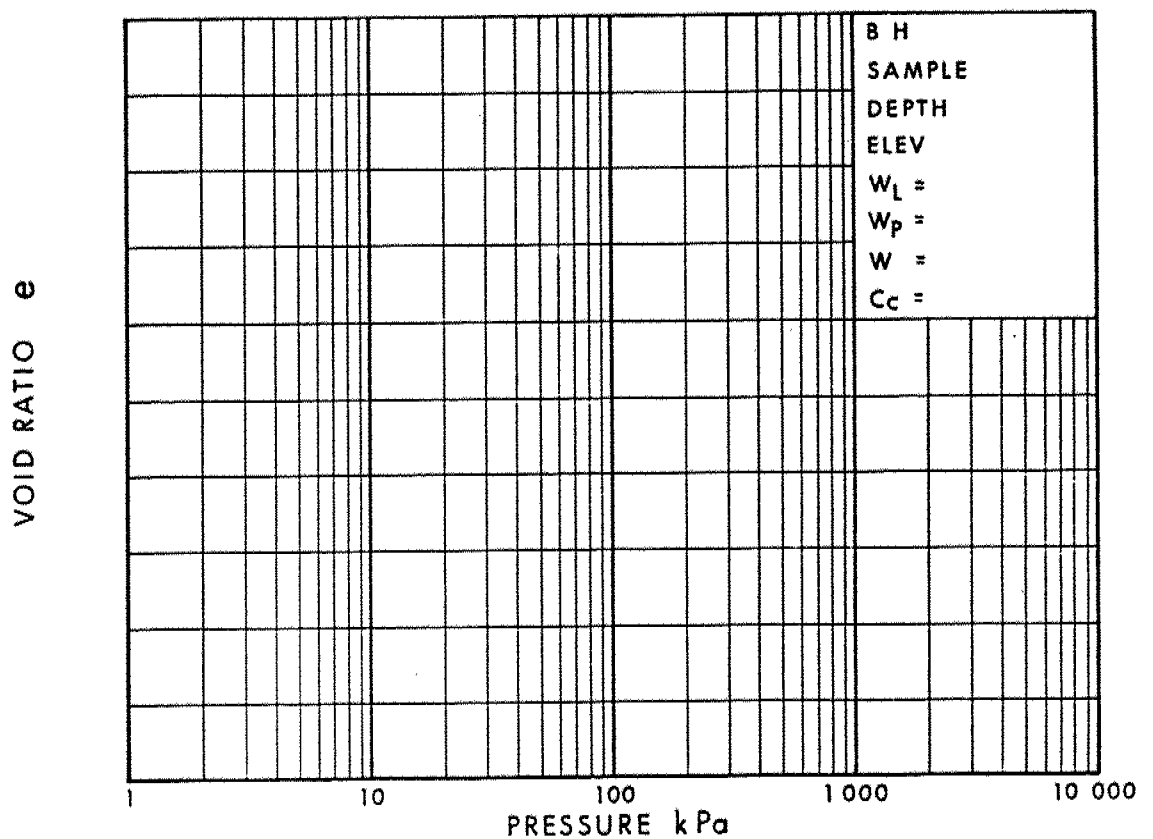
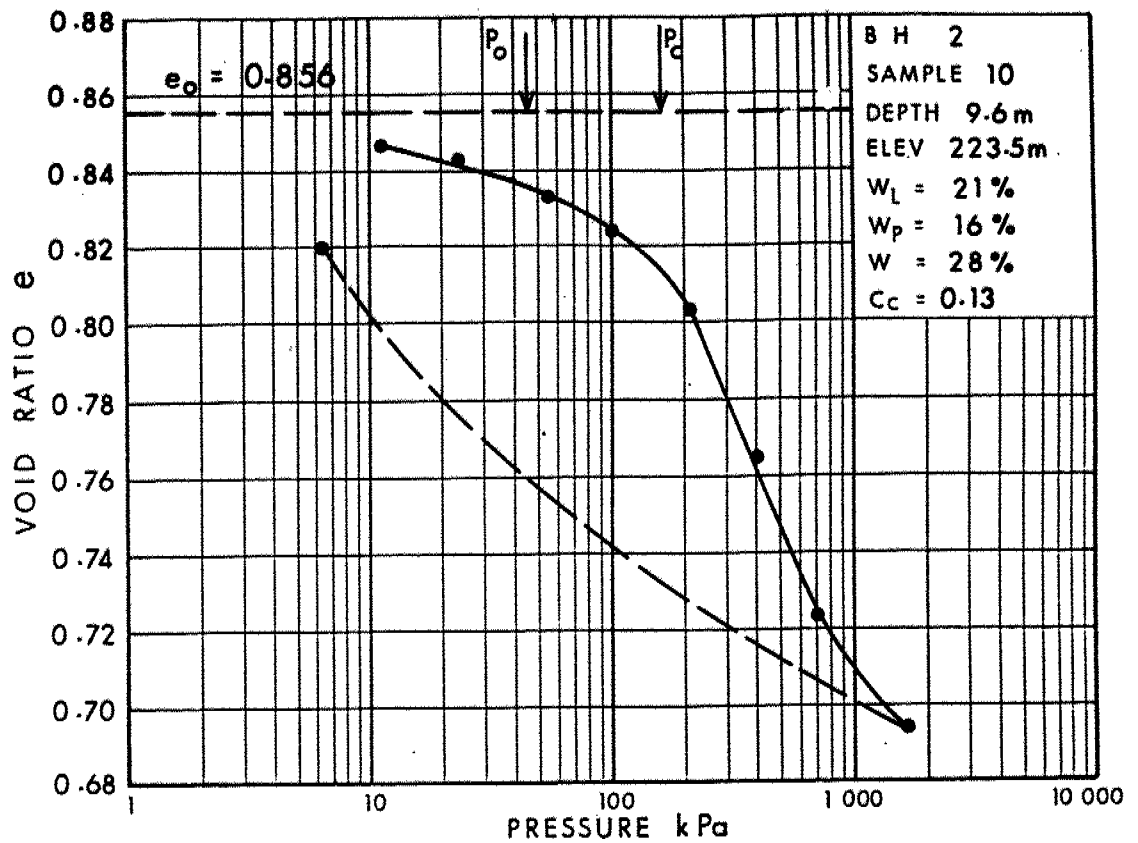


Fig 6

W P 351-87-01

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 351-87-01 LOCATION Sta. 19 + 327.6; O/S 13.6m Lt (Line 'A') ORIGINATED BY MI  
DIST 18 HWY 538 BOREHOLE TYPE HS Auger COMPILED BY TS  
DATUM Geodetic DATE 90 05 05-06 CHECKED BY TS

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 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
233.0	Ground Surface																
0.0	Silty Sand with random interbeds of silt  Brown Grey  V. Loose		1	CS	—												
			2	SS	5												
			3	SS	1												
			4	SS	2												
228.6			5	SS	4												
4.4	Clayey Silt with random interbeds of silt Grey, Firm to Stiff		6	SS	1												
			7	TW	PH												
			8	TW	PH												
			9	TW	PH												
			10	SS	1												
			11	TW	PH												
			12	SS	3												
			13	SS	1												
217.3			14	SS	2												
15.7			End of Borehole														
203.1																	
29.9	End of Cone Test																

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

20  
15-5 (%) STRAIN AT FAILURE  
10

# RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 351-87-01 LOCATION Sta 19 + 331: 0/S 13.2m RT (Line 'A') ORIGINATED BY MI  
 DIST 18 HWY 638 BOREHOLE TYPE HS Auger COMPILED BY TS  
 DATUM Geodetic DATE 90 06 06 CHECKED BY TS

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					
233.1	Ground Surface																
0.0	Brown Gray Silty Sand with random interbeds of silt V. Loose		1	CS	-		232										0 41 49 10
			2	SS	1												0 65 (35)
			3	SS	1												
			4	SS	1												
228.7			5	SS	3												
4.4			6	SS	2		228									18.9	0 0 75 25
			7	TW	PH												
			8	TW	PH												
	Clayey Silt with random zones of silt Gray, Firm to Stiff		9	TW	PH		226										
			10	TW	PH												
			11	SS	1		224									18.9	0 0 80 20
220.5			12	SS	1		222										
12.6	End of Borehole																

# RECORD OF BOREHOLE No 3

1 OF 1

METRIC

W.P. 351-87-01 LOCATION Sta. 19 + 378.4; O/S 23.2 m Rt (Line 'A') ORIGINATED BY MI  
 DIST 18 HWY 638 BOREHOLE TYPE HS Auger COMPILED BY TS  
 DATUM Geodetic DATE 90 06 07 CHECKED BY TS

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 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							20 40 60 80 100	20 40 60 80 100	10 20 30					
233.0	Ground Surface													
0.0	Brown, trace organics		1	CS	-									
	Grey		2	SS	1									
	Sandy Silt with random interbeds of silt		3	SS	1									
229.5	V. Loose		4	SS	1									
3.5			5	SS	1									
			6	SS	1									
			7	SS	1									
			8	TW	PH									
	Clayey Silt with random interbeds of silt													
	Grey, Firm to Stiff		9	SS	2									
			10	TW	PH									
			11	SS	4									
220.4			12	SS	3									
12.6	End of Borehole													

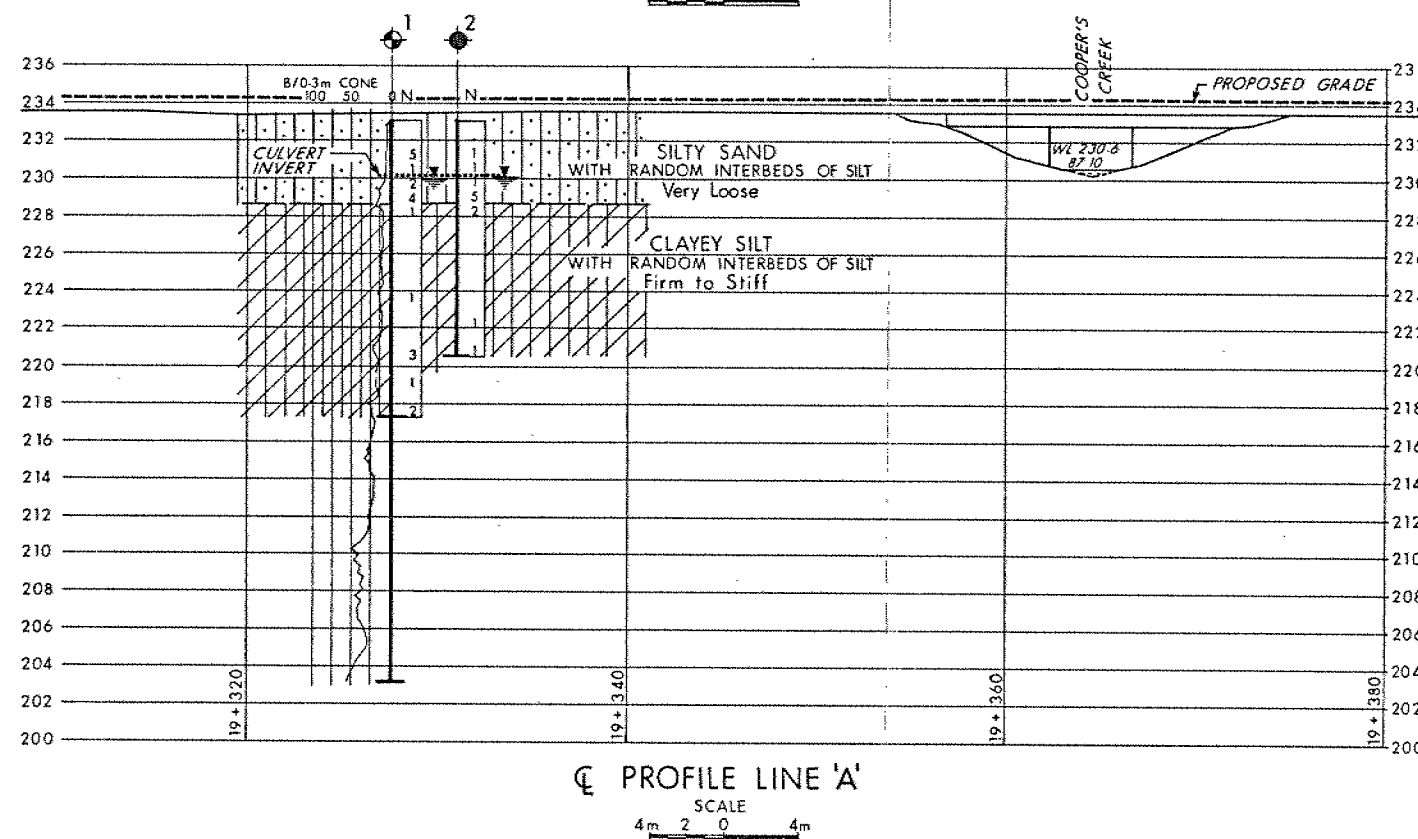
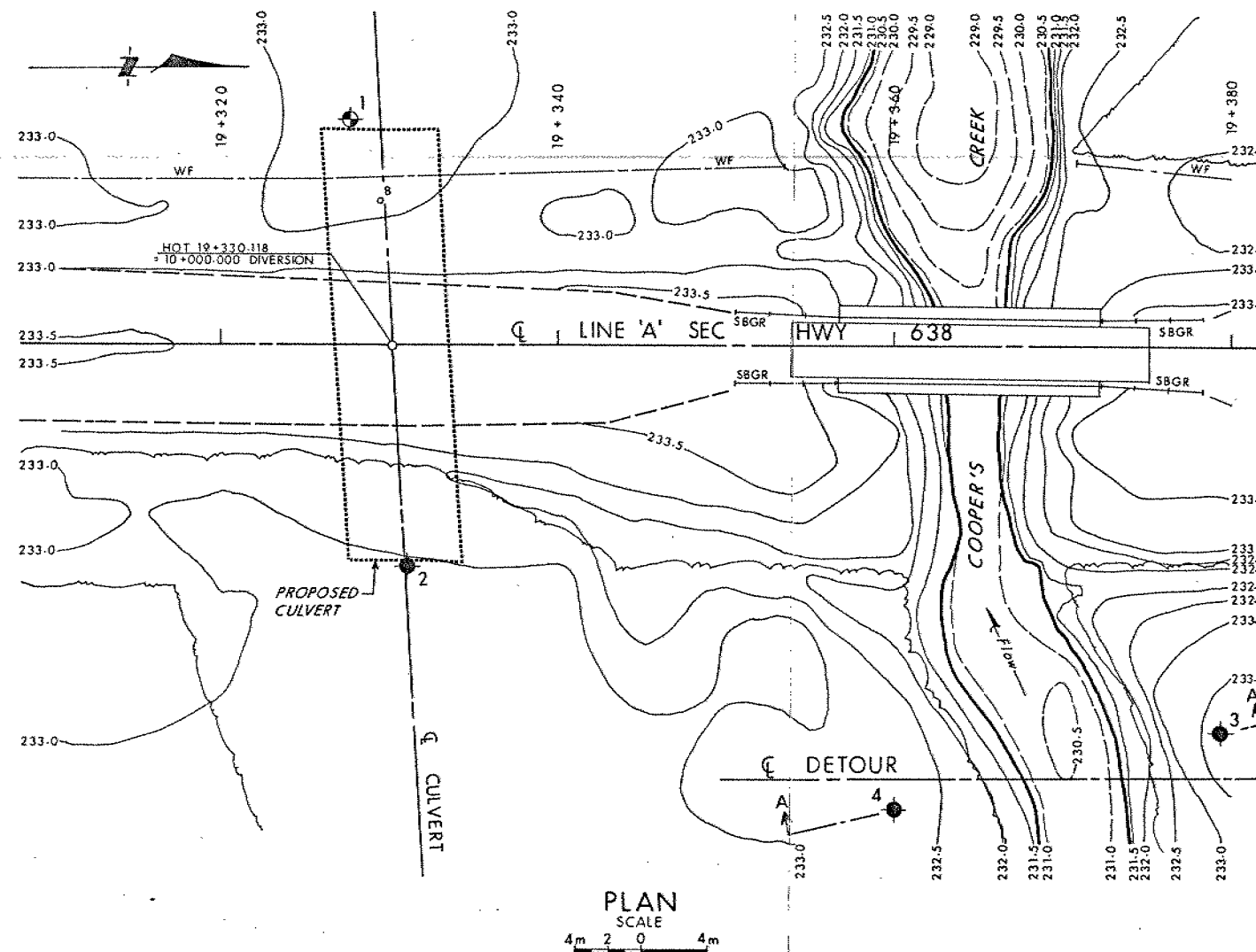
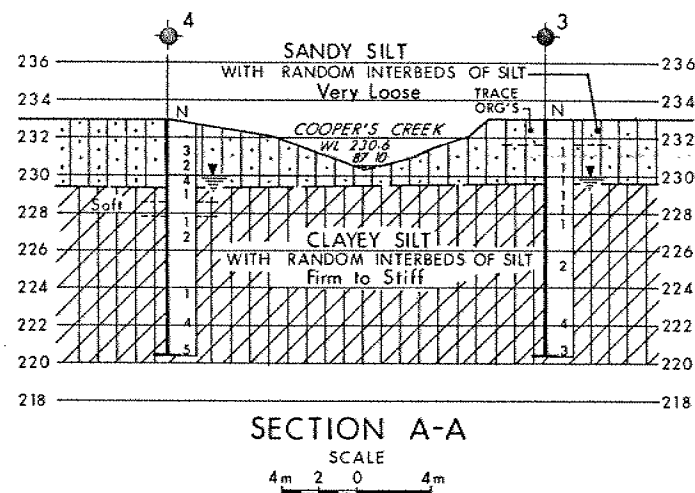
# RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 351-87-01 LOCATION Sta 19 + 360; O/S 27.8m Rt. (Line 'A') ORIGINATED BY MI  
 DIST 18 HWY 538 BOREHOLE TYPE HS Auger COMPILED BY TS  
 DATUM Geodetic DATE 90 06 08 CHECKED BY TS

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					
233.0	Ground Surface																
0.0																	
	Brown		1	CS	-		232										
	Grey		2	SS	3												
	Sandy Silt with random interbeds of silt, trace organics		3	SS	2												
229.3	V. Loose		4	SS	4		230										
3.7			5	SS	1												
	Soft		6	TW	PH		228										
			7	SS	1												
			8	SS	2												
							226										
			9	TW	PH												
							224										
			10	SS	1												
			11	SS	4		222										
220.4																	
			12	SS	5												
12.6	End of Borehole																



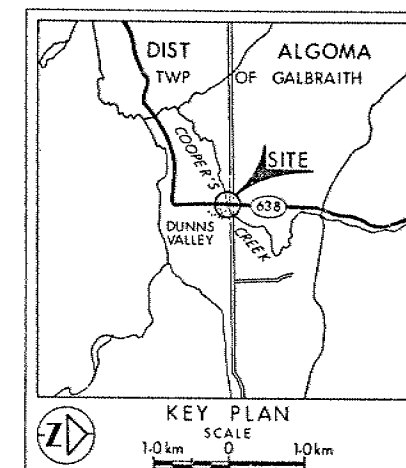
**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
IN KILOMETRES + METRES.

CONT No  
WP No 351-87-01

COOPER'S CREEK  
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)
- WL at time of investigation 90 06

No	ELEVATION	STATION	OFFSET
1	233.0	19+327.6	13.6 m Lt
2	233.1	19+331.0	13.2 m Rt
3	233.0	19+379.4	23.2 m Rt
4	233.0	19+360.0	27.8 m Rt

**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.

REV.	DATE	BY	DESCRIPTION
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Geocres No 41J-50

HWY No 638	DATE 90 09 06	SITE 385-137
SURM'D TS	CHECKED	DWG 3518701-A
DRAWN DT	CHECKED	