

G.I.-30 SEPT. 1976

GEOCRES No. 417-49DIST. 18 REGION W.P. No. 350-87-01CONT. No. W. O. No. STR. SITE No. 385-136HWY. No. 638LOCATION Hwy 638 & Wannamaker
CreekNo of PAGES -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 350-87-01 DIST 18
HWY 638 STR SITE 38S-136

Wannamaker Creek Culvert

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FOUNDATION INVESTIGATION REPORT
For
Hwy. 638/Wannamaker Creek Culvert
W.P. 350-87-01, Site 38S-136
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 to replace an existing Bailey bridge at the site. The subsurface conditions encountered at the site and recommendations pertaining to structure foundations and related earthworks are included in the scope of this report.

SITE DESCRIPTION AND GEOLOGY

The site is located approximately 90 m south of the existing Wannamaker Creek-Hwy. 638 crossing situated within Dunn's Valley in the Township of Galbraith, District of Algoma. A single Bailey bridge presently carries the unpaved Hwy. 638 over Wannamaker Creek at the site. A rock fill-timber crib supports the approach fills at the bridge abutments. There appears to be no signs of rotational nor translational abutment displacement.

Wannamaker Creek flows in a south to southwesterly direction and meanders in a valley that has a crest width in the order of 10 m and a floor width of 5-6 m. The depth of the creek valley is in the order of 3 m and valley slopes are generally 1H:1V or steeper. The water level in the creek at the time of the investigation was approximately 1 to 1.5 m.

The area surrounding the site is generally flat and the land is covered by shrubs and forest. Approximately 200 m northwest of the site, a rock ridge approximately 0.5 km in height exists.

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.

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 07 and 90 06 08 and consisted of two sampled boreholes advanced to depths of 13.0 m. A dynamic cone penetration test, advanced to a depth of 25.9 m also accompanied one of the boreholes (BH 1).

The boreholes were advanced using conventional hollow stem augering techniques. A track mounted continuous flight auger drill rig was employed for the operation.

In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 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 shelly tube sampler in accordance with standard practice (ASTM D1587). In situ vane tests were also conducted in sequence between the aforementioned sampling intervals to determine the undisturbed and remoulded undrained shear strengths of soil. The test was conducted employing the standard MTO 'N' vane in accordance with ASTM D2573.

All subsoil samples were identified in the field and returned to the laboratory for further examination and applicable testing.

Water levels monitored throughout the duration of the investigation were obtained in the open boreholes. All boreholes were backfilled upon completion of the fieldwork.

Survey information related to the location and elevation of boreholes was provided by the 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) Grain Size Distributions
- 3) Unit Weights
- 4) Natural Moisture Contents
- 5) Undrained Shear Strength Determination (Unconfined Compression, Quick Triaxial)
- 6) Consolidation Tests

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 subsurface conditions at the site are generally uniform and consists of an extensive deposit of a cohesive clayey silt with random interbeds of cohesionless silt. The extent of the deposit was not determined during the investigation.

A detailed description of the subsurface conditions, including soil descriptions, 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 and summarized below. A plan of the site illustrating the locations and elevations of the boreholes and subsoil stratigraphical sections are provided on Dwg. 3508701-A.

Clayey Silt with random interbeds of Silt

The surficial deposit at the site consists of a cohesive clayey silt containing random interbeds of cohesionless plastic silt. The thickness of the deposit was not determined during the investigation but it can be inferred from data obtained from a dynamic cone penetration test that the deposit probably extends to a minimum depth of 25.9 m or an equivalent elevation of 202.9 m. The deposit has been oxidized for an approximate surficial thickness of 1.5 m and hence is brown in colour within this upper zone. Below the oxidized zone, the deposit is unoxidized and grey.

Random interbeds of plastic silt are also present within the deposit. The thickness of these interbeds are generally in the order of 25 mm to 75 mm.

The clayey silt deposit is comprised primarily of clay and silt grain sizes ranging generally from 25-35% and 65-75% respectively. Traces of organics and traces to some sand are also present within the surficial 1.5 to 2.0 m of the deposit. A grain size distribution envelope for this deposit as determined by mechanical sieve and hydrometer analysis is given in Figure 1 in the Appendix.

In view of the cohesive nature of the deposit, the deposit is categorized according to its behaviour and consequently, Atterberg Limit tests were carried

out to define the plasticity of the soil. The results are plotted in Figure 2 in the Appendix and summarized in Table 1 below.

Table 1 - Clayey Silt

	<u>Range</u>	<u># of Tests</u>
Liquid Limit ($w_L\%$)	24-31	7
Plasticity Index (I_p)	6-12	7
Natural Moisture Content ($w\%$)	26-38	7
Liquidity Index ($\frac{w-w_p}{I_p}$)	1.1-2.1	7
Unit Weight (kN/m^3)	18.2-18.8	4
Undrained Shear Strength (c_u)(kPa)		
- Field Vane	12-40	17
- Lab	16-23	4
Sensitivity	2-4	17

The test results reveal that the deposit is of low plasticity and hence can be categorized as a clayey silt. Natural moisture contents generally exceed the liquid limit of the soil and hence the soil has a liquidity 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 12 to 40 kPa and hence the soil can be categorized as soft to firm. A shear strength (c_u) vs. elevation (m) profile is provided on Figure 3 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 2 to 4 indicating that the soil has a low sensitivity.

Standard Penetration tests carried out in the clayey silt deposit revealed 'N' values equivalent generally to 1 blow/0.3 m confirming the weak subsoil conditions at the site.

The results (e-log p curves) of consolidation tests on representative samples of the clayey silt deposit that illustrate the compressibility characteristics of the soil are shown in Figures 4 and 5 in the Appendix. The results reveal that the deposit is underconsolidated to normally consolidated, a state of condition which indicates 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 0-14 kPa below the effective overburden pressure.

The pertinent properties of the subsoil samples tested for consolidation are provided in the Figures 4 and 5 previously mentioned. The results reveal that the initial void ratio of the soil ranges from 1.0 to 1.36 and the compression indices range from 0.1 to 0.37.

The coefficient of consolidation (cv) used to determine the time rate of consolidation settlement was computed using Taylor's Method (1948). The value ranged from 0.03 m²/day to 0.05 m²/day.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes. Measurements obtained at the time of investigation revealed levels at an elevation of 225.7 m which corresponds to a depth of 3.0 m below the existing ground surface located at the crest of the creek valley. The depth of water in the creek at the time of the investigation was shallow and approximately 1 to 1.5 m, equivalent to an elevation of approximately 226.5 to 227 m.

Groundwater levels, however, are subject to seasonal fluctuations and hence can vary throughout the season.

DISCUSSION AND RECOMMENDATIONS

It has been 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 Hwy, 638 and also the Wannamaker 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 225.5 m whilst the proposed grade elevation is 229.5 m. Consequently, a culvert roof cover of approximately 1 m and approach fills in the order of 4 m will be required.

A temporary creek diversion is planned to facilitate the new culvert construction. A plan illustrating the temporary creek diversion and the proposed new culvert is shown on Dwg. 3508701-A 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 (225.5 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 softened 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 300 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 (ϕ)	35°	30°
Unit Weight (kN/m ³)	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. (m)</u>	<u>Unit Weight* (kN/m³)</u>	<u>Shear Strength Parameters (cu) (kPa)</u>	<u>(ϕ) (°)</u>
Fill	229.5-225.5	20	-	30
Clayey Silt	225.5-219	18.2	20	-
	<219	18.2	30	-

*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 4 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 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 20 to 25 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 300 mm. It is expected that approximately 50% of the settlement is

to be realized within three 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 Wannamaker Creek primarily downstream at the culvert outlet will require excavation cuts in the order of magnitude of 4 m in the native clayey silt material. The excavated slopes shall be no steeper than 2H:1V and shall be provided with the scour protection satisfying the hydrological requirements at the site.

CONSTRUCTION CONSIDERATIONS

Temporary Diversion

To facilitate the construction of the culvert, a temporary diversion of the Wannamaker Creek has been illustrated on the E-plan (E-1371-638-1). Impervious earth dikes composed of suitable clay material (CH-see OPSS 1205) can be used upstream to prevent water inflow into the foundation excavation.

Dewatering

A dewatering scheme will be required to discharge ponded water that may be present at the proposed culvert location and also to discharge any water seepage that may develop from surface runoff. In general, little seepage is expected during the foundation excavation and construction, despite the fact that the excavation will intersect the water table, due to the relative impervious nature of the clayey silt material. However, localized silt seams interbedded in the clayey silt may bleed upon exposure. It is recommended that conventional sump pump techniques with perimeter ditches be applied to assure that the foundation construction is advanced in the dry.

Temporary Excavation Slopes

Temporary excavation slopes required in the native clayey silt deposit to facilitate the construction of the culvert should not be steeper than $1\frac{1}{2}$ H:1V for excavation cuts up to 5 m in depth. Deeper cuts should be excavated at a 2H:1V slope.

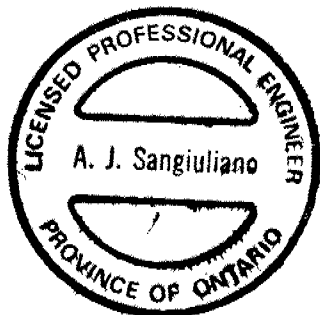
Founding Soil Preservation

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".

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

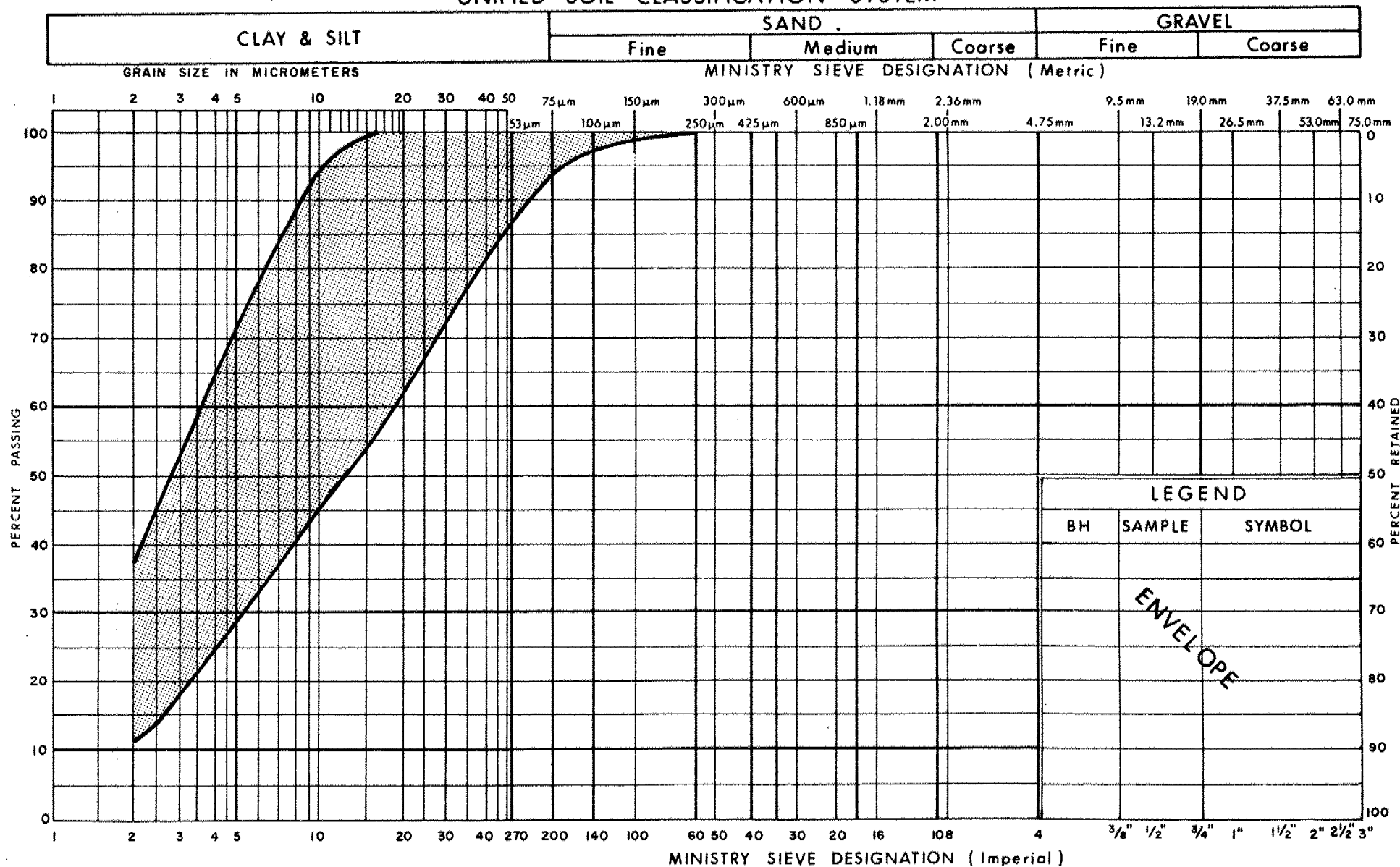
MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

UNIFIED SOIL CLASSIFICATION SYSTEM

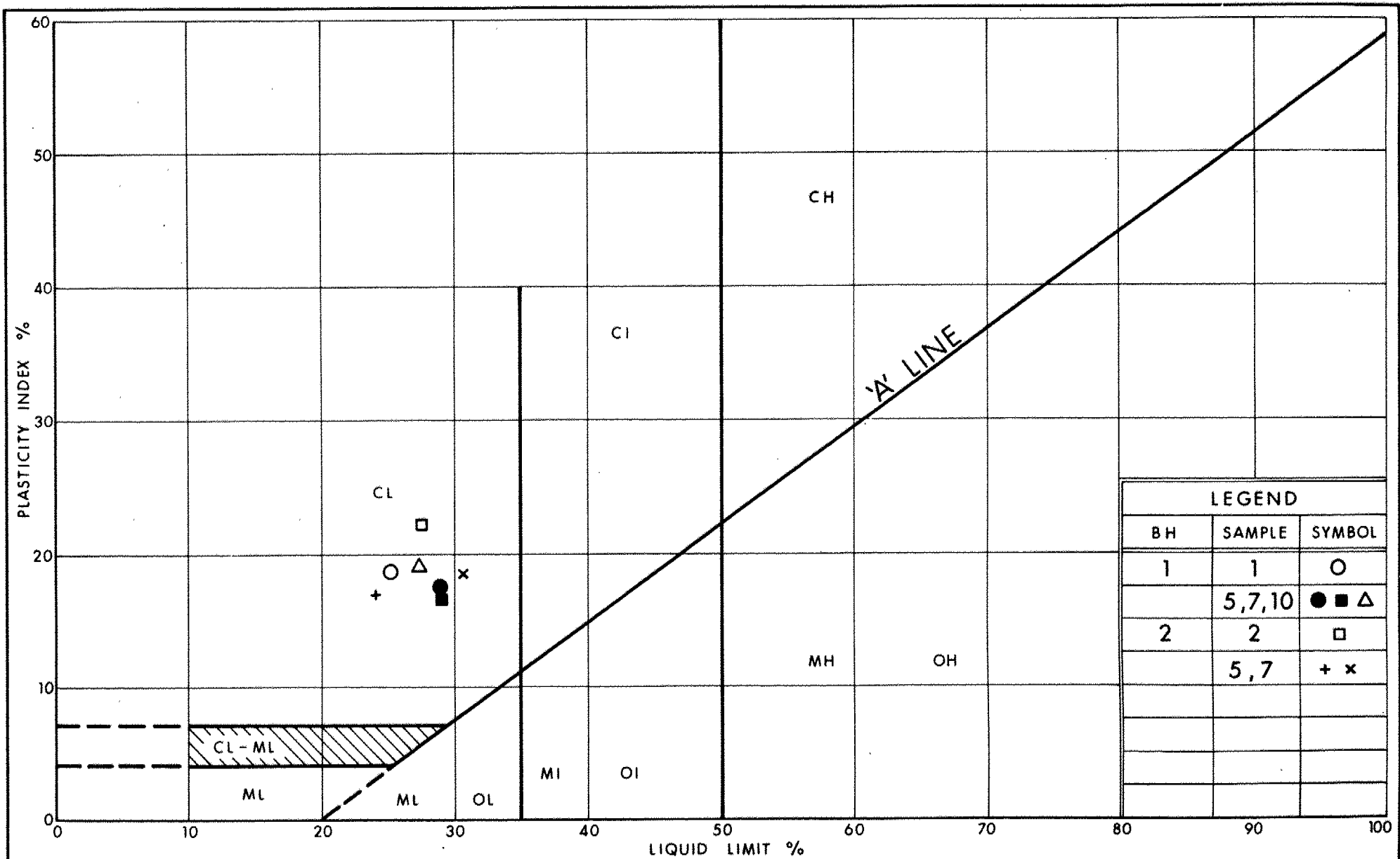


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**GRAIN SIZE DISTRIBUTION
CLAYEY SILT**
WITH RANDOM INTERBEDS OF SILT

FIG No 1

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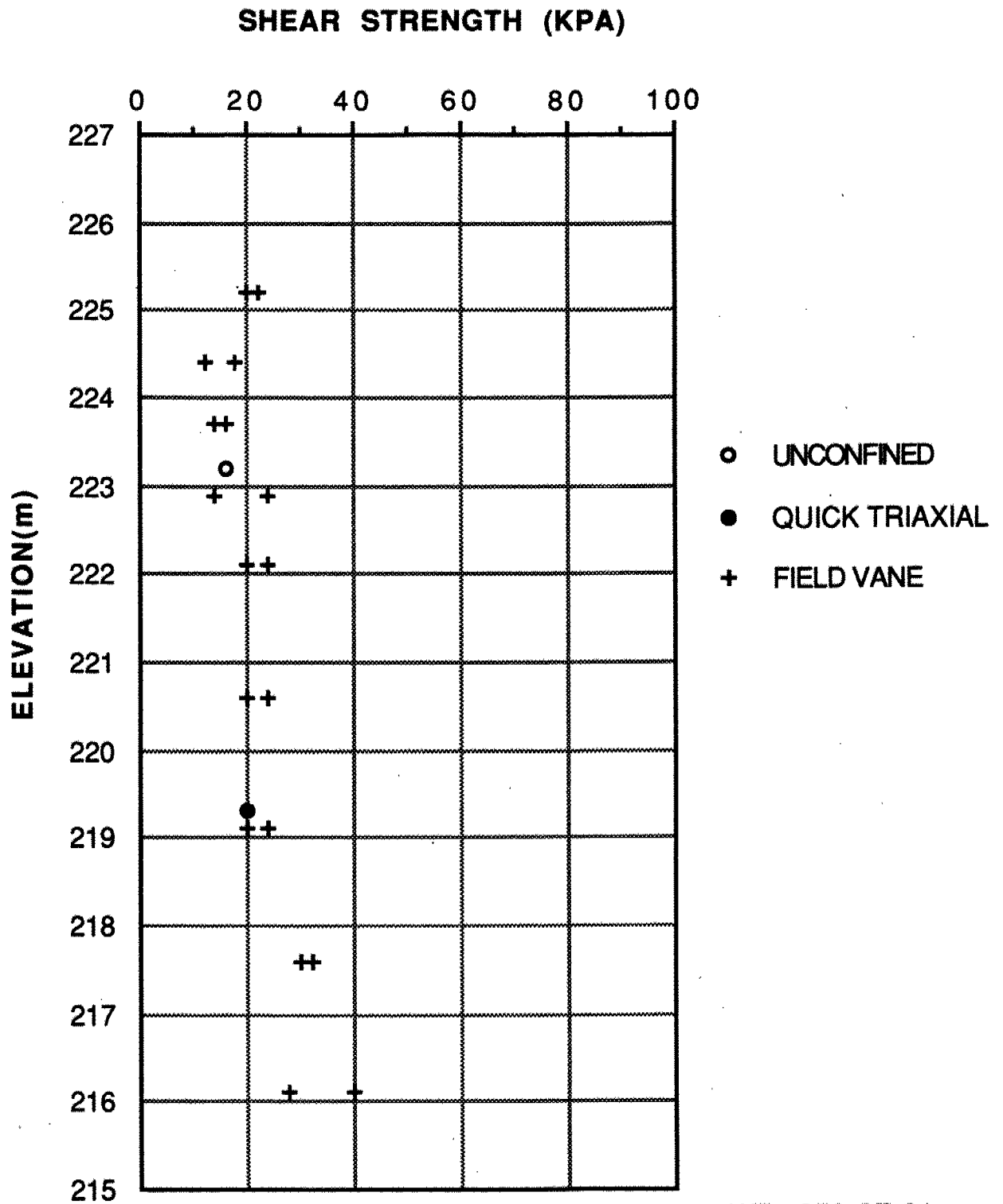
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PLASTICITY CHART
CLAYEY SILT
WITH RANDOM INTERBEDS OF SILT

FIG No 2

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FIGURE 3
SHEAR STRENGTH(CU) Vs ELEVATION



VOID RATIO - PRESSURE CURVES

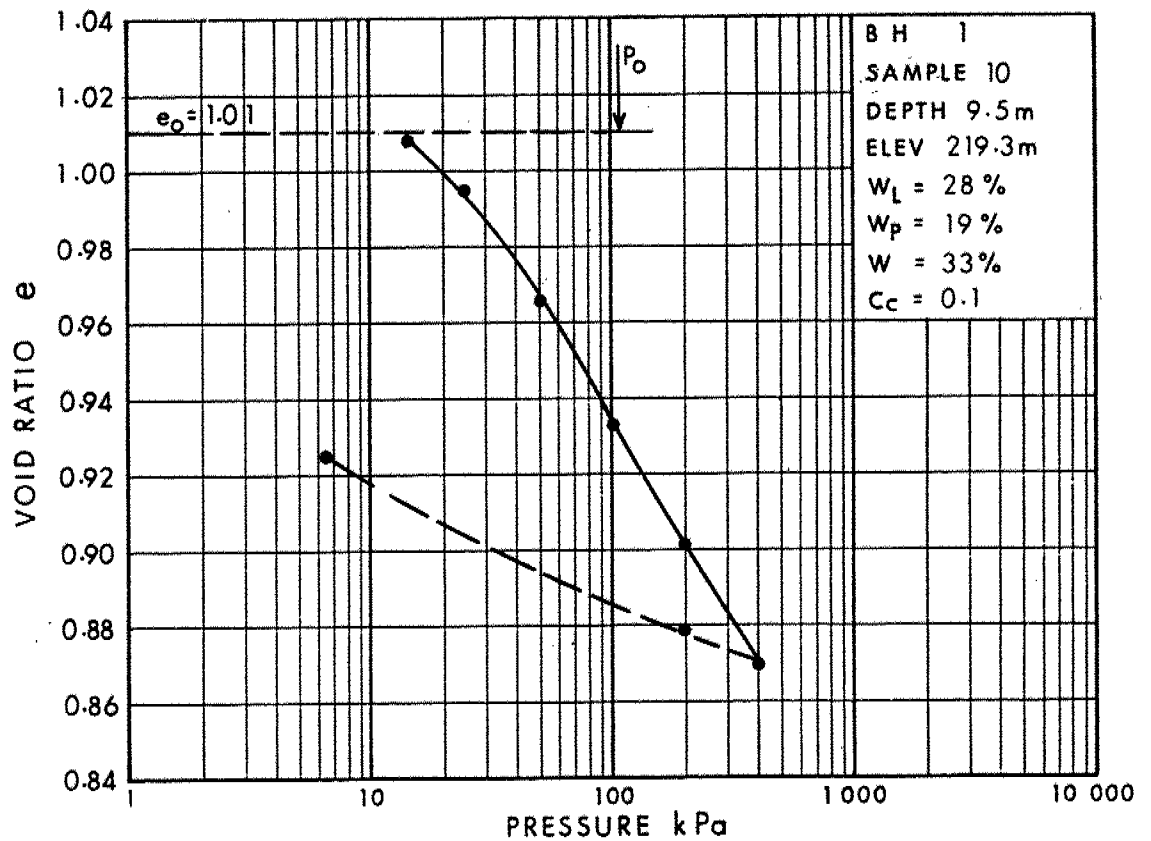
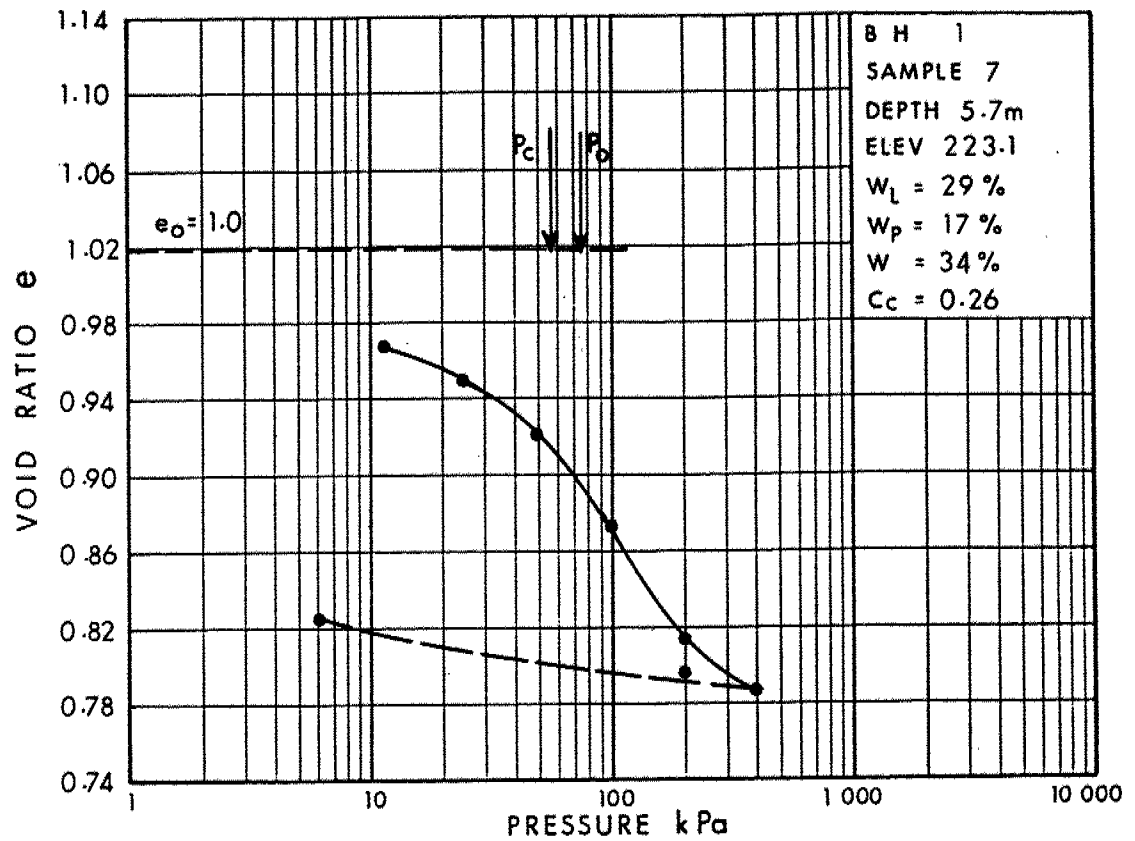


Fig 4

W P 350-87-01

VOID RATIO - PRESSURE CURVES

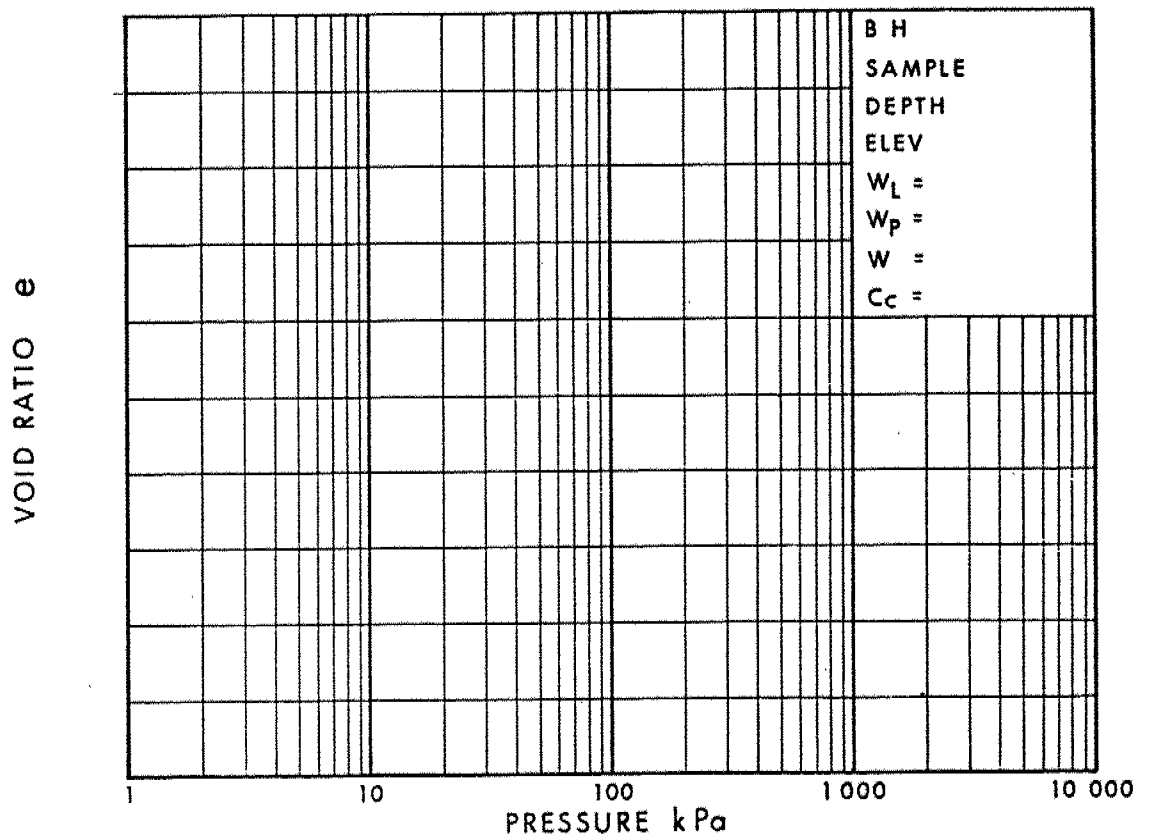
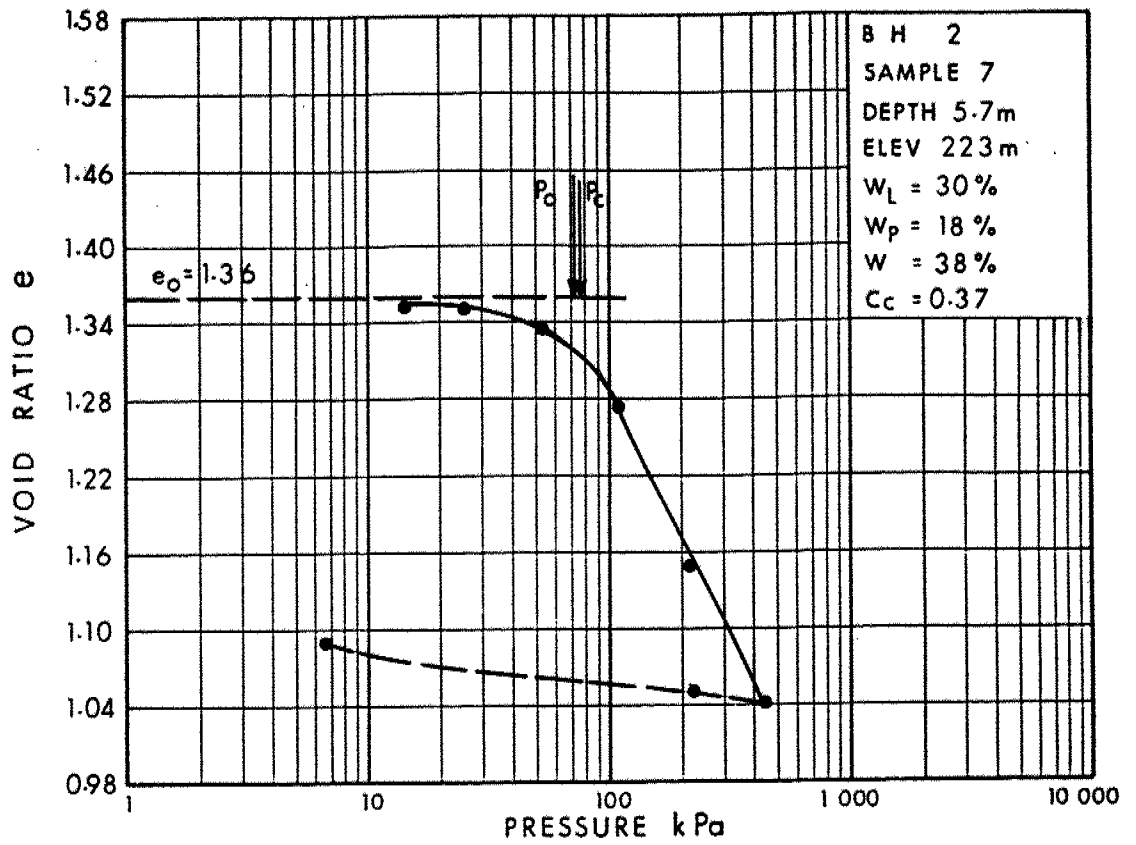


Fig 5

WP 350-87-01

METRIC

[illegible]

+3, x3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 350-87-01 LOCATION Ste 17 + 381.8: O/S 7m Lt (Line 'A') ORIGINATED BY MI
DIST 18 HWY 638 BOREHOLE TYPE HS Auger COMPILED BY TS
DATUM Geodetic DATE 90 06 07-08 CHECKED BY TS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
226.7	Ground Surface																
0.0	some sand, trace organics Brown Grey		1	CS	-												0 34 56 10
			2	SS	1												
			3	SS	1												
			4	SS	1												
			5	TW	PH											18.8	0 1 74 25
			6	SS	1												
			7	TW	PH											18.2	0 0 74 26
			8	SS	1												
			9	SS	1												
			10	TW	PH												
			11	SS	1												
			12	SS	1												
215.7																	
13.0	End of Borehole																

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

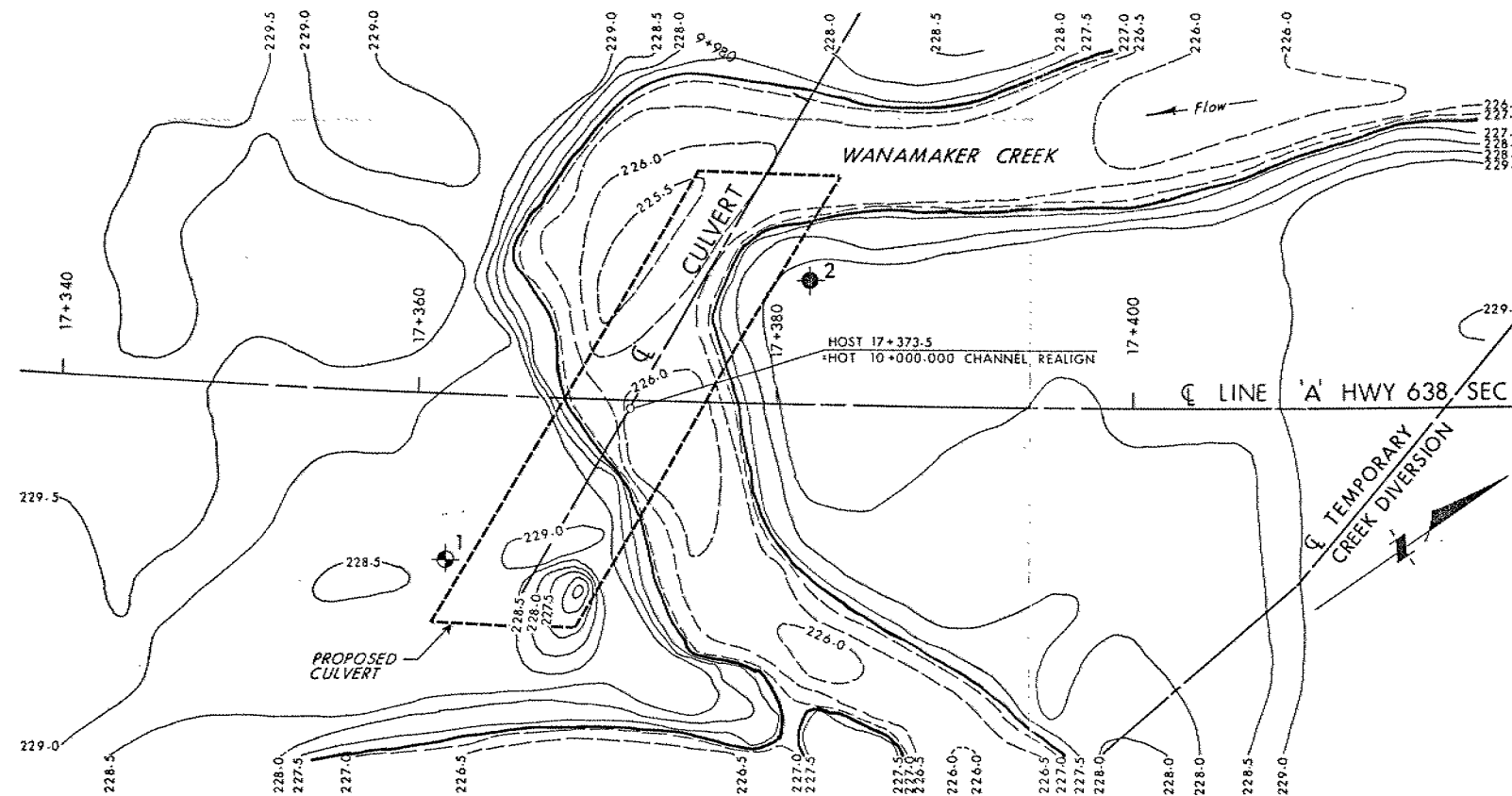
CONT No
WP No 350-87-01

WANAMAKER CREEK

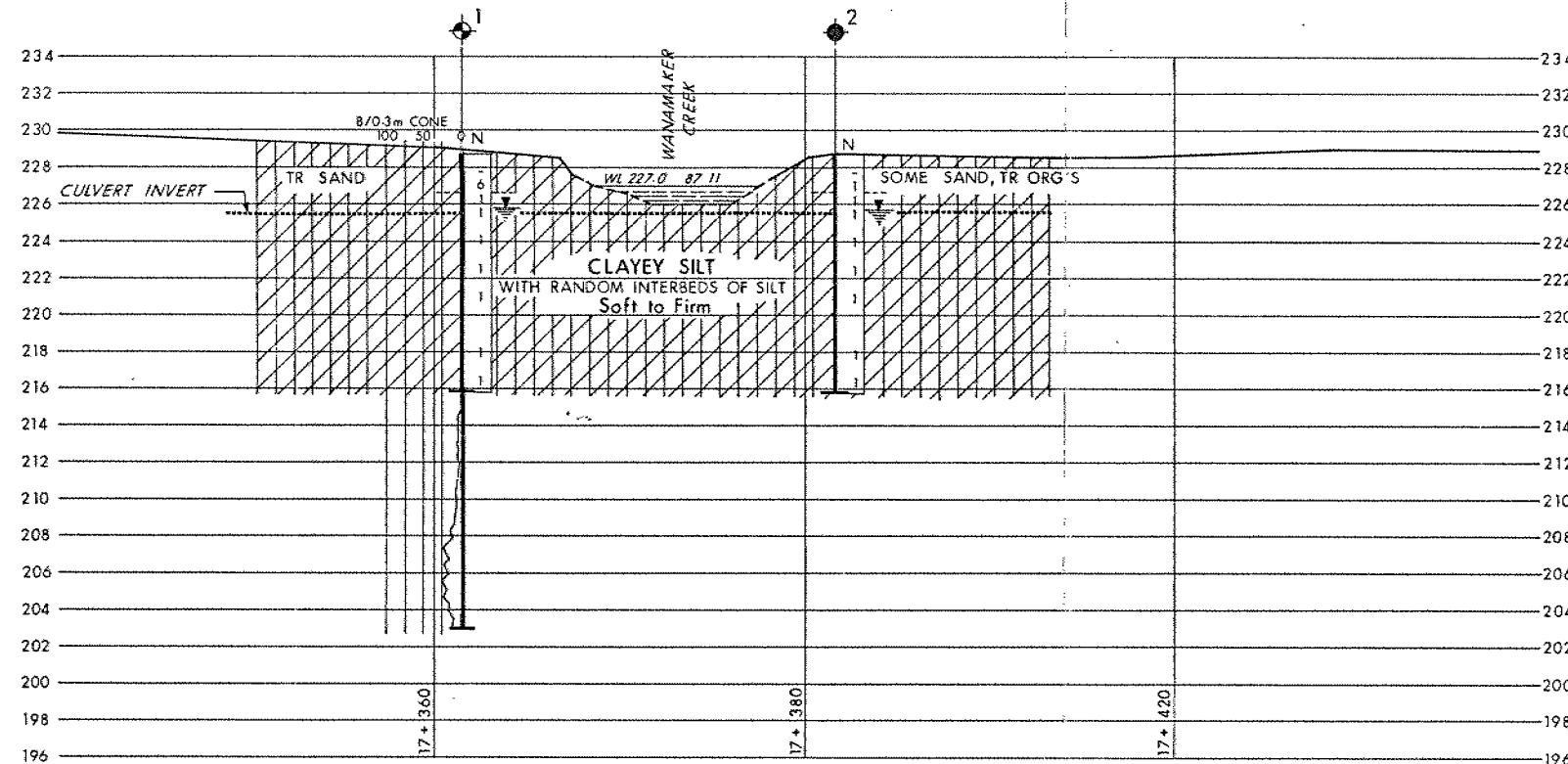
BORE HOLE LOCATIONS & SOIL STRATA



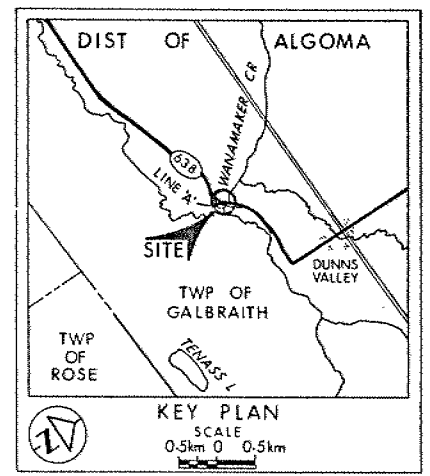
SHEET



PLAN
SCALE
4m 2 0 4m



PROFILE LINE 'A'
SCALE
4m 0 4m



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 90 06

No	ELEVATION	STATION	OFFSET
1	228.8	17+361.8	9.4m Rt
2	228.7	17+381.8	7.0m Lt

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
1	90 08 27		

Geocres No 41J-49

HWY No 638 SEC	DIST 18
SUBWD TS CHECKED	DATE 90 08 27 SITE 385-136
DRAWN DT CHECKED	DWG 3508701-A