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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



FOUNDATION INVESTIGATION REPORT

CONTRACT NO 82 - 216



Ministry of
Transportation and
Communications

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

EXPLANATION OF TERMS USED IN REPORT

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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/0.3 m)	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:

R Q D (%)	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^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION INVESTIGATION REPORT

For

Little Sturgeon River Bridge

(18.9 Km West of West Jct. of Hwy. 17B)

W.P. 145-80-01

District 13, North BayINTRODUCTION:

Geocon (1975 Ltd.) was retained by the Ministry of Transportation and Communications, Ontario (M.T.C.) to carry out a foundation investigation at the site of a proposed Bridge reconstruction at the crossing of the Little Sturgeon River approximately 25 km west of North Bay in the M.T.C. Northern Region, Ontario.

The purpose of the investigation was to obtain subsurface information for use in design and construction of foundations for the abutments and piers comprising the proposed Bridge structure and to obtain subsurface information on the alignment of the proposed temporary detour route.

The field work for this investigation was carried out in the period November 10th to December 21st, 1980. A total of ten boreholes was put down to depths ranging from 1.8 m to 54.9 m for a cumulative depth of about 158 m. In addition, six uncased dynamic cone penetration tests (pentests) were carried out, five of which were adjacent to boreholes. The majority of the pentests were terminated at a refusal of greater than 100 blows per 0.3 m except in the case of Boreholes 5 and 6 where pentests were terminated at the limit of the portable equipment used.

SITE AND GEOLOGY

The site of the proposed Bridge reconstruction is on the Little Sturgeon River approximately 25 kilometres west of North Bay on Highway 17 between North Bay and Sturgeon Falls. The existing Bridge at the site, reportedly constructed in 1937, is a three-span structure with a steel truss support for the central deck span and steel beam approach

spans. The central span between the two concrete piers is about 28 metres and the approach spans are each about 9 metres in length. The land side of the approach spans are supported on concrete abutments which in turn are carried on timber piles. The tops of the timber piles have been exposed for a short section beneath the West Abutment. No specific details are available to us regarding the foundations of the existing Bridge.

Visually, the abutment concrete has numerous cracks, some spalling and other signs of deterioration. Some rotation of the abutments has apparently taken place as indicated by a prominent crack in the asphalt pavement at each of the abutments. Differential movement has resulted in asphalt surface at the abutment which is raised between 10 to 25 mm relative to the approach span pavement surface.

The bed of the Little Sturgeon River beneath the Bridge is lined with recent deposits of sand and gravel, and there is evidence that some surficial rip rap protection was provided on the lower slopes of the approach fill embankments to below river level. The river banks are relatively steep. The banks have a brown silty topsoil and are covered by grass and small trees. On the west bank, north of the Bridge, the ground appears to have been shaped to form an approach embankment possibly for a previous bridge crossing. A car parking area extends close to the top of the river bank on the east side.

Two concrete box culverts are present at the site to the east and west of the Bridge extending beneath Highway 17 and draining northwestwards and northeastwards, respectively. The culverts drain into gulleys in the river bank, exposing silt and clayey silt soils. The easterly culvert is some distance from the Bridge and a relatively steep sided gulley is present between the culvert and the Little Sturgeon River. The westerly culvert is close to the existing Bridge and shows evidence of deterioration of the concrete and camber possibly under the weight of the highway approach embankment fill.

Available geological information indicates that the local bedrock is

Precambrian gneiss overlain by Pleistocene and recent deposits of gravel, sand, silt, boulder clay and varved clay and silt, or in larger areas, swamp accumulations. Outcrops of bedrock occur in the rapids of the Little Sturgeon River approximately one mile north of the bridge site. The bedrock becomes overlain by glaciolacustrine sediments from immediately below these rapids, and southwards to the mouth of the river where it flows into Lake Nipissing.

SUBSURFACE CONDITIONS

The stratigraphy encountered at the boreholes of this investigation is shown on the individual Records of Boreholes in the Appendix of this report, together with details of sampling and drilling and results of field and laboratory tests. A plan of the site showing borehole locations and inferred stratigraphic sections are presented on the enclosed Drawings No. 2 and 2A.

In the area of the proposed abutments, the natural soils are overlain by some 6.6 and 1.8 metres of fill on the western and eastern approaches, respectively. The fill on the western side consists of 0.6 metres of existing pavement materials underlain by 6.0 metres of silt, some clay, containing traces of sand, gravel and organics. From the available evidence,⁽³⁾ it appears that silt fill comprises backfill to the existing concrete box culvert, and that the fill probably decreases in thickness in the direction of the existing abutment. On the eastern side, the approach fill consists of a thin layer of sand and gravel pavement materials overlying natural ground.

(3) Department of Northern Development, District of Sturgeon Falls, 1936. "Plan and Profile Meadowside Bridge, Bridge Site Over Little Sturgeon River." Drawing No. B2110. Scale 1 inch = 10 feet. March 5th, 1936.

The cohesive soils encountered at the site have been divided into four separate strata on the basis of visual and tactile examination and the results of field and laboratory tests as follows:

<u>Approximate Thickness Range (m)</u>	<u>Principal Soil Types</u>	<u>Colour</u>	<u>Approximate Range of Elevation of Top of Stratum (m)</u>
1) 3 to 7.5	Silty clay to silt	Grey and Brown	196 to 200
2) 2.5 to 3.0	Silty clay	Dark Grey	191 to 193
3) 22.5 to 23.0	Silty clay, Silt - Varved	Light and Dark Grey	189 to 190
4) 13 to 14	Silty clay, Silt - Varved	Light and Dark Grey and Brown	166 to 167

The consistencies of the strata listed above down to approximately elevation 188 are generally firm, with the exception of conditions found at Borehole 2 where these strata are soft to firm. The upper varved stratum has a consistency ranging from firm to stiff and the lower varved stratum is generally stiff, ranging to very stiff.

It is judged from the results of consolidation tests on samples from the various cohesive strata and the upper part of the varved clay, that these strata are slightly overconsolidated with respect to present overburden pressures.

Underlying the cohesive soils a granular stratum was encountered at a depth of approximately 49 metres corresponding to a range of elevation from 153 to 154. It is inferred that this stratum, which consists primarily of sand, with traces of silt and gravel is glacial drift, and it is thus identified herein as a till. The till stratum, inferred to have a generally dense relative density was penetrated to depths of 6.0 and 2.5 metres.

No significant thicknesses of organic material were encountered at the four shallow boreholes put down along the proposed temporary detour route alignment. The upper one to two metres of natural silty clay to silt penetrated in these boreholes is very loose in some places.

Groundwater levels were recorded in piezometers installed in Boreholes 1 and 2, a standpipe in Borehole 1, and in the open borehole in the case of Borehole 4. The most recent observations (taken on December 21st, 1980) are shown on the Records of Boreholes and Drawing No. 2.

Considering the relatively short time period between installation and observations, it is not certain whether the water levels as shown represent stabilized levels.

DETAILED SUBSURFACE INFORMATION

The individual strata encountered at the boreholes are described in the following sections.

Topsoil (Pt)

A surficial layer of topsoil was penetrated in three of the four boreholes located on the proposed detour route alignment. The topsoil was mainly silt mixed with humus and other vegetation debris and ranged in thickness from 75 to 100 mm. Beneath the west approach embankment, traces of inferred original topsoil were observed at the junction of the fill and natural soils.

Fill

The boreholes at the proposed abutment sites were put down through the edge and shoulder of the existing Highway 17 pavement. At the proposed west abutment, 150 mm of asphalt and 450 mm of sand and gravel were penetrated while at the east abutment, 2.8 m of sand and gravel was penetrated on the shoulder of the highway.

Underlying the existing pavement materials in the area of the proposed west abutment, some 6.0 metres of fill was encountered. The fill materials are predominantly silt, some clay mixed with traces of sand, gravel and traces of organics (topsoil and occasional wood fragments).

Water content determinations from samples of the silty fill ranged from about 19 to 30 percent.

The two shallow boreholes on the proposed detour alignment to the west of the Little Sturgeon River encountered a surficial fill layer of 0.7 m and 1.3 m thickness of silty sand, trace to some gravel and trace of organics. The eastern most borehole along the detour alignment, located at the edge of Highway 17 on the north shoulder, encountered 0.8 m of sand and gravel fill. Water content determinations in the silty sand fill ranged from 15 to 18 percent.

Standard Penetration Tests carried out within existing approach embankments gave 'N' values ranging from 21 to 5, indicating a compact to loose relative density. 'N' values ranging from 7 to 15, were obtained in the silty sand and sand and gravel fill encountered elsewhere. A 0.3 m surface layer of sand and gravel was encountered at Borehole 2 adjacent to the Little Sturgeon River and has been included in this section although this granular material is likely a recent deposit laid by the river and not fill.

Silty Clay (CL-ML to CI)

A stratum of silty clay of low plasticity was encountered underlying the embankment fills, and at ground surface at the other boreholes. Where fully penetrated, the stratum ranges in thickness from about 3.0 to 7.5 metres at the proposed Bridge site and some 6.0 to 0.7 metres at the proposed Bailey Bridge site. The soil has a high silt content and is slightly cohesive. The silty clay is mainly grey in colour with brown areas (possibly due to oxidation), has occasional very thin laminae of

grey silt and/or fine sand, and a few pockets or lenses of grey silt. The majority of the recovered samples contain a relatively uniform trace of organics.

Atterberg limit tests carried out on six selected samples gave Liquid Limits ranging from 27 to 42 percent, Plastic Limit 22 to 31 percent for resultant Plasticity Indices of 4 to 15.5. Natural moisture contents of these samples ranged from 29 to 35 percent, respectively. The Plasticity Chart, (Figure 1 in the Appendix) shows the above results, indicating silt and clay of low to medium plasticity grading to organic silt.

Three grain size distribution tests were carried out on the same samples as selected for the Atterberg limit tests and the resultant curves are shown on Figure 2 in Appendix. The samples as tested contain a trace of sand, 77 to 87 percent silt sizes and 12 to 15 percent clay sized particles.

Wet unit weights determined in the course of the triaxial and consolidation tests were both close to 17.9 kN/m^3 .

One unconsolidated undrained triaxial test was carried out on a sample recovered from the silty clay stratum and gave an undrained shear strength, taken as one half the compressive strength, of 12 kPa.

In-situ shear strengths, determined by field vane ranged from about 32 kPa to 80 kPa. Based on the natural and remoulded shear strength values obtained by field vane, the sensitivity is in the range of 2 to 4. Shear strengths were also measured using a Geonor miniature laboratory vane in the bottom of the thin walled steel tube samplers at the time of sample recovery. The shear strengths as measured by laboratory vane ranged from about 30 kPa to 55 kPa. The sensitivity of the stratum, from natural and remoulded shear strength values using the laboratory vane, ranged from 4 to 7. There is a significant variation in undrained shear strengths as obtained by the two types of vane apparatus, and the one triaxial test. On the available evidence, the stratum is inferred to be generally of firm consistency.

One consolidation test was carried out on a sample from the silty clay stratum, and the results are presented as a pressure/void ratio curve on Figure 6 in Appendix. The computed Compression Index (C_c) is 0.25.

Silty Clay (CI)

Underlying the silty clay of low plasticity a stratum of silty clay of intermediate plasticity was encountered in the five deeper boreholes of the investigation.

Where fully penetrated, the stratum ranged in thickness from 2.5 m to 3.0 m. The silty clay is dark grey in colour and several samples showed discontinuous small lenses or pockets of light grey silt dispersed through the dark grey silty clay. Thin horizontal laminae of grey silt to fine sand were occasionally present.

Atterberg limit tests were carried out on six selected samples, two from siltier portions and four from silty clay portions. The samples with higher silt content had a Liquid Limit of 24 percent, Plastic Limits of 18 percent to 20 percent for resultant Plasticity Indices of 4 and 6. Reference to the Plasticity Chart, Figure 1, in the Appendix indicates for Sample 9, Borehole 1 and Sample 5, Borehole 2, a low plasticity inorganic silty clay (CL-ML). Natural water content of these samples was 34 percent and 44 percent, respectively. In the case of the samples with higher clay content, Liquid Limits ranged from 37 to 44 percent, Plastic Limits of 20 percent to 22 percent, for resultant Plasticity Indices of 16 to 24. These results as shown on the Plasticity Chart indicate an inorganic silty clay (CI). Natural water contents of these samples ranged from 32 to 42 percent, respectively.

Grain size distribution tests were carried out on two of the samples selected for Atterberg Limit tests and the resultant curves are shown on Figure 3 in Appendix. The grain size distributions are closely similar, and the samples as tested contain 2 to 8 percent sand, 67 to 75 percent

silt sizes and 23 to 25 percent clay sized particles.

Wet unit weights determined in the course of triaxial and consolidation tests carried out on samples from the silty clay stratum ranged from 16.3 kN/m^3 to 18.5 kN/m^3 .

Three unconsolidated undrained triaxial tests carried out on selected samples of the silty clay stratum gave shear strengths (taken as one half the compressive strength) ranging from 20 kPa to 43 kPa. In order to investigate possible sample disturbance, two triaxial tests were performed on different portions of a single thin walled sample from Borehole 4. The shear strength of the sample from the end of the tube was 23 kPa and the sample from the middle of the tube was 20 kPa.

In-situ undrained shear strengths, determined by field vane generally ranged from 30 kPa to 55 kPa. Based on a comparison of the natural and remoulded shear strengths obtained, the field vane results indicate a sensitivity in the range of 2 to 4. Shear strengths determined on site using a Geonor miniature laboratory vane range from 24 kPa to 40 kPa. The sensitivity of the silty clay stratum based on laboratory vane values generally ranged from 6 to 10. On the evidence of measured individual shear strengths, the consistency of the silty clay stratum ranges from firm to stiff, being generally firm.

Three consolidation tests were carried out on selected samples from the silty clay stratum, and the results are presented on Figures 7 and 8 in the Appendix. The Compression Indices (C_c) from these tests were from 0.27, 0.44 and 0.84.

Clay (CH), Silty Clay (CL) and Silt (ML) Varved (Upper Stratum)

Underlying the silty clay in the two deeper boreholes and the two boreholes adjacent to the river on the west side, a varved clay stratum was encountered. The stratum was fully penetrated in the two deeper boreholes for a thickness of between 22.5 and 23.0 metres, corresponding to an approximate elevation range of 190 to 167. The structure exhibited

by samples consists of repeated layers of dark grey clay and light grey silty clay and silt, with transitions observed with each varve unit as the relative contents of clay and silt change from silt to a high plasticity clay. Generally, individual layers range from about 8 mm up to 30 mm in thickness. A similar structure was exhibited below elevation 167 but on the basis of strength results and visual examination two distinct varved clay strata are distinguished. The varved clay to about elevation 167 is referred to as the upper varved clay.

Atterberg Limit tests were carried out on a number of selected samples after visual separation of the individual layers. The results of these tests are shown on the Plasticity Chart, Figure 1 and on the Records of Boreholes in the Appendix. Typically, for the dark grey clay layers, Liquid Limits ranged from 60 to 77 percent, Plastic Limits 24 to 39 percent for resultant Plasticity Indices of 29 to 44, whereas for the siltier layers, Liquid Limits ranged from 27 to 40 percent, Plastic Limits from 16 to 25 percent for resultant Plasticity Indices from 7 to 16.

Grain size distribution tests were carried out on three samples which were selected for Atterberg Limit tests. The grain size tests were carried out on separated layers as shown on Figure 4, in Appendix. The finer layers tested contained 47 and 23 percent silt and 53 and 77 percent clay sized particles. The coarser layers contained between 68 and 45 percent silt and between 32 and 55 percent clay sized particles.

Wet unit weights determined in the course of the triaxial and consolidation tests ranged from 16.0 kN/m^3 to 18.5 kN/m^3 . Unit weights in each case were determined prior to testing.

Six unconsolidated undrained triaxial tests carried out on samples recovered from this upper varved stratum gave shear strengths ranging from 30 kPa to 50 kPa. Two tests were performed on separate sections of one thin walled steel tube sample. The shear strengths were 41 kPa, for the portion of the sample close to the centre of the tube and 30 kPa for the portion close to the end of the tube.

In-situ undrained shear strengths determined by field vane ranged from 35 kPa to 125 kPa. Based on the natural and remoulded shear strength values the field vane results indicate a sensitivity in the range of 2 to 5. Undrained shear strengths as measured on site by laboratory vane ranged from 21 kPa to 75 kPa. The sensitivity of the varved stratum using a laboratory vane ranged from 6 to 11. On the evidence of shear strength values, the consistency of the upper varved stratum was generally firm to stiff.

Three consolidation tests were carried out on selected samples of the upper varved stratum and the results of these tests are shown on Figures 9 and 10 in the Appendix. For each test the samples tested included both layers horizontal. Generally the samples tested included both layers of silt and clay sized particles. The computed Compression Indices (C_c) from these tests are 0.72, 1.31 and 0.96.

Clay (CH), Silty Clay (CL) and Silt (ML), Varved (Lower Stratum)

The upper varved stratum is underlain by a lower varved stratum, characterized by the presence of a brown to reddish brown clay layer interbedded with the light grey silt and dark grey silty clay. The repeated varve units show evidence of transition from silt through silty clay. Generally, the recovered samples show individual layers in this stratum ranging in thickness from some 7 mm up to about 12 mm.

Atterberg Limit tests were carried out on separated varve layers from one selected sample and these results are shown on the Plasticity Chart, Figure 1 in the Appendix. For the dark grey clay layer the Liquid and Plastic Limits determined were 91 percent and 32 percent, for a brown clay layer 73 percent and 23 percent, and for a siltier layer 28 percent and 20 percent. One additional Atterberg Limit test carried out on a sample recovered from close to the base of the stratum gave a Liquid Limit of 62 percent, and Plastic Limit of 26 percent.

Two unconsolidated undrained triaxial tests were carried out on

samples from the lower varved stratum. The undrained shear strengths taken as one half the compressive strength averaged about 40 kPa.

Wet unit weights determined in the course of the triaxial tests were 17.6 kN/m³ and 17.8 kN/m³.

In-situ undrained shear strengths were determined by field vane and ranged from 135 kPa to 240 kPa. Based on the natural and remoulded shear strength values obtained using a field vane, the sensitivity of the varved stratum ranges from 2 to 9. The shear strengths as measured by laboratory vane ranged from 55 kPa to 90 kPa. The sensitivity based on strength values using a laboratory vane ranged from 2 to 4. On the evidence of shear strength values the consistency of the lower varved stratum generally ranges from stiff to very stiff.

Sand, Trace to Some Gravel, Silt and Clay (Till)

Underlying the lower varved stratum, a granular stratum was encountered in the two deeper boreholes. The top of the stratum was found between elevations 153 and 154. The recovered samples consisted of gravel, silt and clay. Visually, the granular soil has no structure. It is considered to be glacial drift, and has thus been classified herein as till.

Grain size distribution tests were carried out on standard split spoon samples from the granular stratum and the results of three tests are presented as an envelope on Figure 5 in the Appendix. The results indicate between 6 and 13 percent gravel, 52 and 70 percent sand, 13 and 31 percent silt and 4 and 6 percent clay sized particles. The results do not reflect the content of sizes greater than about 37 mm. On geological evidence, larger sizes could be expected to be encountered within the formation, possibly up to and including boulder sizes.

Standard Penetration Tests carried out within the till stratum gave 'N' values ranging from 60 to greater than 100. The 'N' values infer a very dense relative density, although it should be noted that the measured values could have been influenced by coarse sizes in the till, and that they reflect the significant depth of drilling.

Groundwater Conditions

For purposes of monitoring groundwater levels across the site one piezometer was sealed within the till at Borehole 1 and one piezometer in the area of the proposed West Pier at Borehole 2. In addition a standpipe was installed in Borehole 1 and the water level observed in the open hole subsequent to drilling in Borehole 4. The latest groundwater levels observed, taken on 21st December, 1980, are shown on the Records of Boreholes Sheets and the enclosed Drawings 2 and 2A.

The water level in the Little Sturgeon River at the time of the investigation was measured at elevation 194.7, as top of ice at Pentest 3 on December 19th, 1980.

Groundwater levels in the existing approach embankment fills ranged between approximate elevations 198 and 200, or some 2 to 4 metres above the water level in the Little Sturgeon River at the time. At the proposed West Pier location, the groundwater level was measured some 3 metres below ground surface, which correspond to about 2.0 metres below river level at the time of drilling.

Since the fill and natural soils at the site are fine-grained and of relatively low permeability, the measured groundwater levels may not have stabilized over the period of the readings.



T. J. Kazmierowski, P. Eng.
Foundations Engineer



M. Devata, P. Eng.
Senior Foundations Engineer

RECORD OF BOREHOLE No 1

METRIC

16

W P 145-80-01 LOCATION STA. 10+812.8 o/s 3.9m RT. of HWY. 17 ORIGINATED BY M.H.
DIST 13 HWY 17 BOREHOLE TYPE CME 55 TRUCK MOUNTED DRILL RIG - HOLLOW STEM AUGER - COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 11 12 TO 1980 12 01 BW CASING CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80					
202.6	ROAD LEVEL							20	40	60	80	20	40	60	kN/m ³	GR SA SI CL
202.0	Sand and Gravel 45m		2	SS	21		WL	80	12	21						
0.6	Fill - Silt, Some Clay. Trace Sand, Gravel and Organics Loose to Compact Grey and Brown		3	SS	5		WL	80	12	21						
			4	SS	6											
			5	TW	PH											
196.0			6	TW	PH											
6.6	Clayey Silt Occasional lamina of silt, trace organics.		7	TW	PH											
192.8	Firm Grey and Brown		8	TW	PH											
9.8	Silty clay, occasional pockets or lamina of silt and fine sand		9	TW	PH											
189.8	Firm Dark Grey		10	TW	PH											
12.8	Silty clay, clayey silt and silt-varved Firm to Stiff Light and Dark Grey Layers		11	TW	PH											
			12	TW	PH											
			13	TW	PH											
			14	TW	PH											
			15	TW	PH											
			16	TW	PH											
			17	TW	PH											
			18	TW	PH											
			19	TW	PH											
			20	TW	PH											
			21	TW	PH											
			22	TW	PH											
			23	TW	PH											
			24	TW	PH											
167.2	Silty clay, clayey silt and silt-varved Stiff to Very Stiff Light Grey, Dark Grey and Brown		25	TW	PH											
35.4			26	TW	PH											
			27	TW	PH											
			28	TW	PH											
153.1			29	TW	PH											
49.5	Sand, trace gravel (Till)		30	SS	79											
150.6			31	SS	220/0.15m											
52.0	END OF BOREHOLE (a) Asphalt 150															

Note: Pentest location 15 metres
east of Borehole

SEE FIGURES
FOR
CONSOLIDATION
TEST
RESULTS

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



METRIC

17

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE STA. 10+847.1 o/s 3.2m RT. of HWY. 17 ORIGINATED BY G.F.P.
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING, BW CASING COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 12 12 to 1980 12 18 CHECKED BY R.C.S.

[illegible]

+3, x5: Numbers refer to Sensitivity

20
15 \odot
10



METRIC

18

CHECKED BY R.C.S.

[illegible]

Note: Increase in penetration resistance at approximately El. 182.
overnight delay.

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 4

METRIC

19

W P 145-80-01

LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+878.1 o/s LT. 19.6m @ HWY. 17

ORIGINATED BY G.F.P.

DIST 13 HWY 17

BOREHOLE TYPE CME 55 BOMBARDIER MOUNTED DRILL RIG - HOLLOW STEM

COMPILED BY M.C.Z.

DATUM GEODETIC

DATE 1980 12 06 TO 1980 12 09

AUGER - BW CASING

CHECKED BY R.C.S.

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	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	50 100 150 200 250					
202.5	GROUND LEVEL												
199.7	Fill - sand and gravel Brown	1	SS	5		200	80	12 21		40			
2.8		2	TW	PH		195				0			
	Silt to Clayey Silt. Occasional lamina of silt and fine sand, trace of organics	3	TW	PM						0			
192.2	Firm Brown and Grey	4	TW	PM						0			
10.3	Silty clay, occasional pockets and thin lamina of silt and fine sand	5	TW	PM		190				0			
189.4	Firm Dark Grey	6	TW	PH						0			
13.1		7	TW	PM						0			
	Silty clay, clayey silt and silt - varved	8	TW	PM						0			
	Firm to Stiff	9ab	SS	-		185				0			
		10	TW	PH						0			
		11	TW	PH						0			
	Light and Dark Grey Layers	12	TW	PH		180				0			
		13	TW	PH						0			
		14	TW	PH						0			
		15	TW	PH		175				0			
166.5										0			
36.0	Silty clay, clayey silt and silt - varved	16	TW	PH		165				0			
	Stiff									0			
	Light Grey, Dark Grey and Brown	17	TW	PH		160				0			
		18	TW	PH						0			
153.7		19	SS	100/0.02m		155				0			
48.8	Sand, some silt, traces gravel and clay (Till)	20	SS	51						0			
		21	SS	90						0			
		22	SS	50						0			
		23	SS	75						0			
		24	SS	-		150				0			
147.6										0			
54.9	END OF BOREHOLE									0			
Note: 1. Increase in penetration resistance at approximately El. 187. Overnight delay. 2. Perforation location 2 metres east of Borehole.													SEE FIGURES FOR CONSOLIDATION TEST RESULTS

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 5

METRIC 20

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING
DATUM GEODETIC DATE 1980 11 20

ORIGINATED BY M.H.
COMPILED BY M.C.Z.
CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
198.3	GROUND LEVEL																
0.0	Clayey Silt. trace organics		1	TW	PM		195	x4	5					0			CONSOLIDATION TEST 0 4 83 13
	Firm to Stiff		2	TW	PM			x4		2				0			
	Grey and Brown		3	TW	PM			x4						0			
			4	TW	PM			x6						0			
			5	TW	PM			x4						0			
191.1			6	TW	PM			x7	4					0			
7.2	Silty clay		7	TW	PM		190	x10	23	blows/0.3m				0			
187.8	Firm to Stiff Dark Grey		8	TW	PM			x8		3							
10.5	END OF BOREHOLE																SEE FIGURES FOR CONSOLIDATION TEST RESULTS
Note: Pentest location 2 metres east of Borehole.																	

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 6

METRIC 21

W P 145-80-01 LOCATION STA. 10+832.2 o/s 17.0m LT. & HWY. 17
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING
DATUM GEODETIC DATE 1980 11 11 to 1980 11 13
ORIGINATED BY M.H.
COMPILED BY M.C.Z.
CHECKED BY R.C.S.

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					
197.7	GROUND LEVEL																
0.0	Clayey Silt. Firm to Stiff		1	TW	PM		195	4	6	3			0				
	Grey		2	TW	PM			6	4				0				
			3	TW	PM			3									
			4	TW	PM			4									
191.5	Silty clay		5	TW	PM			4					10-1				
6.2	Firm		6	TW	PM		190	4								17.8	CONSOLIDATION TEST
188.8	Dark Grey		7	TW	PM			4								17.8	0 2 75 23
8.9	(a)		8	TW	PM			6					0				
187.5	END OF BOREHOLE																
10.2	(a) Silty clay, clayey silt and silt - varved Firm Light and Dark Grey Layers																SEE FIGURES FOR CONSOLIDATION TEST RESULTS

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 8

METRIC 22

W P 145-80-01 LOCATION STA. 10+782.0 o/s 8.0m LT. & HWY. 17 ORIGINATED BY M.H.
 DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
 DATUM GEODETIC DATE 1980 11 14 CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	Wp	W	Wl		
202.4	(a) GROUND LEVEL		1	SS	1.0												
0.0	(b)		2	SS	1.0												
1.3	(c)		3	SS	1.0												
2.4	END OF BOREHOLE																
	(a) Topsoil 150																
	(b) Fill - silty sand and gravel, trace organics Loose to Compact Brown																
	(c) Silt Very Loose Brown to Grey																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 9

METRIC 23

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+800.7 o/s 10.6m LT. & HWY. 17
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING
DATUM GEODETIC DATE 1980 11 14
ORIGINATED BY M.H.
COMPILED BY M.C.Z.
CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
201.9	(a) GROUND LEVEL																GR SA SI CL
0.0	(b)																
0.7	Sandy silt Very Loose Grey		1	2	7		200										
2.4	END OF BOREHOLE																
	(a) Topsoil 100.																
	(b) Fill - silty sand, trace gravel and organics																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 10

METRIC 24

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE ORIGINATED BY M.H.
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 11 14 CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
199.9	(a) GROUND LEVEL		1	SS	3.3								
0.0	Clayey silt to silt		2	SS	3.3								
197.5	Very Loose Brown		3	SS	3.3								
2.4	END OF BOREHOLE		4	SS	3.3								
	(a) Topsoil 75												

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15 - 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 11

METRIC 25

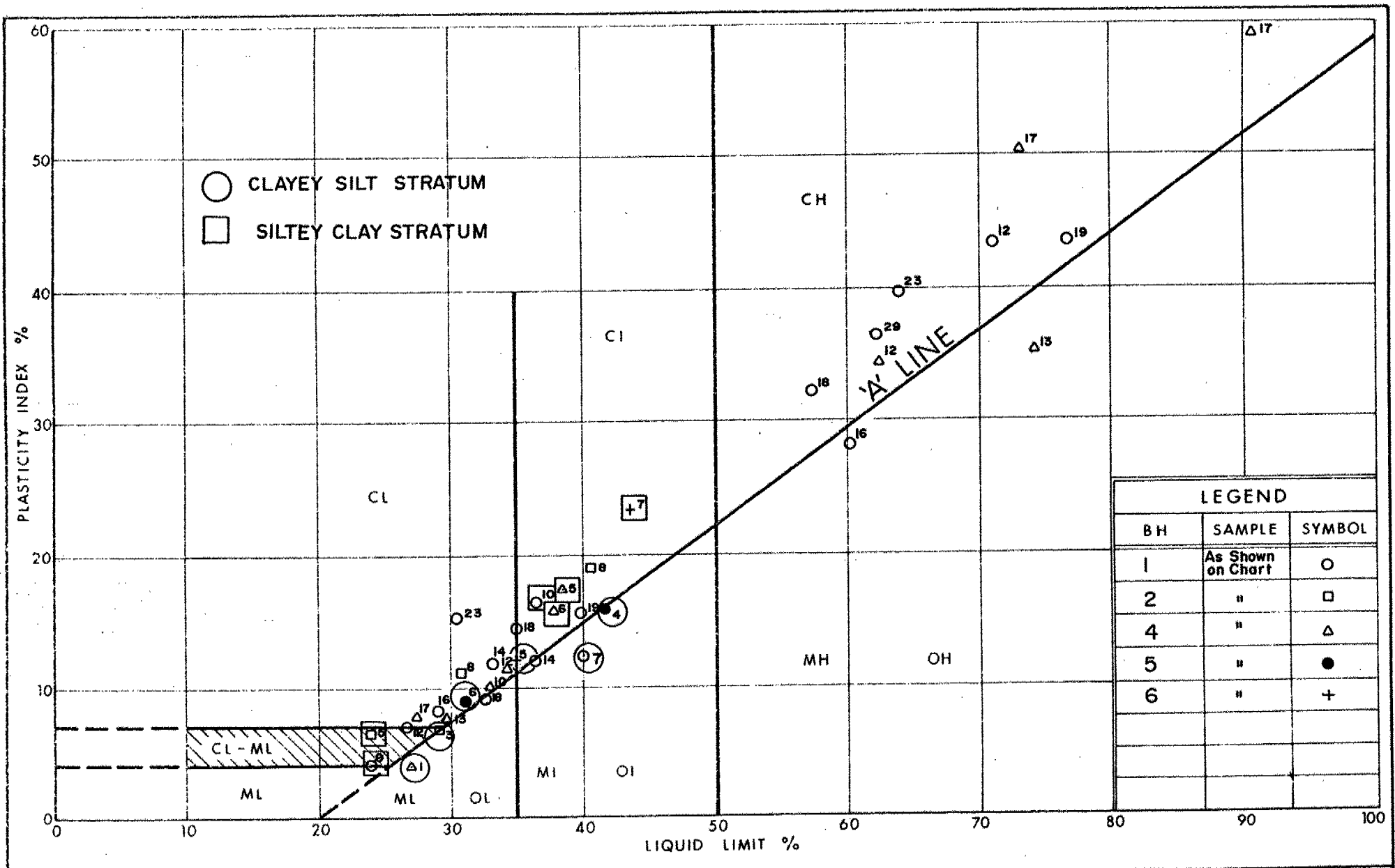
W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+943.6 o/s 9.1m LT. & HWY. 17 ORIGINATED BY M.H.
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 11 14 CHECKED BY R.C.S.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (EST)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80					
200.0	GROUND LEVEL															
0.0	(a)															
0.8	(b)															
1.8	END OF BOREHOLE															
	(a) Fill - sand and gravel Loose Brown															
	(b) Silt Compact Brown															

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10



Ontario

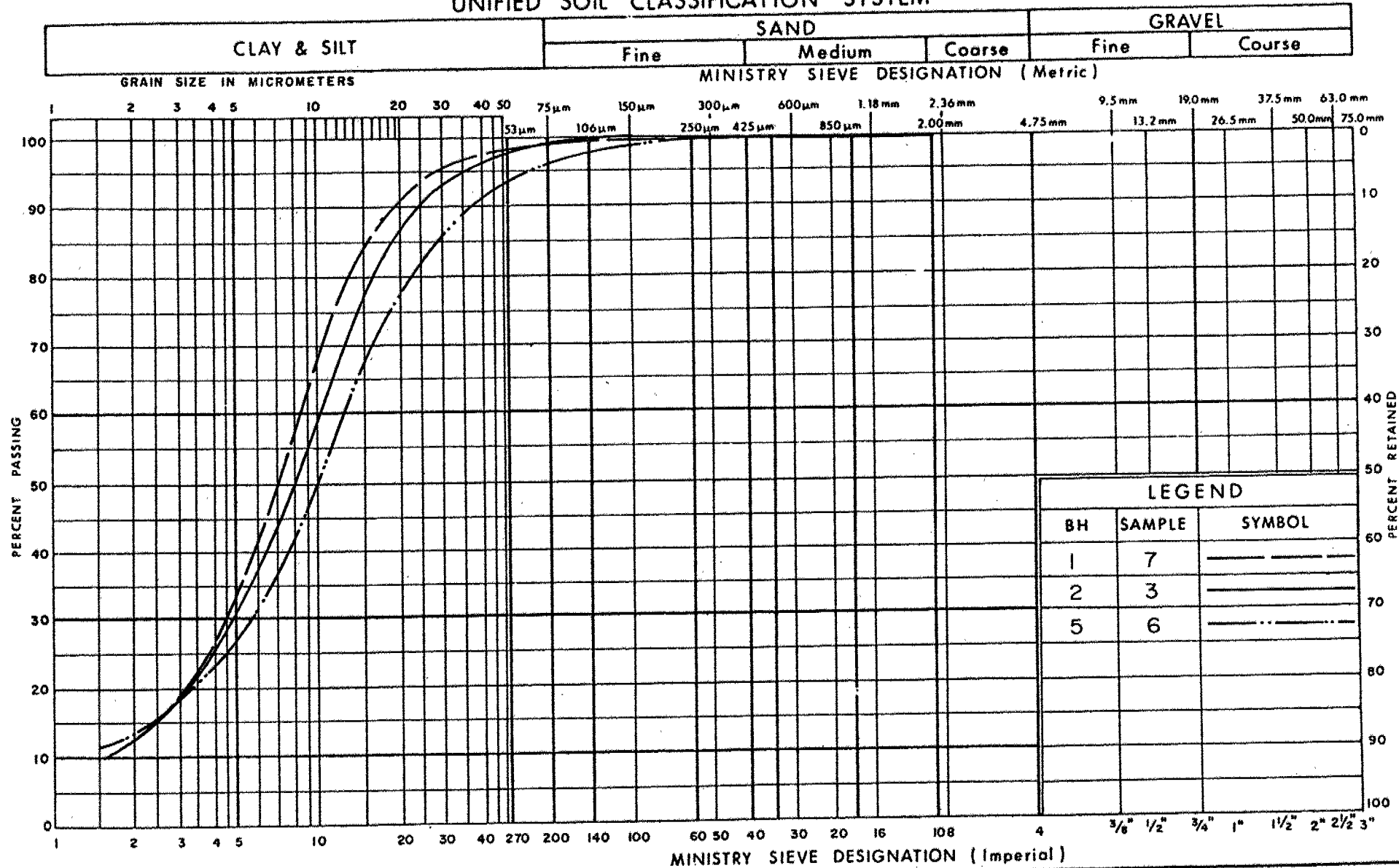
Ministry of
Transportation and
Communications

PLASTICITY CHART
 COMBINED TEST RESULTS ON SAMPLES OF COHESIVE STRATA
 NOTE Where sample numbers are duplicated, tests carried out on
 separated varve layers.

FIG No 1

W P 145-80-01

UNIFIED SOIL CLASSIFICATION SYSTEM



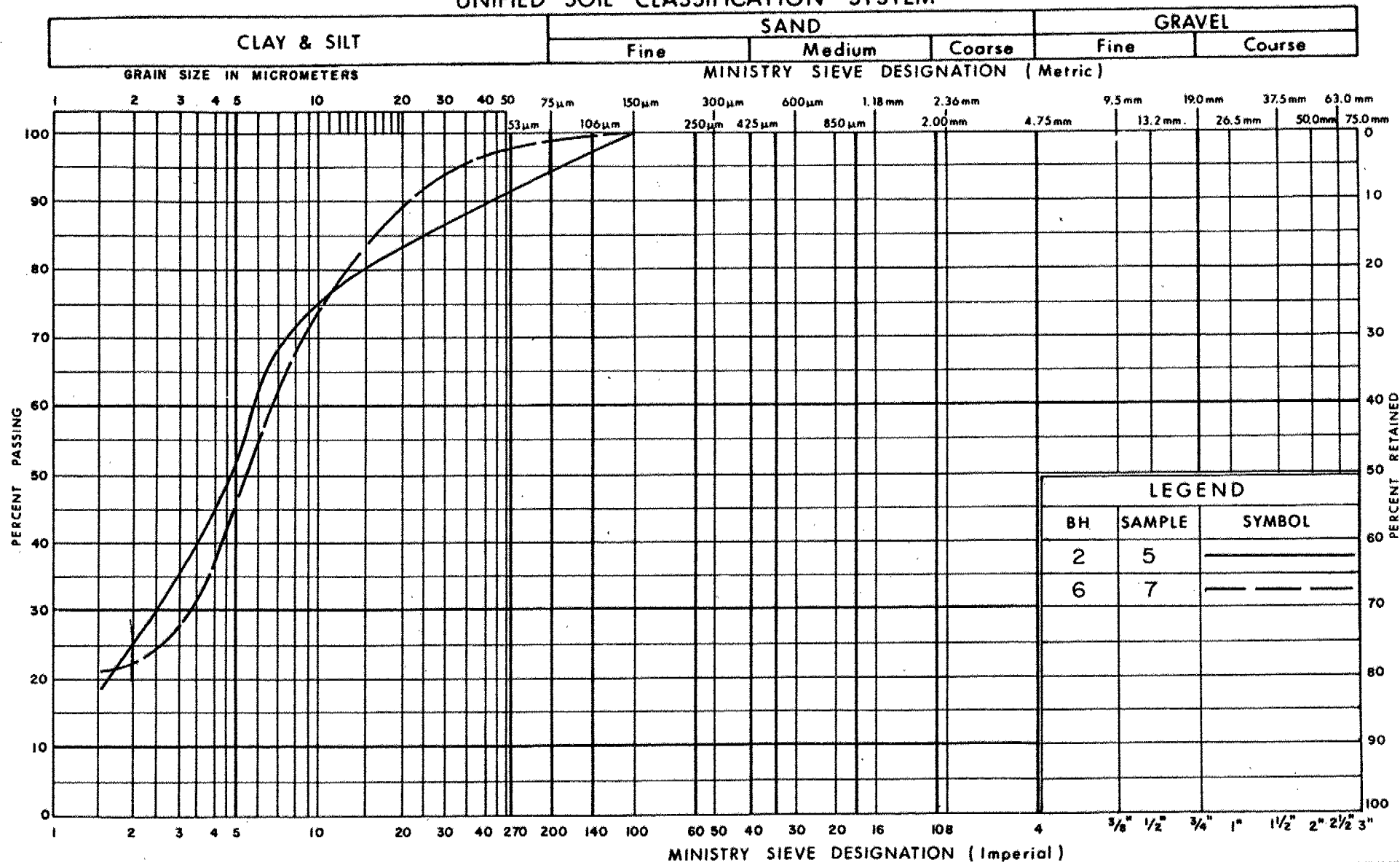
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
(CL-ML to CI)

FIG No 2

W P 145-80-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

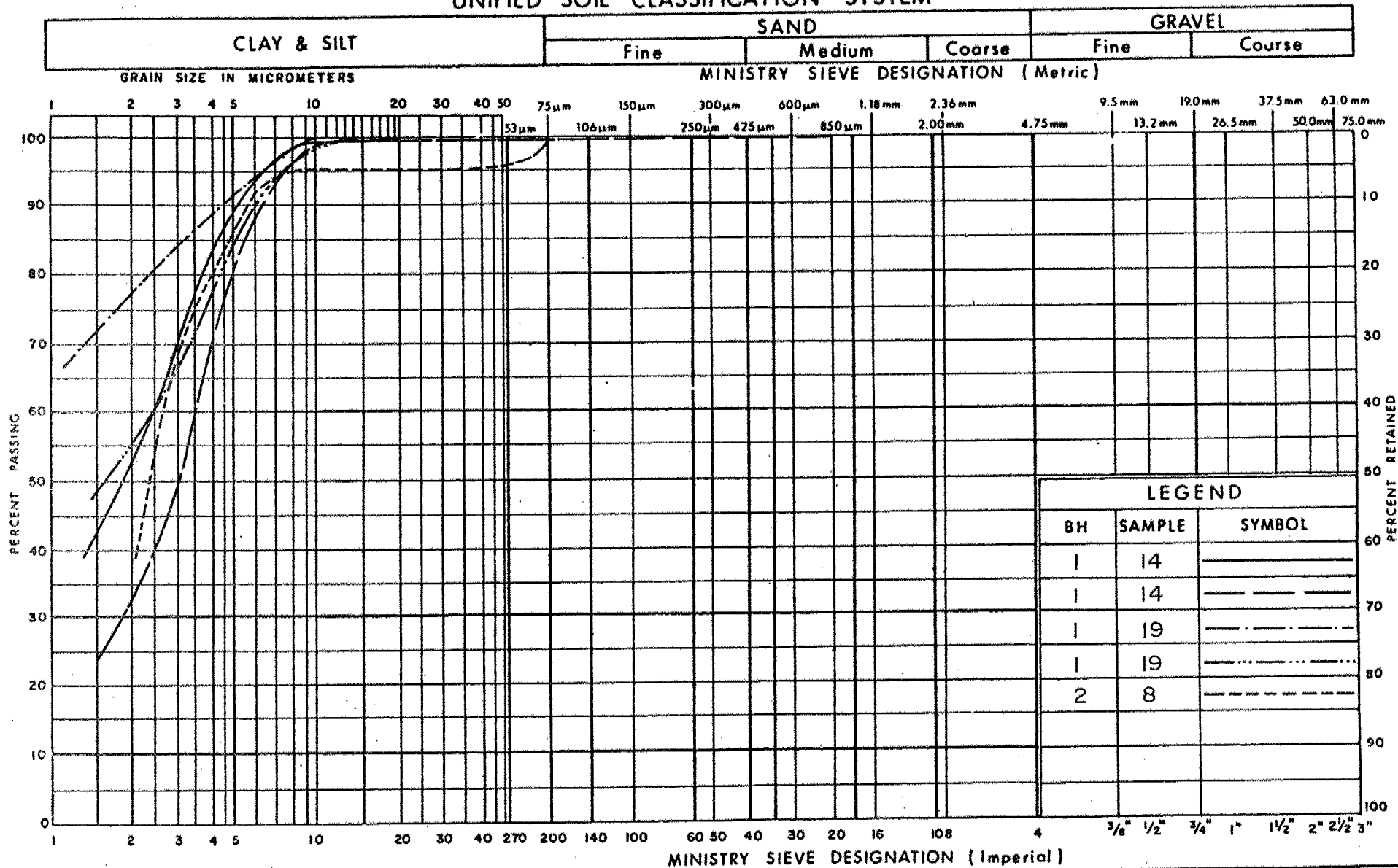
 Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAYEY SILT
(CI)

FIG No 3

W P 145 - 80 - 01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

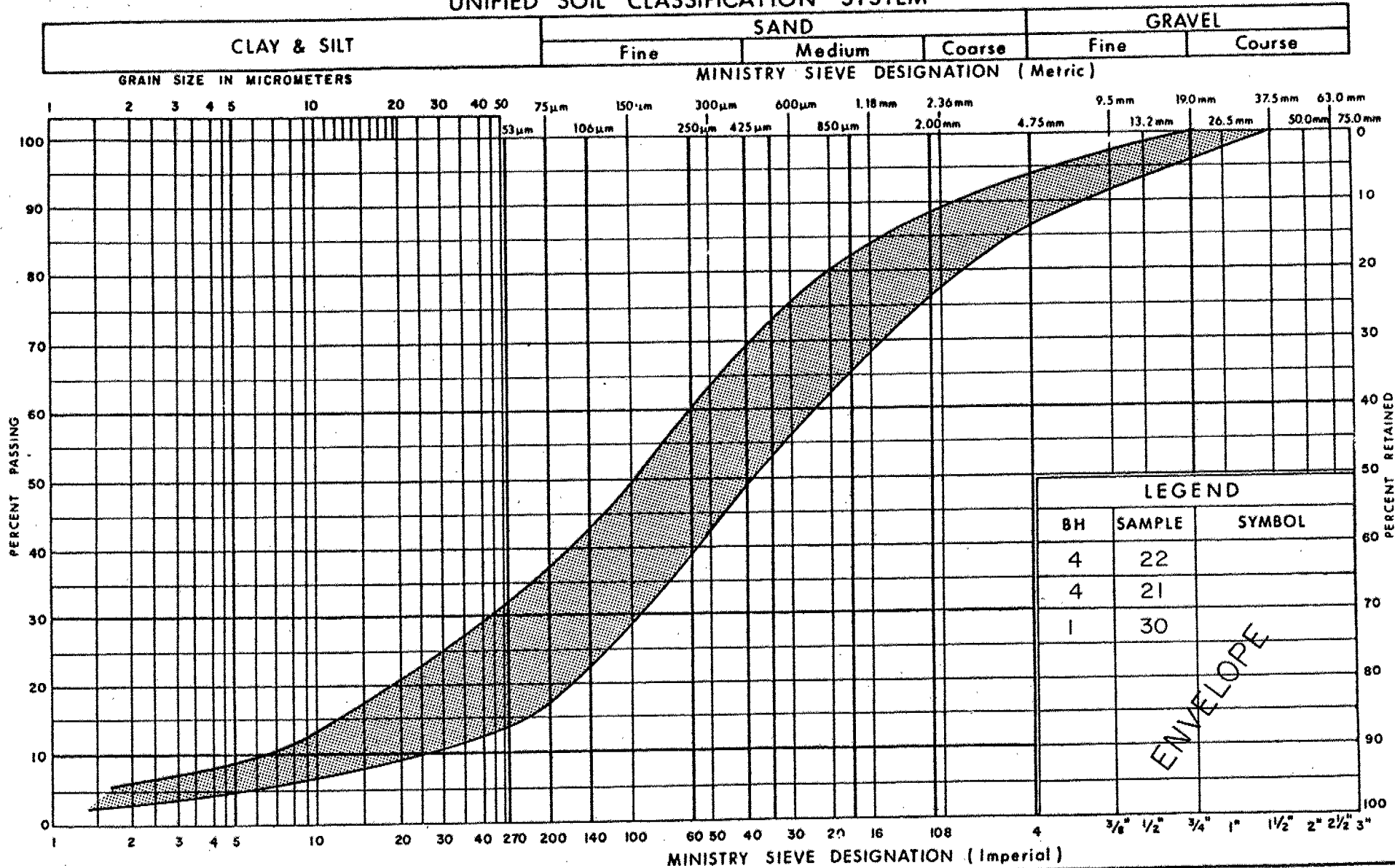
GRAIN SIZE DISTRIBUTION SILTY CLAY & CLAYEY SILT (CL, CH, ML)

NOTE: Tests carried on separated varve layers

FIG No 4

W P 145-80-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

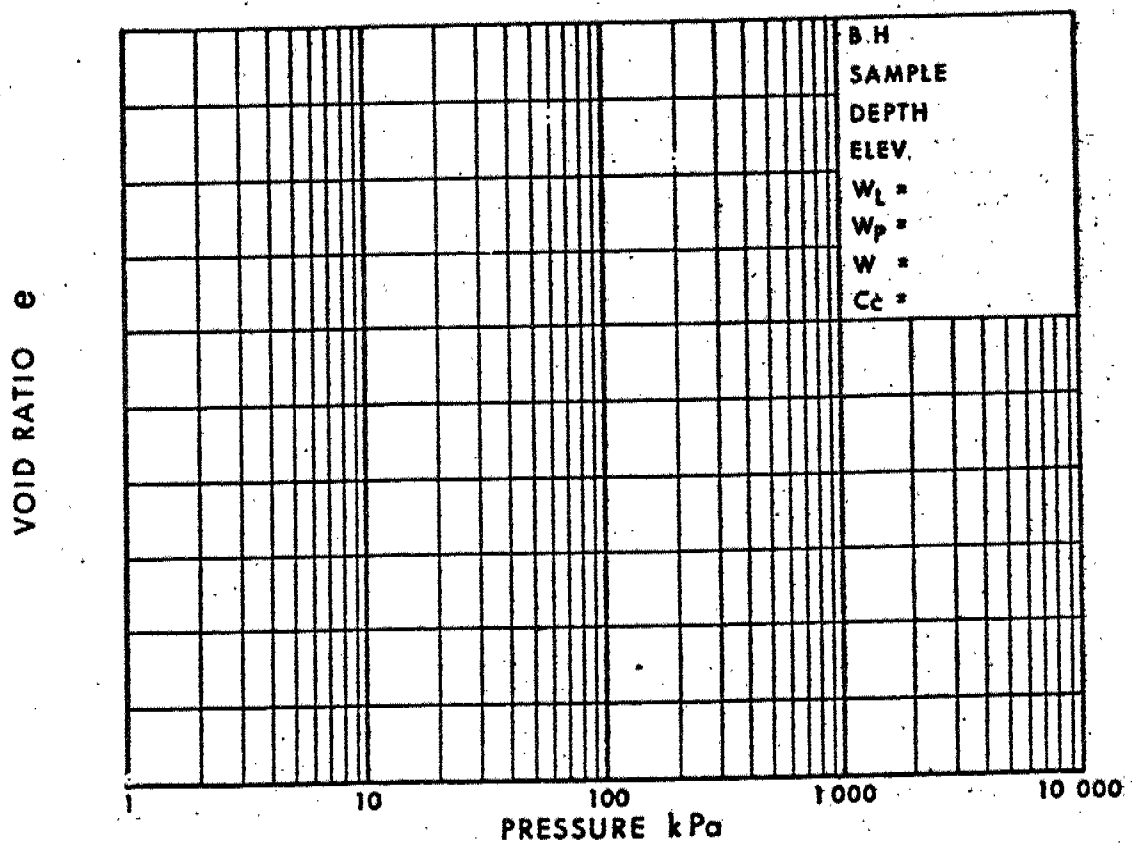
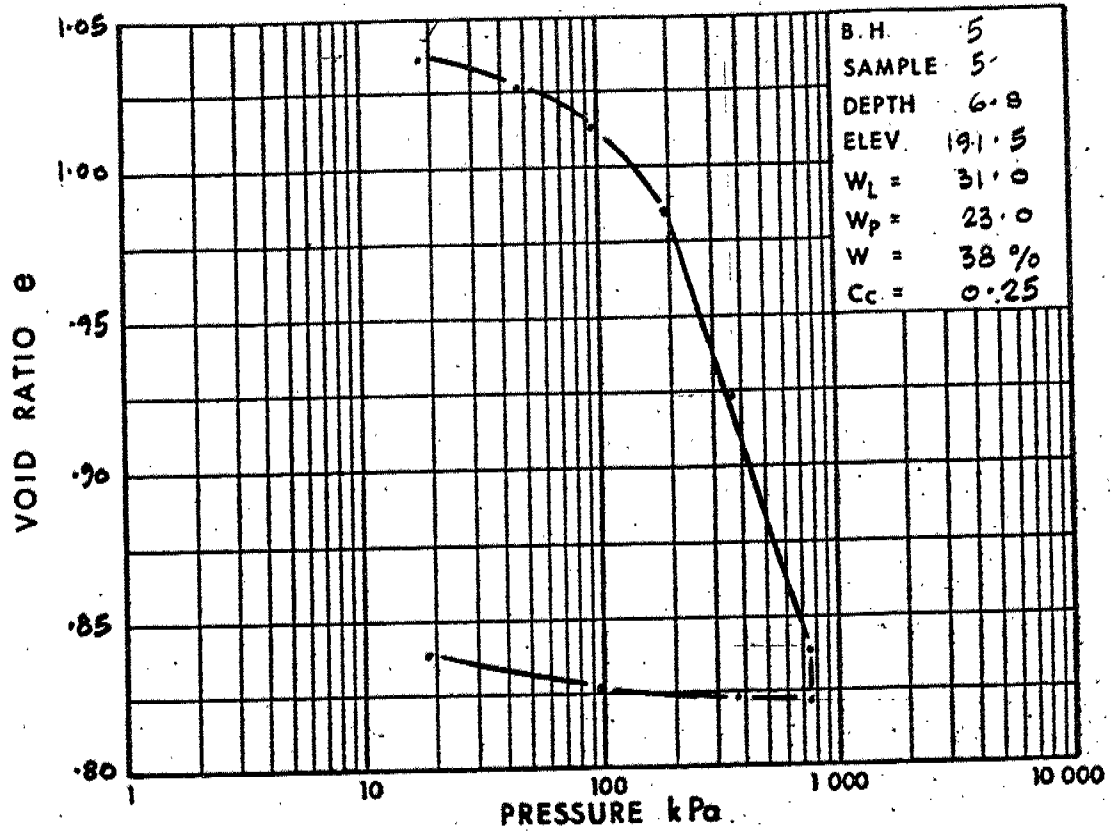
GRAIN SIZE DISTRIBUTION
SAND, TRACE TO SOME GRAVEL, SILT & CLAY (TILL)

FIG No 5

W P 145-80-01

VOID RATIO - PRESSURE CURVES

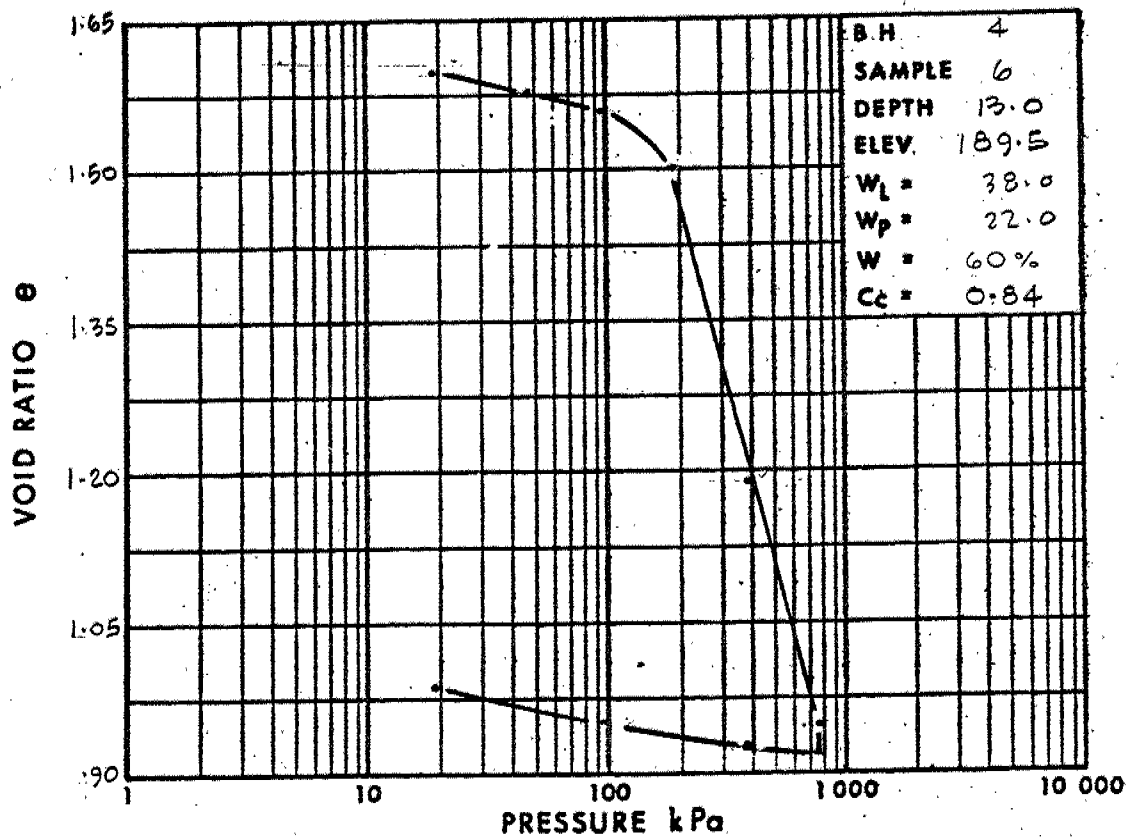
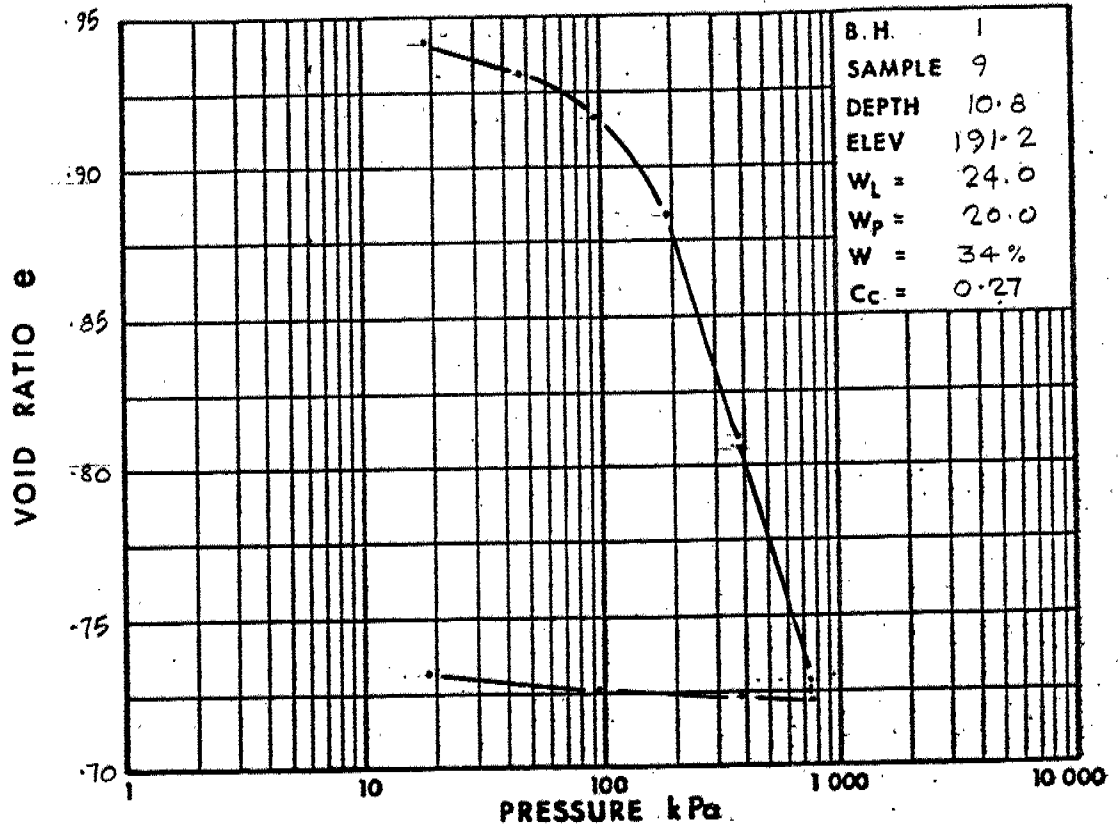
31



W P 145-80-01

VOID RATIO - PRESSURE CURVES

32

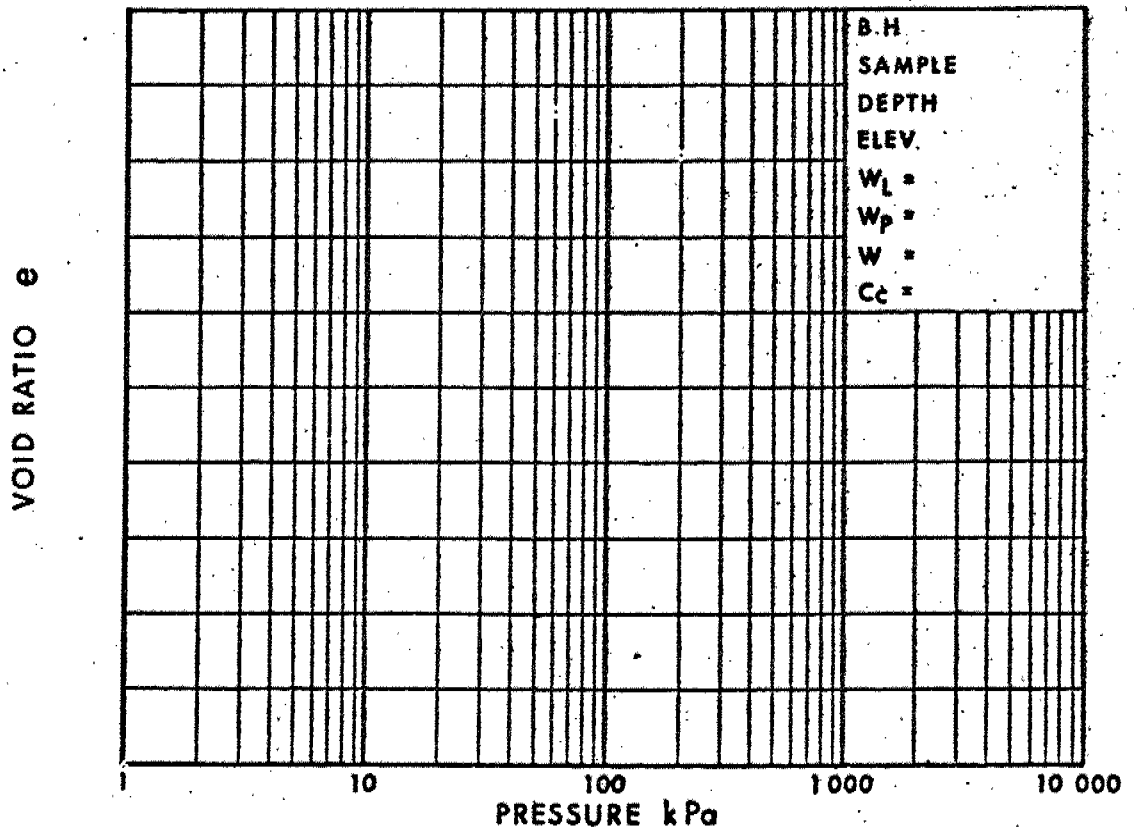
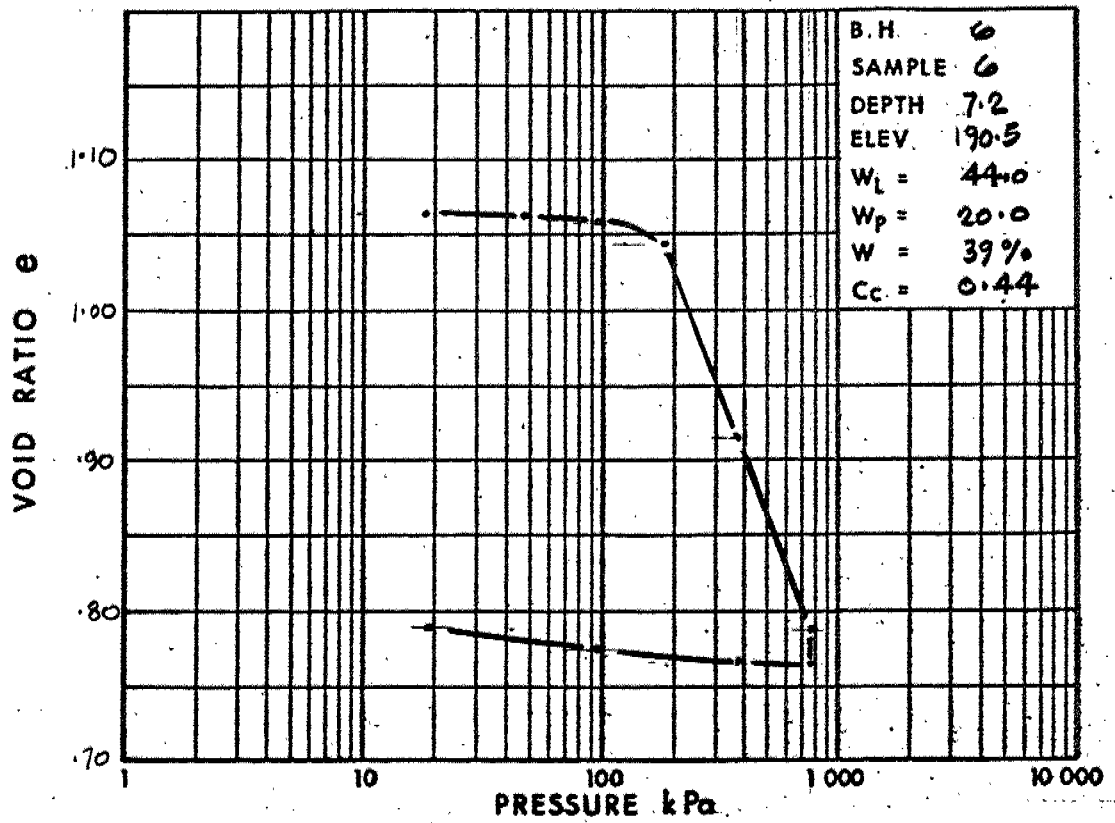


WP 145-80-01

FIG No 7

VOID RATIO - PRESSURE CURVES

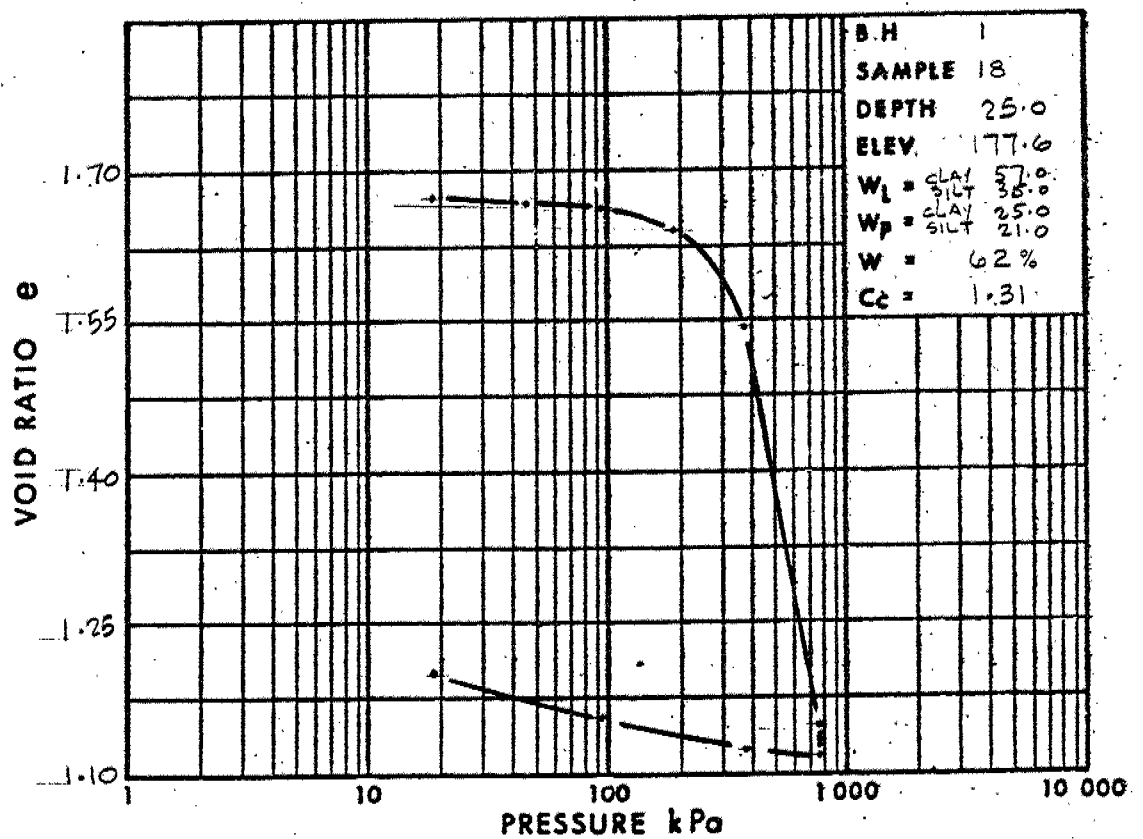
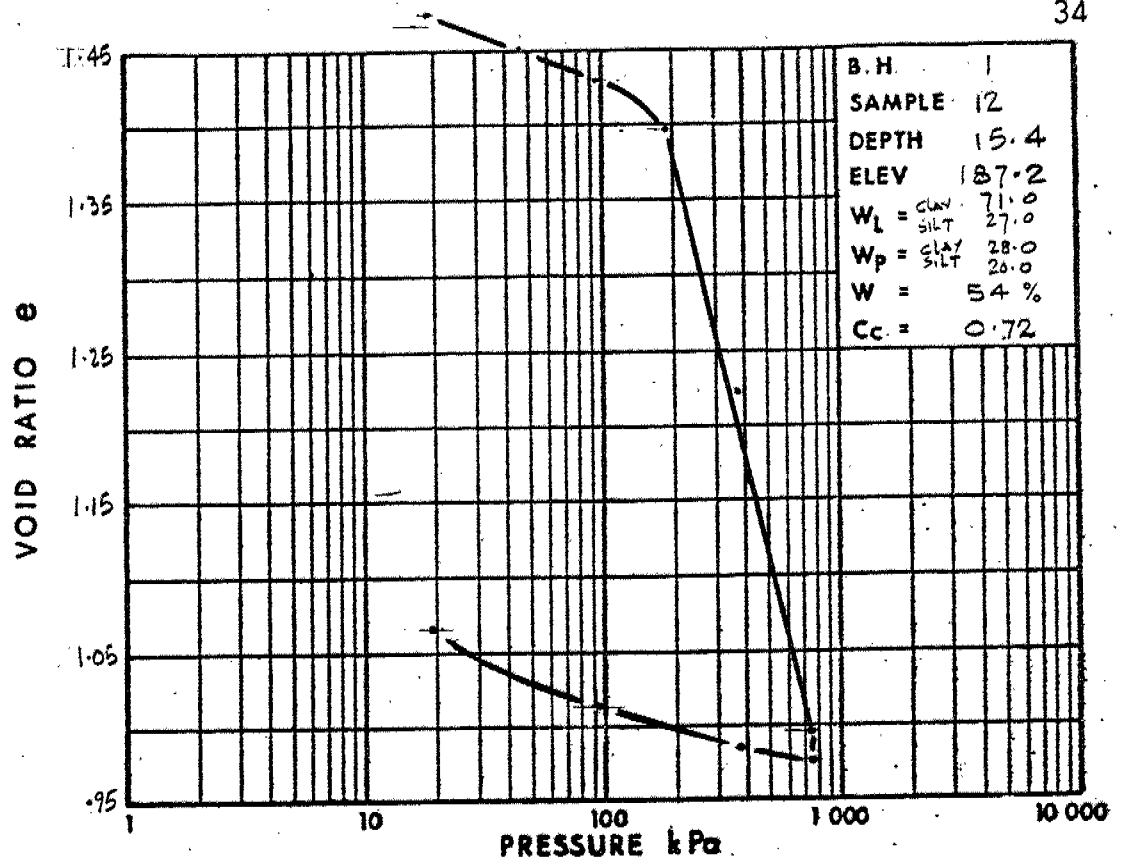
33



W P 145-80-01

VOID RATIO - PRESSURE CURVES

34

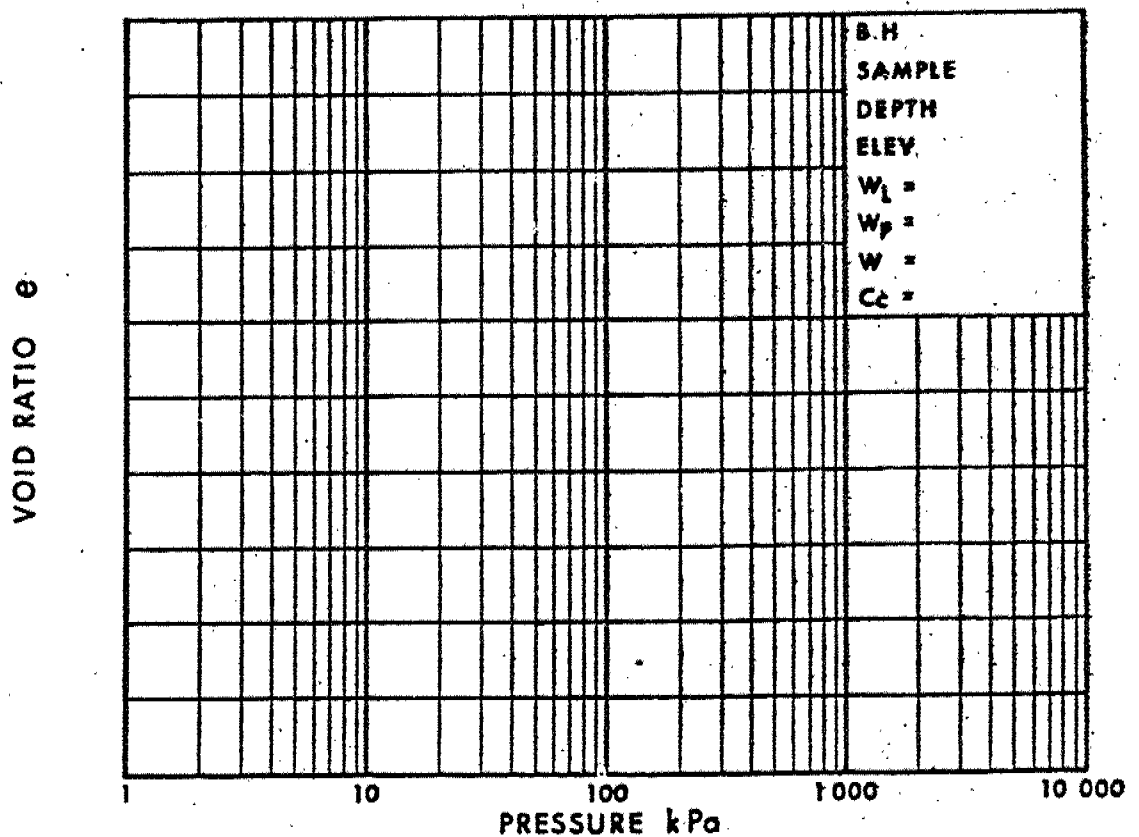
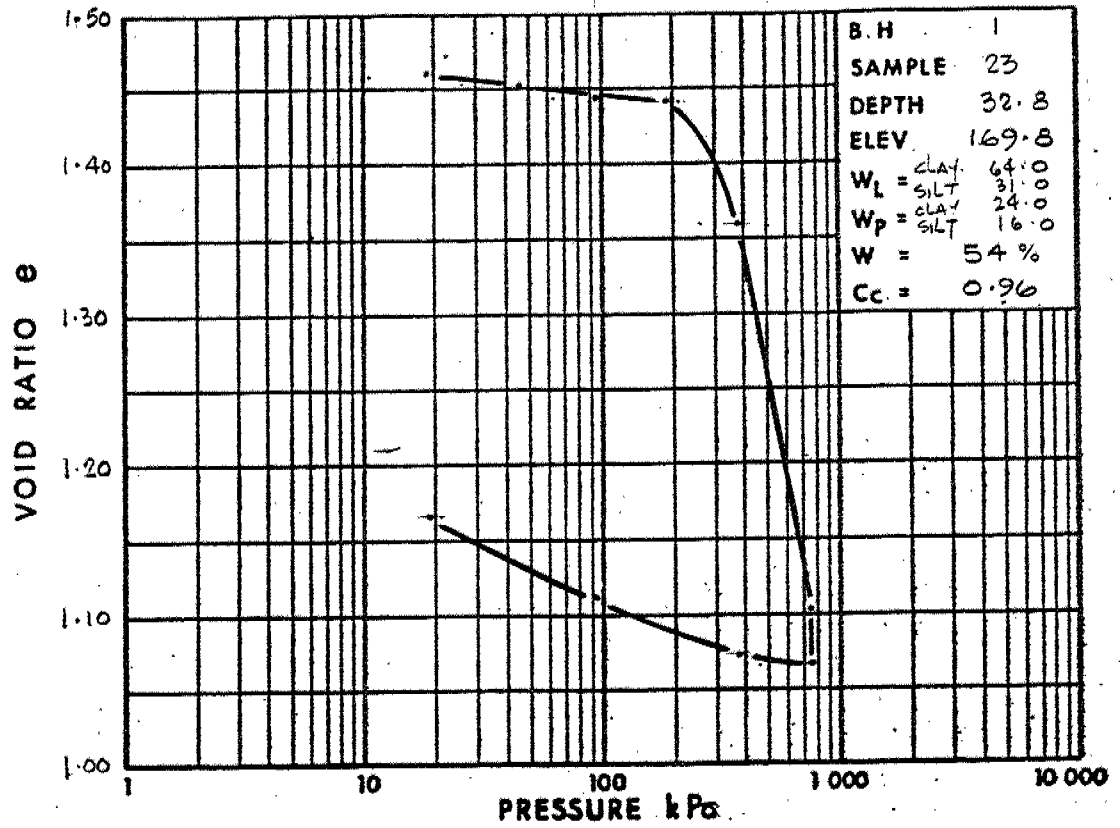


W P 145-80-01

FIG No 9

VOID RATIO - PRESSURE CURVES

35



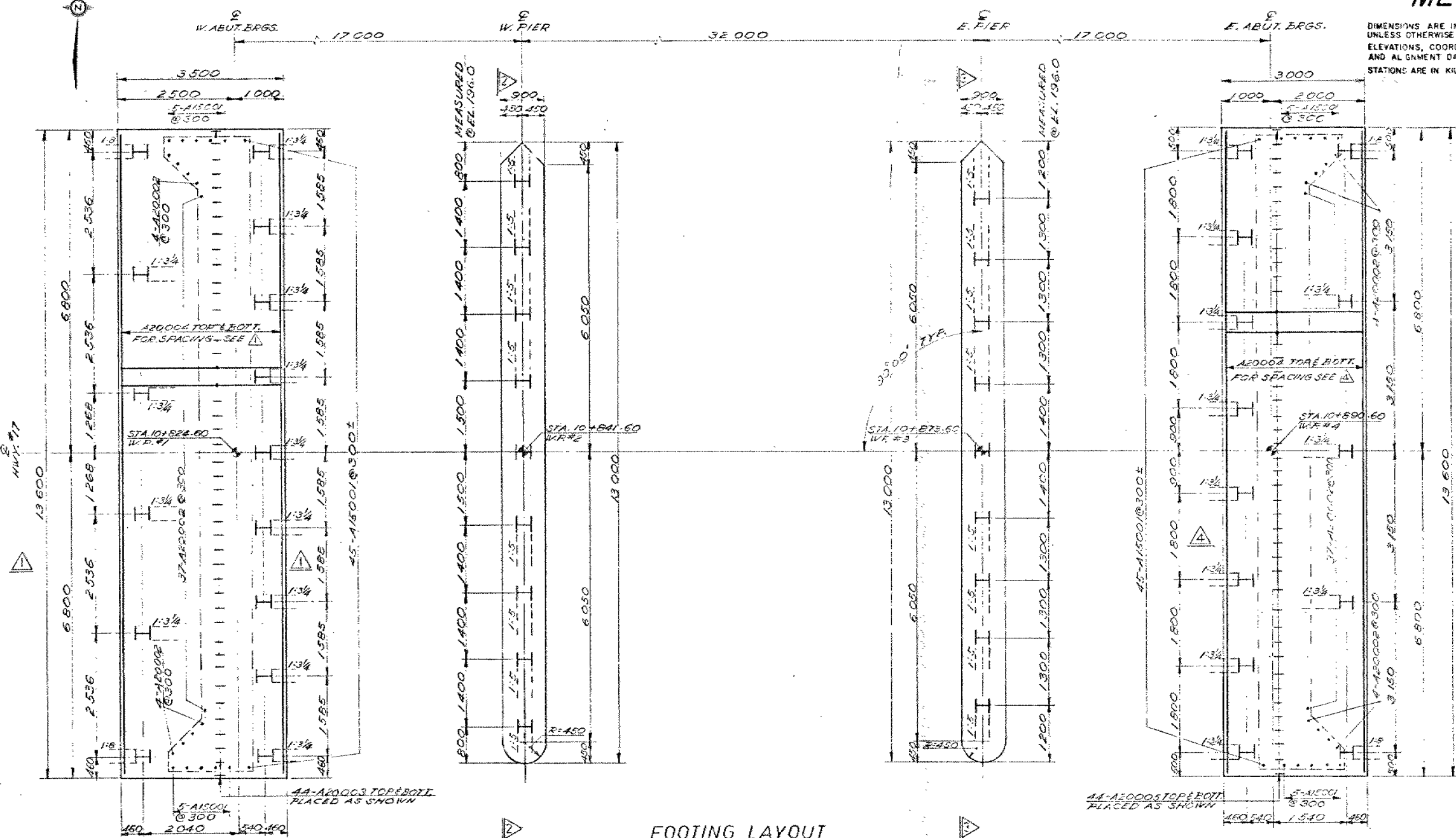
W P 145-80-01

FIG No 10

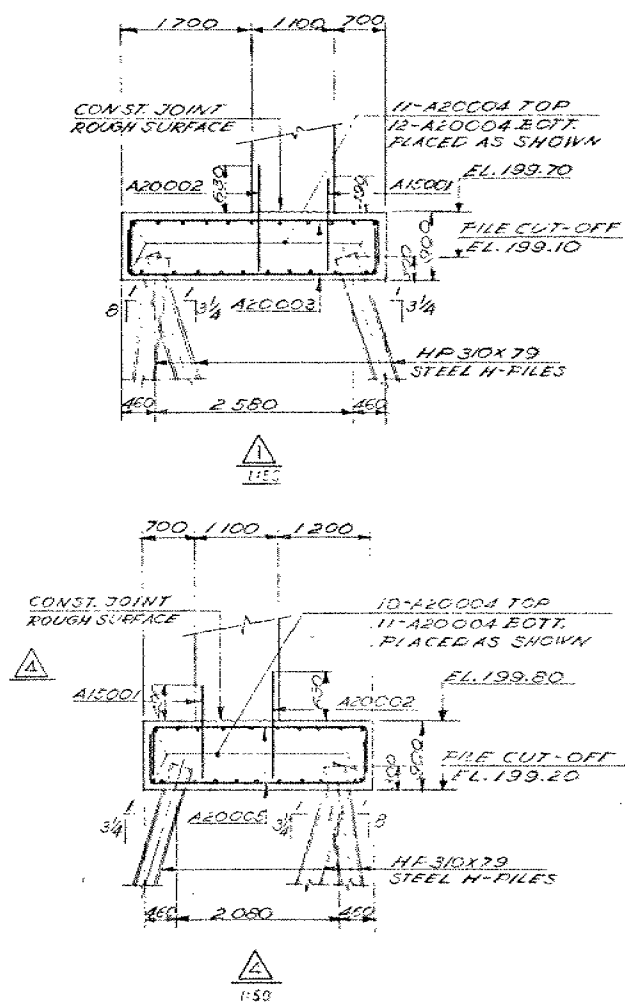
METRIC

LITTLE STURGEON RIVER BRIDGE
 550M WEST OF WEST JUNCTION OF HWY 17E
 FOOTING LAYOUT

DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE SHOWN.
 ELEVATIONS, COORDINATES, CURVE AND ALIGNMENT DATA ARE IN METRES.
 STATIONS ARE IN KILOMETRES + METRES.



FOOTING LAYOUT



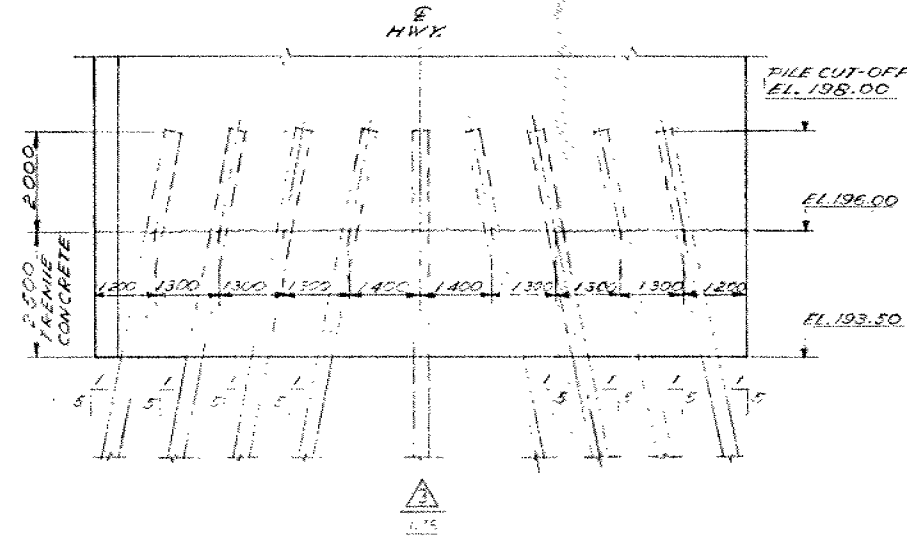
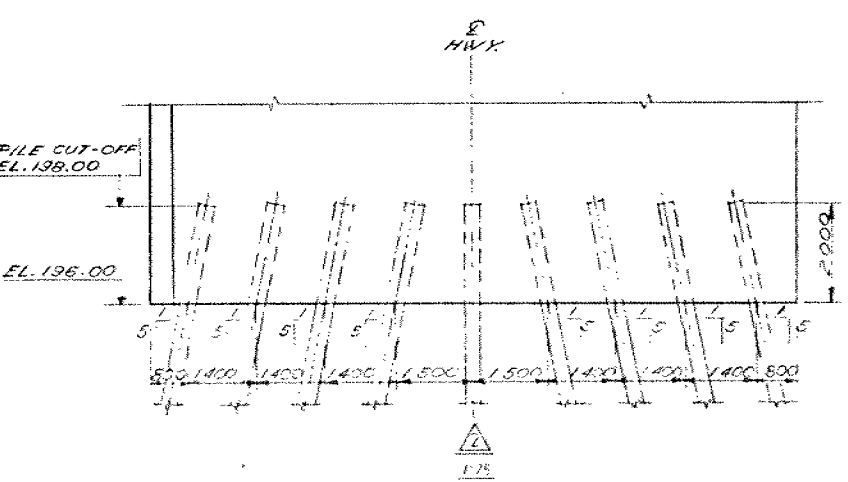
LIST OF STEEL H-PILES						
LOCATION	TYPE	N° REQ'D	LENGTH	PILE DESIGN DATA		REMARKS
				LOAD @ S.L.S. TYPE II	FACTORED CAPACITY @ U.L.S.	
W. ABUTMENT	HP 310x79	15	51,500	665 kN/FILE	1150 kN/FILE	1995 kN/FILE
W. PIER	HP 310x110	9	49,000	900 kN/FILE	1600 kN/FILE	2700 kN/FILE
E. PIER	HP 310x110	9	49,000	900 kN/FILE	1600 kN/FILE	2700 kN/FILE
E. ABUTMENT	HP 310x79	13	51,500	665 kN/FILE	1150 kN/FILE	1995 kN/FILE

NOTES:
 * PILE LENGTHS SHOWN ARE THEORETICAL LENGTHS BELOW CUT-OFF ELEVATIONS.



DRAWING NOT TO BE SCALED
 100 mm ON ORIGNAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION



CONT 82-216



REPORT TO
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS
ONTARIO

FOUNDATION INVESTIGATION
PROPOSED BRIDGE RECONSTRUCTION
HIGHWAY 17/LITTLE STURGEON RIVER

NORTH BAY

#13

W.P. 102-80-01

ONTARIO

145-80-01

Distribution:

- 12 copies - The Ministry of Transportation
and Communications, Ontario,
Pavement and Foundation Design Section
- 2 copies - Geocon (1975) Ltd.

24 March, 1981

GEOCON

GEOGRES No 3PL-50

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APPENDIX I

PROCEDURE AND FIELD EQUIPMENT

APPENDIX II

RECORD OF INDIVIDUAL BOREHOLES

LABORATORY FIGURES 1 to 12

DRAWINGS: 1458001-A
 1458001-B

1.0 INTRODUCTION

Geocon (1975) Ltd. has been retained by the Ministry of Transportation and Communications, Ontario (M.T.C.) to carry out a foundation investigation at the site of a proposed Bridge reconstruction at the crossing of the Little Sturgeon River approximately 25 km west of North Bay in the M.T.C. Northern Region, Ontario. Our proposal for the investigation was submitted to the M.T.C. on November 7th, 1980. Meetings were held at the M.T.C. Downsview offices on November 26th and December 8th, to review progress information from the field investigations as it influenced the selection of alternative foundation systems. At these meetings the original drilling programme was amended as subsurface conditions being encountered were significantly different from those anticipated at the time of submission of the original proposal, particularly in respect to depth to a competent bearing formation.

The purpose of the investigation was to obtain subsurface information for use in design and construction of foundations for the abutments and piers comprising the proposed Bridge structure and to obtain subsurface information on the alignment of the proposed temporary detour route.

2.0 SITE AND GEOLOGY

The site of the proposed Bridge reconstruction is on the Little Sturgeon River approximately 25 kilometres west of North Bay on Highway 17 between North Bay and Sturgeon Falls.

The existing Bridge at the site, reportedly constructed in 1937, is a three-span structure with a steel truss support for the central deck span and steel beam approach spans. The central span between the two concrete piers is about 28 metres and the approach spans are each about 9 metres in length. The land side of the approach spans are supported on concrete abutments which in turn are carried on timber piles. The tops of the timber piles have been exposed for a short section beneath the West Abutment. No specific details are available to us regarding the foundations of the existing Bridge.

2.0 SITE AND GEOLOGY (continued)

Visually, the abutment concrete has numerous cracks, some spalling and other signs of deterioration. Some rotation of the abutments has apparently taken place as indicated by a prominent crack in the asphalt pavement at each of the abutments. Differential movement has resulted in asphalt surface at the abutment which is raised between 10 to 25 mm relative to the approach span pavement surface.

It was observed also that the baseplates of the steel truss at the piers are dissimilar in that the plates on the North Bay side of the Bridge are bolted directly onto the top of the concrete pier, whereas on the Sturgeon Falls side shims have been inserted between the plate and the top of pier and the anchor bolts in the pier are inclined towards the abutment. It is not clear whether these features at the piers were part of the original construction or represent measures to compensate for later settlement or other movements during the operation of the Bridge.

The bed of the Little Sturgeon River beneath the Bridge is lined with recent deposits of sand and gravel, and there is evidence that some surficial rip rap protection was provided on the lower slopes of the approach fill embankments to below river level. As shown on the M.T.C. Plan E-5099-1, the river banks are relatively steep. The banks have a brown silty topsoil and are covered by grass and small trees. On the west bank, north of the Bridge, the ground appears to have been shaped to form an approach embankment possibly for a previous bridge crossing. A car parking area extends close to the top of the river bank on the east side.

Two concrete box culverts are present at the site to the east and west of the Bridge extending beneath Highway 17 and draining northwestwards and northeastwards, respectively. The culverts drain into gulleys in the river bank, exposing silt and clayey silt soils. The easterly culvert is some distance from the Bridge and a relatively steep sided gully is present between the culvert and the Little

2.0 SITE AND GEOLOGY (continued)

Sturgeon River. The westerly culvert is close to the existing Bridge and shows evidence of deterioration of the concrete and camber possibly under the weight of the highway approach embankment fill.

Available geological information^(1, 2) indicates that the local bedrock is Precambrian gneiss overlain by Pleistocene and recent deposits of gravel, sand, silt, boulder clay and varved clay and silt, or in larger areas, swamp accumulations. Outcrops of bedrock occur in the rapids of the Little Sturgeon River approximately one mile north of the bridge site. The bedrock becomes overlain by glaciolacustrine sediments from immediately below these rapids, and southwards to the mouth of the river where it flows into Lake Nipissing.

3.0 SUMMARIZED SUBSURFACE CONDITIONS

The stratigraphy encountered at the boreholes of this investigation is shown on the individual Records of Boreholes in Appendix II of this report, together with details of sampling and drilling and results of field and laboratory tests. A plan of the site showing borehole locations and inferred stratigraphic sections are presented on the enclosed Drawings 1458001-A and 1458001-B.

The field boring programme comprised one borehole in the area of each of the proposed abutment locations, one borehole within the plan limits of the proposed west pier and two boreholes, one on each side of the Little Sturgeon River, some 18 m north of the existing Bridge at the proposed location of the detour route and temporary Bailey Bridge. In addition, four shallow boreholes were put down along the proposed alignment of the detour route to be used during reconstruction of the existing Bridge.

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- (1) Ontario Department of Mines, 1968. 'Preliminary Geological Map No. R381, North Bay Sheet.' Scale 1 inch to 2 miles.
 - (2) Gartner, John F.; 1980, 'North Bay Area (NTS31L/SW), Districts of Nipissing and Parry Sound: Ontario Geological Survey, Northern Ontario Engineering Geology Terrain Study 101,' 19p. Accompanied by Maps 5041 and 5044, Scale 1:100,000.

3.0 SUMMARIZED SUBSURFACE CONDITIONS (continued)

In the area of the proposed abutments, the natural soils are overlain by some 6.6 and 1.8 metres of fill on the western and eastern approaches, respectively. The fill on the western side consists of 0.6 metres of existing pavement materials underlain by 6.0 metres of silt, some clay, containing traces of sand, gravel and organics. From the available evidence,⁽³⁾ it appears that silt fill comprises backfill to the existing concrete box culvert, and that the fill probably decreases in thickness in the direction of the existing abutment. On the eastern side, the approach fill consists of a thin layer of sand and gravel pavement materials overlying natural ground.

The cohesive soils encountered at the site have been divided into four separate strata on the basis of visual and tactile examination and the results of field and laboratory tests as follows:

<u>Approximate Thickness Range (m)</u>	<u>Principal Soil Types</u>	<u>Colour</u>	<u>Approximate Range of Elevation of Top of Stratum (m)</u>
1) 3 to 7.5	Clayey silt to silt	Grey and Brown	196 to 200
2) 2.5 to 3.0	Silty Clay to Clayey Silt	Dark Grey	191 to 193
3) 22.5 to 23.0	Silty Clay, Clayey Silt, Silt - Varved	Light and Dark Grey	189 to 190
4) 13 to 14	Silty Clay, Clayey Silt, Silt - Varved	Light and Dark Grey and Brown	166 to 167

The consistencies of the strata listed above down to approximately elevation 188 are generally firm, with the exception of conditions found at Borehole 2 where these strata are soft to firm. The upper varved stratum has a consistency ranging from firm to stiff and the lower varved stratum is generally stiff, ranging to very stiff.

It is judged from the results of consolidation tests on samples from the strata of clayey silt, silty clay and the upper part of

(3) Department of Northern Development, District of Sturgeon Falls, 1936. "Plan and Profile Meadowside Bridge, Bridge Site Over Little Sturgeon River." Drawing No. B2110. Scale 1 inch = 10 feet. March 5th, 1936.

3.0 SUMMARIZED SUBSURFACE CONDITIONS (continued)

the varved clay, that these strata are slightly overconsolidated with respect to present overburden pressures.

Underlying the cohesive soils a granular stratum was encountered at a depth of approximately 49 metres corresponding to a range of elevation from 153 to 154. It is inferred that this stratum, which consists primarily of sand, with traces of silt and gravel is glacial drift, and it is thus identified herein as a till. The till stratum, inferred to have a generally dense relative density was penetrated to depths of 6.0 and 2.5 metres.

No significant thicknesses of organic material were encountered at the four shallow boreholes put down along the proposed temporary detour route alignment. The upper one to two metres of natural clayey silt to silt penetrated in these boreholes is very loose in some places.

Groundwater levels were recorded in piezometers installed in Boreholes 1 and 2, a standpipe in Borehole 1, and in the open borehole in the case of Borehole 4. The most recent observations (taken on December 21st, 1980) are shown on the Records of Boreholes and Drawing 1458001-A.

Considering the relatively short time period between installation and observations, it is not certain whether the water levels as shown represent stabilized levels.

4.0 DETAILED SUBSURFACE INFORMATION

The individual strata encountered at the boreholes are described in the following sections. The M.T.C. Modified Unified Soil Classification symbols have been applied to the overburden soils where applicable.

4.1 Topsoil (Pt)

A surficial layer of topsoil was penetrated in three of the four boreholes located on the proposed detour route alignment. The topsoil was mainly silt mixed with humus and other

4.0 DETAILED SUBSURFACE INFORMATION

4.1 Topsoil (Pt) (continued)

vegetation debris and ranged in thickness from 75 to 100 mm. Beneath the west approach embankment, traces of inferred original topsoil were observed at the junction of the fill and natural soils.

4.2 Fill

The boreholes at the proposed abutment sites were put down through the edge and shoulder of the existing Highway 17 pavement. At the proposed west abutment, 150mm of asphalt and 450mm of sand and gravel were penetrated while at the east abutment, 2.8m of sand and gravel was penetrated on the shoulder of the highway.

Underlying the existing pavement materials in the area of the proposed west abutment, some 6.0 metres of fill was encountered. The fill materials are predominantly silt, some clay mixed with traces of sand, gravel and traces of organics (topsoil and occasional wood fragments).

Water content determinations from samples of the silty fill ranged from about 19 to 30 percent.

The two shallow boreholes on the proposed detour alignment to the west of the Little Sturgeon River encountered a surficial fill layer of 0.7m and 1.3m thickness of silty sand, trace to some gravel and trace of organics. The eastern most borehole along the detour alignment, located at the edge of Highway 17 on the north shoulder, encountered 0.8 m of sand and gravel fill. Water content determinations in the silty sand fill ranged from 15 to 18 percent.

Standard Penetration Tests carried out within existing approach embankments gave "N" values ranging from 21 to 5, indicating a compact to loose relative density. "N" values ranging from 7 to 15, were obtained in the silty sand and sand and gravel fill encountered elsewhere. A 0.3 m surface layer

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.2 Fill (continued)

of sand and gravel was encountered at Borehole 2 adjacent to the Little Sturgeon River and has been included in this section although this granular material is likely a recent deposit laid by the river and not fill. From site observations before the ice formed, greater thicknesses of similar materials may be present within the plan areas of the proposed piers of the new bridge structure, possibly with occasional small boulders displaced from the toe areas of the existing embankments.

4.3 Clayey Silt (CL-ML to CI)

A stratum of clayey silt was encountered underlying the embankment fills, and at ground surface at the other boreholes. Where fully penetrated, the stratum ranges in thickness from about 3.0 to 7.5 metres at the proposed Bridge site and some 6.0 to 7.0 metres at the proposed Bailey Bridge site. The soil has a high silt content and is slightly cohesive. The clayey silt is mainly grey in colour with brown areas (possibly due to oxidation), has occasional very thin laminae of grey silt and/or fine sand, and a few pockets or lenses of grey silt. The majority of the recovered samples contain a relatively uniform trace of organics.

Atterberg limit tests carried out on six selected samples gave Liquid Limits ranging from 27 to 42 percent, Plastic Limit 22 to 31 percent for resultant Plasticity Indices of 4 to 15.5. Natural moisture contents of these samples ranged from 29 to 35 percent, respectively. The Plasticity Chart, (Figure 1 in Appendix II) shows the above results, indicating silt and clay of low to medium plasticity grading to organic silt.

Three grain size distribution tests were carried out on the same samples as selected for the Atterberg limit tests and the resultant curves are shown on Figure 2 in Appendix II. The samples as tested contain a trace of sand, 77 to 87 percent silt sizes and 12 to 15 percent clay sized particles.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.3 Clayey Silt (CL-ML to CI) (continued)

Wet unit weights determined in the course of the triaxial and consolidation tests were both close to 17.9 kN/m^3 .

One unconsolidated undrained triaxial test was carried out on a sample recovered from the clayey silt stratum and gave an undrained shear strength, taken as one half the compressive strength, of 12 kPa.

In-situ shear strengths, determined by field vane ranged from about 32 kPa to 80 kPa. Based on the natural and remoulded shear strength values obtained by field vane, the sensitivity is in the range of 2 to 4. Shear strengths were also measured using a Geonor miniature laboratory vane in the bottom of the thin walled steel tube samplers at the time of sample recovery. The shear strengths as measured by laboratory vane ranged from about 30 kPa to 55 kPa. The sensitivity of the stratum, from natural and remoulded shear strength values using the laboratory vane, ranged from 4 to 7. There is a significant variation in undrained shear strengths as obtained by the two types of vane apparatus, and the one triaxial test. This is discussed later in the report. On the available evidence, the stratum is inferred to be generally of firm consistency.

A Standard Penetration Test carried out in the stratum adjacent to the river bed gave an "N" value of 2 and the following split spoon sample was pushed manually. Thus, the clayey silt stratum is somewhat softer at this location, possibly due to erosion and deposition.

One consolidation test was carried out on a sample from the clayey silt stratum, and the results are presented as a pressure/void ratio curve on Figure 6 in Appendix II. The computed Compression Index (C_c) is 0.25.

4.4 Silty Clay (CI)

Underlying the clayey silt a stratum of silty clay was encountered in the five deeper boreholes of the investigation.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.4 Silty Clay (CI) (continued)

Where fully penetrated, the stratum ranged in thickness from 2.5 m to 3.0 m. The silty clay is dark grey in colour and several samples showed discontinuous small lenses or pockets of light grey silt dispersed through the dark grey silty clay. Thin horizontal laminae of grey silt to fine sand were occasionally present.

Atterberg limit tests were carried out on six selected samples, two from siltier portions and four from silty clay portions. The samples with higher silt content had a Liquid Limit of 24 percent, Plastic Limits of 18 percent to 20 percent for resultant Plasticity Indices of 4 and 6. Reference to the Plasticity Chart, Figure 1, Appendix II indicates for Sample 9, Borehole 1 and Sample 5, Borehole 2, a low plasticity inorganic clayey silt with a CL-ML, Unified Soil Classification symbol. Natural water content of these samples was 34 percent and 44 percent, respectively. In the case of the samples with higher clay content, Liquid Limits ranged from 37 to 44 percent, Plastic Limits of 20 percent to 22 percent, for resultant Plasticity Indices of 16 to 24. These results as shown on the Plasticity Chart indicate an inorganic silty clay having an M.T.C. Soil Classification of CI. Natural water contents of these samples ranged from 32 to 42 percent, respectively.

Grain size distribution tests were carried out on two of the samples selected for Atterberg Limit tests and the resultant curves are shown on Figure 3 in Appendix II. The grain size distributions are closely similar, and the samples as tested contain 2 to 8 percent sand, 67 to 75 percent silt sizes and 23 to 25 percent clay sized particles.

Wet unit weights determined in the course of triaxial and consolidation tests carried out on samples from the silty clay stratum ranged from 16.3 kN/m^3 to 18.5 kN/m^3 .

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.4 Silty Clay (CI) (continued)

Three unconsolidated undrained triaxial tests carried out on selected samples of the silty clay stratum gave shear strengths (taken as one half the compressive strength) ranging from 20 kPa to 43 kPa. In order to investigate possible sample disturbance, two triaxial tests were performed on different portions of a single thin walled sample from Borehole 4. The shear strength of the sample from the end of the tube was 23 kPa and the sample from the middle of the tube was 20 kPa.

In-situ undrained shear strengths, determined by field vane generally ranged from 30 kPa to 55 kPa. Based on a comparison of the natural and remoulded shear strengths obtained, the field vane results indicate a sensitivity in the range of 2 to 4. Shear strengths determined on site using a Geonor miniature laboratory vane ranged from 24 kPa to 40 kPa. The sensitivity of the silty clay stratum based on laboratory vane values generally ranged from 6 to 10. On the evidence of measured individual shear strengths, the consistency of the silty clay stratum ranges from firm to stiff, being generally firm. As for the clayey silt stratum, the variations in measured undrained strengths by the three types of test, are discussed later in the report.

Three consolidation tests were carried out on selected samples from the silty clay stratum, and the results are presented on Figures 7 and 8 in Appendix II. The Compression Indices (C_c) from these tests were from 0.27, 0.44 and 0.84.

4.5 Silty Clay, Clayey Silt and Silt,
Varved (CH, CL, ML) (Upper Stratum)

Underlying the silty clay in the two deeper boreholes and the two boreholes adjacent to the river on the west side, a varved clay stratum was encountered. The stratum was fully penetrated in the two deeper boreholes for a thickness of between 22.5 and 23.0 metres, corresponding to an approximate

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.5 Silty Clay, Clayey Silt and Silt,
Varved (CH, CL, ML) (Upper Stratum) (continued)

elevation range of 190 to 167. The structure exhibited by samples consists of repeated layers of dark grey silty clay and light grey clayey silt and silt, with transitions observed with each varve unit as the relative contents of clay and silt change from silt to a high plasticity clay. Generally, individual layers range from about 8 mm up to 30 mm in thickness. A similar structure was exhibited below elevation 167 but on the basis of strength results and visual examination two distinct varved clay strata are distinguished. The varved clay to about elevation 167 is referred to as the upper varved clay.

Atterberg Limit tests were carried out on a number of selected samples after visual separation of the individual layers. The results of these tests are shown on the Plasticity Chart, Figure 1 and on the Records of Boreholes in Appendix II. Typically, for the dark grey clay layers, Liquid Limits ranged from 60 to 77 percent, Plastic Limits 24 to 39 percent for resultant Plasticity Indices of 29 to 44, whereas for the siltier layers, Liquid Limits ranged from 27 to 40 percent, Plastic Limits from 16 to 25 percent for resultant Plasticity Indices from 7 to 16.

Grain size distribution tests were carried out on three samples which were selected for Atterberg Limit tests. The grain size tests were carried out on separated layers as shown on Figure 4, in Appendix II. The finer layers tested contained 47 and 23 percent silt and 53 and 77 percent clay sized particles. The coarser layers contained between 68 and 45 percent silt and between 32 and 55 percent clay sized particles.

Wet unit weights determined in the course of the triaxial and consolidation tests ranged from 16.0 kN/m^3 to 18.5 kN/m^3 . Unit weights in each case were determined prior to testing.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.5 Silty Clay, Clayey Silt and Silt,
Varved (CH, CL, ML) (Upper Stratum) (continued)

Six unconsolidated undrained triaxial tests carried out on samples recovered from this upper varved stratum gave shear strengths ranging from 30 kPa to 50 kPa. Two tests were performed on separate sections of one thin walled steel tube sample. The shear strengths were 41 kPa, for the portion of the sample close to the centre of the tube and 30 kPa for the portion close to the end of the tube.

In-situ undrained shear strengths determined by field vane ranged from 35 kPa to 125 kPa. Based on the natural and remoulded shear strength values the field vane results indicate a sensitivity in the range of 2 to 5. Undrained shear strengths as measured on site by laboratory vane ranged from 21 kPa to 75 kPa. The sensitivity of the varved stratum using a laboratory vane ranged from 6 to 11. There is a significant variation in undrained shear strengths as obtained by the two types of vane apparatus, and the triaxial tests. This is discussed later in the report. On the evidence of shear strength values, the consistency of the upper varved stratum was generally firm to stiff.

Three consolidation tests were carried out on selected samples of the upper varved stratum and the results of these tests are shown on Figures 9 and 10 in Appendix II. For each test the samples were trimmed with the layers horizontal. Generally the samples tested included both layers of silt and clay sized particles. The computed Compression Indices (C_c) from these tests are 0.72, 1.31 and 0.96.

4.6 Silty Clay, Clayey Silt and Silt,
Varved (CH, CL, ML) (Lower Stratum)

The upper varved stratum is underlain by a lower varved stratum, characterized by the presence of a brown to reddish brown clay layer interbedded with the light grey silt and dark grey silty clay. The repeated varve units show evidence of

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.6 Silty Clay, Clayey Silt and Silt,
Varved (CH, CL, ML) (Lower Stratum) (continued)

transition from silt through clayey silt to clay. Generally, the recovered samples show individual layers in this stratum ranging in thickness from some 7 mm up to about 12 mm.

Atterberg Limit tests were carried out on separated varve layers from one selected sample and these results are shown on the Plasticity Chart, Figure 1 in Appendix II. For the dark grey clay layer the Liquid and Plastic Limits determined were 91 percent and 32 percent, for a brown clay layer 73 percent and 23 percent, and for a siltier layer 28 percent and 20 percent. One additional Atterberg Limit test carried out on a sample recovered from close to the base of the stratum gave a Liquid Limit of 62 percent, and Plastic Limit of 26 percent.

Two unconsolidated undrained triaxial tests were carried out on samples from the lower varved stratum. The undrained shear strengths taken as one half the compressive strength averaged about 40 kPa.

Wet unit weights determined in the course of the triaxial tests were 17.6 kN/m^3 and 17.8 kN/m^3 .

In-situ undrained shear strengths were determined by field vane and ranged from 135 kPa to 240 kPa. Based on the natural and remoulded shear strength values obtained using a field vane, the sensitivity of the varved stratum ranges from 2 to 9. The shear strengths as measured by laboratory vane ranged from 55 kPa to 90 kPa. The sensitivity based on strength values using a laboratory vane ranged from 2 to 4. As for the overlying strata there is a large variation in undrained shear strengths as obtained by the two types of vane apparatus and the triaxial tests. This is discussed later in the report. On the evidence of shear strength values the consistency of the lower varved stratum generally ranges from stiff to very stiff.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.7 Sand, Trace to Some Gravel, Silt and Clay (Till)

Underlying the lower varved stratum, a granular stratum was encountered in the two deeper boreholes. The top of the stratum was found between elevations 153 and 154. The recovered samples consisted mainly of sand sizes with varying contents of gravel, silt and clay. Visually, the granular soil has no structure. It is considered to be glacial drift, and has thus been classified herein as till.

Grain size distribution tests were carried out on standard split spoon samples from the granular stratum and the results of three tests are presented as an envelope on Figure 5 in Appendix II. The results indicate between 6 and 13 percent gravel, 52 and 70 percent sand, 13 and 31 percent silt and 4 and 6 percent clay sized particles. The results do not reflect the content of sizes greater than about 37 mm. On geological evidence, larger sizes could be expected to be encountered within the formation, possibly up to and including boulder sizes.

Standard Penetration Tests carried out within the till stratum gave "N" values ranging from 60 to greater than 100. The "N" values infer a very dense relative density, although it should be noted that the measured values could have been influenced by coarse sizes in the till, and that they reflect the significant depth of drilling.

4.8 Groundwater Conditions

For purposes of monitoring groundwater levels across the site one piezometer was sealed within the till at Borehole 1 and one piezometer in the area of the proposed West Pier at Borehole 2. In addition a standpipe was installed in Borehole 1 and the water level observed in the open hole subsequent to drilling in Borehole 4. The latest groundwater levels observed, taken on 21st December, 1980, are shown on the Records of Boreholes in Appendix II and the enclosed Drawings 1458001-A and -B.

4.0 DETAILED SUBSURFACE INFORMATION (continued)

4.8 Groundwater Conditions (continued)

The water level in the Little Sturgeon River at the time of the investigation was lower than the 195.9 elevation dated October 11th, 1980 shown on the Drawings and measured at elevation 194.7, as top of ice at Pentest 3 on December 19th, 1980.

Groundwater levels in the existing approach embankment fills ranged between approximate elevations 198 and 200, or some 2 to 4 metres above the water level in the Little Sturgeon River at the time. At the proposed West Pier location, the groundwater level was measured some 3 metres below ground surface, which corresponded to about 2.0 metres below river level at the time of drilling.

Since the fill and natural soils at the site are fine-grained and of relatively low permeability, the measured groundwater levels may not have stabilised over the period of the readings.

5.0 DISCUSSION

The existing Bridge has a centre span of about 28 m and approach spans each about 9 m. It is understood that the centreline of the Piers for the new Bridge would each be located about 3 m closer to the centre of the river than the existing Piers, to give a new centre span length of about 22 m. At the same time, the new abutments would each be located about 3 m further from the centre of the river than the existing abutments, so that the new approach spans will each be about 15 m, assuming that this would be acceptable from the standpoint of the geotechnical factors involved. The existing grade and width of Highway 17 and the present skewed crossing of the Little Sturgeon River would be maintained. This is of some significance because, aside from possible temporary changes during construction, the approach embankments beyond the new abutments will remain essentially the same as at present.

It is understood that the new Bridge would be of steel plate girder construction with a reinforced concrete deck. Both continuous and simply-supported alternatives are under consideration although there is a preference for continuous spans providing that differential settlements are within tolerable limits.

GEOCON

5.0 DISCUSSION (continued)

Anticipated bridge loads are about 6000 kN at each of the new Piers. The loads on the abutments have not been established, and pending more accurate data being available from Designers, have been assumed to be 3000 kN for purposes of this report. During construction, a temporary detour would utilize a Bailey bridge over the Little Sturgeon River about 18 metres north of the present centreline of Highway 17.

Subsurface conditions at the proposed Bridge site consists of a near-horizontal sequence of stratified silty clays, clayey silt and silts from original ground surface to a depth of some 49 m. The consistency of these soils ranges from soft to very stiff, generally firm to stiff, with an apparent trend of increasing strength with depth. The majority of the overburden soils are varved and thus comprise repeated layers of clay, clayey silt and silt. These are in turn underlain by till. At the existing abutments, the natural soils are overlain by up to 6 m of embankment fill which consists predominantly of silt sized particles supporting the present Highway 17 pavement structure.

In the discussion which follows, geotechnical factors pertinent to support of the Bridge on either friction piles within the cohesive strata, or piles end-bearing in the basal till stratum, are considered.

5.1 Performance of Existing Bridge

From a geotechnical engineering standpoint, it is important to examine available evidence on the performance of the existing approach embankments and Bridge, assuming the latter to be carried on timber piles throughout. From site observations, the abutments show evidence of settlement and rotation along the lines observed for other pile-supported bridge structures founded in cohesive soils, e.g. Stermac et al.⁽⁴⁾ The reasons for the 100 mm to 150 mm thick shims together with bending of

(4) Stermac, A.G.; Devata, M.; Selby, K.C., 1968. "Unusual Movements of Abutments Supported on End-Bearing Piles". Canadian Geotechnical Journal, Vol. V., No. 2, pp. 69-79, and Discussion by L. Bjerrum, 1969 C.G.J. Vol. No. pp. 366-367

5.0 DISCUSSION (continued)

5.1 Performance of Existing Bridge (continued)

anchor bolts at the top of the pier pedestals on the west side of the existing Bridge are not clear. The observed abutment rotation may have been partly responsible. On the other hand, shimming may have been required to compensate for settlement of the Pier, or both factors may have been pertinent. No similar shimming exists on the Pier on the east side of the Bridge. Significantly, the main difference in soil conditions between the two sides of the Bridge, is that there appears to have been a considerably greater amount of fill involved in building the west approach embankment as a whole, than the east approach embankment. About 6 m of silt fill was encountered at Borehole 1 in the area of the west embankment, while predominantly natural overburden was found at Borehole 4 in the area of the east embankment. This difference in loading would have generated different settlements at the two abutments, because otherwise the subsoil conditions at each abutment are closely similar.

The history of the approach embankments has been inferred from a topographic plan⁽³⁾ of the area prior to construction of the existing Bridge. On the west side of the river, it appears that a deep gully had existed with an original ground level at about elevation 196. It is inferred that to form the west approach embankment, a concrete box culvert was installed and the gully partly backfilled with locally excavated borrow to pavement subgrade elevation. The existing west abutment pile cap was constructed on the river bank where the original ground surface had an average elevation of 199.

On the east side of the river, a Canadian Pacific Railroad Section House was previously located at Borehole 4. The original ground surface at this location, and the table land immediately east of the river was generally at about elevation 201. Thus there would have been only 2 to 3 m of fill required to build up the east approach in this area. The east abutment pile cap was

5.0 DISCUSSION (continued)

5.1 Performance of Existing Bridge (continued)

constructed on the river bank where the original ground surface was at about elevation 198. The above observations, in conjunction with the longitudinal section shown on Reference Drawing⁽³⁾ show the significant differences in overall fill loading from the approach embankments, which would have affected the west and east abutments, respectively.

5.2 Stress History and Shear Strength Results

The subsoil stress history, interpreted from topographic evidence and oedometer test results is discussed below as it relates to the undrained shear strengths obtained for the cohesive strata.

The oedometer tests on samples from Boreholes 1 and 4 indicate that the cohesive strata are slightly overconsolidated with respect to present ground level. This apparent preconsolidation could be attributed to a number of factors including the aging of the glaciolacustrine cohesive strata⁽⁵⁾, cementation bonds⁽⁶⁾, erosion and weathering effects and sample disturbance. The overconsolidation ratio i.e. the ratio of the apparent preconsolidation pressure, P_c , to the in-situ effective overburden pressure, P_o , of the samples from Boreholes 1 and 4 range from about 1.25 to 1.6. Since the glaciolacustrine strata were likely deposited with the same upper surface elevation, and the original gulley near Borehole 1 is an erosional feature which developed after deposition, it is reasonable to expect similar apparent preconsolidation pressures in Boreholes 1 and 4,

(5) Bjerrum, L., 1972. "Embankments on Soft Ground". A.S.C.E., Proceedings of the Specialty Conference on Performance of Earth Supported Structures. Purdue University, June 1972, Vol. II, pp 1-54.

(6) Quigley, R.M. and Ogunbadejo, T.A., 1972. "Clay Fabric and Oedometer Consolidation of a Soft Varved Clay". Canadian Geotechnical Journal, Vol. 9, No. 2, pp. 165 to 175.

5.0 DISCUSSION (continued)

5.2 Stress History and Shear Strength Results (continued)

with the ground surface at Borehole 1 restored to the approximate post-deposition upper surface elevation. The overconsolidation ratio of the samples from Borehole 1 would have been higher had Borehole 1 been put down prior to placement of the embankment fill. Considering the mode of deposition and surrounding topography, it is probable that the clays have not been subjected to effective stresses greater than the present. Thus they would be normally consolidated with respect to present overburden pressure. The apparent preconsolidation and the overconsolidation ratios of the samples tested from Boreholes 1 and 4 can therefore possibly be explained by aging of the clay⁽⁵⁾, or the clay having a flocculated structure with cementation bonds⁽⁶⁾. The higher overconsolidation ratios found for the samples tested from Boreholes 5 and 6 reflect mainly the bank erosion during the cutting of the Little Sturgeon River channel. However, the apparent preconsolidation pressures of these samples are similar to those samples from comparable elevations at Boreholes 1 and 4. On the basis of the above, it is inferred that (i) the clays at the site have an apparent preconsolidation mainly attributable to the process of aging and, in the varved clays in particular, formation of cementation bonds early in the geologic history of the deposit, (ii) the pc/po ratio is consistent with published results for aged clays with similar plasticities if erosional effects are neglected, (iii) the fill placed for the west approach embankment has likely not resulted in significant preconsolidation of the underlying cohesive strata viz. resulting in a higher shear strength due to consolidation effects and (iv) the various cohesive strata should have comparable undrained shear strengths across the site except in areas adjacent or below the river where erosion and redeposition has occurred.

5.0 DISCUSSION (continued)

5.2 Stress History and Shear Strength Results (continued)

The undrained shear strength of the cohesive strata is related to the degree of preconsolidation. Published data for soft clays from Sweden⁽⁷⁾ indicate an undrained shear strength to preconsolidation pressure ratio (S_u/P_c) of 0.20 to 0.25 for clays with similar plasticities and overconsolidation ratios as the cohesive strata at the Little Sturgeon River Bridge site. Experience with the New Liskeard varved clay, which has similar geologic origins as the varved clay at the site indicates, a minimum S_u/p_c ratio of about 0.25, where S_u is taken as the field vane shear strength⁽⁸⁾. The test results at the subject site indicate field vane S_u/p_c ratios ranging between 0.21 and 0.30 but generally between 0.23 to 0.26. The range of S_u/p_c ratios for the laboratory vane and triaxial strength results are 0.14 to 0.22, and 0.10 to 0.21, respectively. The field vane strength results agree with the expected values and the laboratory vane and triaxial strengths are lower than expected. In this regard, the laboratory vane tests were carried out on the soil in the ends of thin-walled tubes and it is known that the strengths obtained are lower than if the soil in the centre of the tube was tested. It is also known from experience that triaxial strengths are lower than strengths obtained by field vane with similar clays elsewhere. It is noted that because the field vane measures the shear strength essentially on a vertical plane, field vanes have been found to measure the maximum shear strength in stratified soils⁽⁹⁾ but the minimum

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- (7) Larsson, Ruff, 1980. "Undrained Shear Strength in Stability Calculation of Embankments and Foundations on Soft Clay". Canadian Geotechnical Journal, Vol. 17, No. 4, National Research Council Canada.
- (8) Lacass, S.; Ladd, Charles and Barsvary, A.K., 1977. "Undrained Behaviour of Embankments on New Liskeard Varved Clay." Canadian Geotechnical Journal, Vol. 14, No. 3, National Research Council Canada.
- (9) Lo, K.Y. and Milligan, V., 1967. "Shear Strength Properties of Two Stratified Clays". Journal of Soil Mechanics and Foundations, Divisions ASCE, Vol. 93, S.M.1.

5.0 DISCUSSION (continued)

5.2 Stress History and Shear Strength Results (continued)

shear strength in normally-consolidated homogeneous soils⁽¹⁰⁾.

For the above reasons careful consideration was given to selection of the undrained shear strengths to be used in analyses.

5.3 Foundations for Bridge

In the light of the subsoil conditions encountered during this investigation, and the performance of the existing Bridge, it is clear that the new Bridge should be founded on piles. The configuration of the pile foundation would be influenced by a number of factors such as (a) tolerable total and differential settlements, (b) effect on stability of existing embankment slopes, (c) comparative economics, and (d) practical construction considerations.

Two alternative pile foundation systems are considered herein from a geotechnical standpoint, viz. either end-bearing piles or friction piles throughout.

5.3.1 End-Bearing Piles

A number of alternative types of end-bearing piles could be considered such as steel H or tube piles. However in view of the considerable depth to the bearing stratum and the desirability of minimising soil disturbance during driving, non-displacement piles of the steel H variety are considered preferable to displacement types. The discussion which follows is therefore based on the use of steel H piles, and specifically the 12BP53 section.

A bearing stratum of glacial till was encountered at the site at a depth of some 49 metres below finished pavement surface of Highway 17. Based on the results of the investigation, it is considered feasible to drive steel H-piles

(10) Delory, F.A. and Salvas, R.J., 1967. "An Investigation of Discrepancies in Undrained Shearing Strength Test Results in Clay", Ontario Joint Highways Research Report No. 45, April 1967.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)5.3.1 End-Bearing Piles (continued)

to adequate end-bearing in the glacial till. Standard Penetration Tests infer that the till is in a very dense condition, "N" values ranging from 50 to greater than 100 to the depths penetrated, although as noted previously, these values can be influenced by coarse sizes in the till greater than the diameter of the sampling spoon. Thus, steel H-piles may be expected to penetrate the till to variable depths before achieving the selected set criteria.

During driving to end-bearing in the till, the piles will have to penetrate through varved clay whose consistency varies with depth from firm to very stiff. For approximately 15 m above the till, the clay is stiff to very stiff, with an undrained shear strength of 100 to 150 kPa. The considerable length of pile involved, and these high shear strengths in the clay above the till, will tend to reduce the effective driving energy reaching the pile tip and producing end-bearing in the till. For purposes of preliminary design, it is recommended that a working load of 535 kN (60 tons) be used for a 12BP53 H-pile, based on carrying capacity considerations alone. Also that a minimum driving energy of 43.0 kilojoules (kJ) (32,000 foot pounds) be used in conjunction with a final set of 15 blows per 25 mm. It is also recommended that a representative pile be test loaded using cyclic loading procedures along the lines recommended by Van Weele⁽¹¹⁾, to establish the ratio between skin friction and end-bearing operating at the time of the test, and also to provide a basis for selecting pile working loads as influenced by considerations of tolerable settlements for the Bridge superstructure.

(11) Van Weele, A.F. 1957. "A Method of Separating the Bearing Capacity of a Test Pile into Skin Friction and Point Resistance." Proc. 4th I.C.S.M.F.E., Vol. I. pp 76-80.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)5.3.1 End-Bearing Piles (continued)

The till may contain cobble and boulder sizes and thus it is advisable to provide a suitable drive point for the steel H-piles to prevent possible tip damage. Provision should also be made for redriving piles.

Piles driven from the approximate elevation of the existing pile caps will extend through some 45 metres of cohesive soils and as such there will be potential for development of negative skin friction on account of reconsolidation of the clay by dissipation of pore pressures developed during the pile driving. The extent to which negative skin friction should be considered in this case is difficult to predict. However, since (i) the clay is slightly over consolidated, (ii) low displacement piles are proposed, and (iii) only a small amount of additional surcharge around the pile group is involved, (due to greater weight of the new fill as compared to the existing fill), the effects of negative skin friction would likely be small. We would be pleased to review this point with you at time of final design, once the configurations of the pile groups, and time schedules for construction are better known.

In general, it is recommended that all lateral loads on the abutments and piers be resisted by batter piles. If lateral loads are low, say less than 4.5 kN per pile, the possibility of resisting such loads by vertical piles only could be considered.

For single end-bearing piles in granular strata, settlements will normally be complete within a short period of time after the application of the full working load. Total settlement of the pile would consist mainly of elastic compression of the pile, elastic and plastic deformation of soil at the pile tip, and some deformation of the clay soils surrounding the pile during reconsolidation.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)5.3.1 End-Bearing Piles (continued)

The effects of group action significantly alter the magnitude of settlement of the end-bearing pile groups, (as compared to settlements of single piles) and therefore also need to be considered.

In the present case, pile top settlements would not necessarily occur concurrently with application of loads from the Bridge itself. Two time-dependent effects would be present, viz. (i) generation of negative skin friction as the clay around the piles remoulded during pile driving, reconsolidates, (and consolidates under the small additional surcharge load) and (ii) transfer of load carried by the pile by skin friction in the clay strata to end-bearing in the till. It would be timely to review these points after the results of pile load tests are available. For the present, it could be assumed that the working load per pile given earlier would accommodate these effects.

For purposes of this discussion, preliminary estimates of design loads and pile group configurations were made for use in judging the order of magnitude of anticipated settlements of the Bridge. Where end-bearing H-pile groups are used at the abutments and piers, in conjunction with a working load of 535 kN (60 tons) per pile, the magnitude of group settlement is judged to be between 15 and 30 mm, with differential settlements between the abutments and piers of about 10 mm. This is believed to confirm the feasibility of using continuous steel girders for the Bridge in this case. It is suggested that for purposes of preliminary foundation design, the 535 kN (60 tons) pile working load be reduced proportionately if desired to limit total and differential settlements to less than the values given above. It is recommended that the question of anticipated settlements for given pile working loads be reviewed finally on the basis of the results of the pile load tests mentioned earlier.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)5.3.2 Friction Piles

Several different types of friction piles could be used such as steel H-piles, concrete piles or timber piles. For the purposes of this discussion timber piles only have been considered and a length of approximately 16 metres and an average diameter of 0.3 m have been assumed. Such piles driven from about the existing pile cap elevations would penetrate strata of clayey silt, silty clay and some 8 to 10 metres of the upper varved clay stratum.

Undrained shear strengths in the clayey silt stratum are generally in the range 25 to 80 kPa, in the silty clay between 25 and 50 kPa, and in the top portion of the varved stratum generally between 35 and 60 kPa. At the abutments the average undrained shear strength along the length of a 16 metre long timber pile is about 45 kPa. Using an adhesion value of 0.8 times the undrained shear strength, as per Tomlinson⁽¹²⁾, and static formulae, it is estimated that an allowable load of about 175 kN (920 tons) per pile could be used in preliminary design of individual timber friction piles. Considering negative skin friction as discussed earlier and assuming a computed Factor of Safety of 3.0, the useful working load for piles in the abutment areas would be reduced to about 150 kN (17 tons) per pile, based on procedures given in the Canadian Foundation Engineering Manual⁽¹³⁾. Similar useful working loads per pile at the pier location would be about 140 kN (16 tons). If load tests are carried out, a Factor of Safety of 2.5 could be used, in which case

(12) Tomlinson, M.J., 1957. "The Adhesion of Piles Driven in Clay Soils". Proceedings of the Fourth Int. C.S.M.F.E. London, 1957.

(13) Canadian Foundation Engineering Manual, Part 3, March 1978. The Canadian Geotechnical Society.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)5.3.2 Friction Piles (continued)

the above pile working loads could be adjusted accordingly. For a given group of piles, it is important that the capacity of the group be selected to give reserve safety against possible shear around the periphery of the group, and failure of the group as a whole in end-bearing. Such considerations may control the load placed on individual piles within the group.

For purposes of preliminary settlement analyses the pile spacing in friction piles groups was assumed to be between three and five times the average diameter 'd' of the pile. At the abutments about 25 piles would be required, and assuming a pile spacing of 4 diameters one way and 5 diameters the other, the group dimensions would be about 4 m by 10 m. Similarly, at the piers, 40 piles were assumed on a 3d by 4d grid, giving group dimensions again of about 4 m by 10 m.

Settlements of friction piles in clay are caused by elastic compression of the piles, elastic and plastic settlements of the clay around the piles and below the pile group, and shear deformation or creep in clay at the wall of the pile. For purposes of this discussion, preliminary estimates of pile group settlements were made. Consolidation settlements of the abutment pile groups computed using the method suggested by Terzaghi and Peck⁽¹⁴⁾ (which assumes spread of load at 1 horizontal to 2 vertical in the soil beginning at two thirds the length of the piles) were about 110 mm, whereas at the piers the group settlement was similarly computed to be about 80 mm. These computations are somewhat approximate, and in any event are subject to review on the basis of the

(14) Terzaghi, K. and Peck, R.B., 1948. "Soil Mechanics in Engineering Practice", J. Wiley and Sons Inc., New York.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)5.3.2 Friction Piles (continued)

final configuration of the Bridge, and the actual loads which will be applied at the abutments and piers.

To estimate the probable magnitude of elastic settlement of the pile groups, some case histories from the published technical literature were examined. These included a program of static pile load tests carried out by the M.T.C. on timber piles in normally consolidated to slightly over consolidated varved clays of Ontario as reported by Milligan et al⁽¹⁵⁾. From such test results it is inferred that immediate elastic settlements of the pile groups at the Little Sturgeon River would be between 15 and 20 mm at working loads of about 175 kN (Mattes and Davis⁽¹⁶⁾ and Poulos⁽¹⁷⁾).

In the case of the Little Sturgeon River Bridge, the differential elastic settlements which occur after the steel beams are made continuous and the concrete deck is in place, is of importance for design. For preliminary design purposes the value of immediate elastic settlement (suitably adjusted to reflect the proportion of load applied after continuity is established in the beams) and the estimated consolidation settlements given earlier may be used. The range of differential settlements between pier and abutment is probably such as to preclude the use of continuous girder design. Consequently, friction piles would appear suitable only if a simply-supported

(15) Milligan, V.; Soderman, L.G. and Rutka, A., 1962. "Experience with Canadian Varved Clays", Proc. Am. Soc. Civ. Eng., Vol. 88, SM4, pp. 31 to 37.

(16) Mattes, N.S. and Poulos, H.G., 1969. "Settlement of Single Compressible Pile". J.S.M.F.D., A.S.C.E., Vol. 95, SMI, pp 189-207.

(17) Poulos, H.G. 1968. "Analysis of the Settlement of Pile Groups" Geotechnique Vol. 18, pp. 449-47.

5.0 DISCUSSION (continued)

5.3 Foundations for Bridges (continued)

5.3.2 Friction Piles (continued)

design was adopted for the Bridge. In this case, the settlements given earlier could be used as a guide to establish the amount of shimming that may be required to maintain the Bridge deck level. For final design of a structure on friction piles, it would be desirable to test a representative pile so that more precise settlement estimates can be made by methods suggested by Poulos⁽¹⁷⁾. Additionally, there would be merit in driving several representative piles in a group, and using piezometers to obtain information regarding the buildup and dissipation of excess porewater pressures in the surrounding soil of the slope. These measurements could be used, in conjunction with effective stress stability analyses to determine more accurately the stability of the approach embankments during and after pile driving.

It is recommended that timber piles be driven using the guidelines given in the Canadian Foundation Engineering Manual (Part 3)⁽¹³⁾.

As discussed for end-bearing piles, lateral loads should be handled by incorporating battered piles in the pile groups. Typically, a batter of 4 vertical to 1 horizontal could be employed.

5.4 Stability Considerations

The stability of the abutment slopes of the existing Bridge for different conditions of loading are described below.

The shear strength profile used in stability analyses was developed using the field vane strengths adjusted by a correction factor proposed by Bjerrum.⁽⁵⁾ The factor applied successfully to field vane strengths for the analysis of the New Liskeard failure⁽⁸⁾ was 0.89 based on the bulk plasticity of the varved

5.0 DISCUSSION (continued)

5.4 Stability Considerations (continued)

clay. The value of 0.90 was therefore used to adjust the field vane strengths at the Bridge site. The generalized undrained shear strength design profile used in the analyses of the embankments is given in Table No. 1 below and on Figure 11 at the rear of the report. It is noted that down to about elevation 187.5, it is inferred that the clayey silt and silty clay strata are soft having an undrained shear strength of 25 kPa. Additionally, the thickness of the clayey silt stratum at Borehole 4 is greater than at Borehole 1, with the upper surface at about elevation 200 rather than 196 as given for Borehole 1.

TABLE NO. 1
Geotechnical Parameters
Used in Stability Analyses

<u>Soil Type</u>	<u>Elevation (metres)</u>	<u>Undrained Shear Strength (kPa)</u>	<u>Bulk Unit Weight₃ (kN/m³)</u>
Silt Fill	202.5 to 196	19.0	18.8
Grey Clayey *			
Silt BH1	196 to 192		
BH4	200 to 192	49.0	18.1
Dark Grey *			
Silty Clay	192 to 189	33.5	17.2
Upper Varved Clay	189 to 183	43.0	16.3
Upper Varved Clay	183 to 180	52.5	16.3
Upper Varved Clay	180 to 170	72.0	16.3

*Note: Below river bottom, an undrained shear strength of 25kPa was assigned to the strata to elevation 187.5.

The critical embankment section at the site in terms of steepness of slope, is along the Highway centreline at the existing east abutment. However, the least favourable conditions from a stability standpoint, occur at the west abutment where there is

5.0 DISCUSSION (continued)

5.4 Stability Considerations (continued)

the greatest thickness of fill. The west abutment was first analysed assuming the failure surface passes below the estimated tip of the existing timber piles. External loads approximating the dead weight of the existing Bridge acting at the abutment and the pier, were incorporated in the analyses. The minimum Factor of Safety computed using the ICES LEASE I computer programme⁽¹⁸⁾ for this case was about 1.5.

The west abutment end slope was then analysed for the condition where there would be no Bridge deck loading, as would be the case just prior to pile driving for the new Bridge. The critical failure surface was not limited to depths below existing pile tips, and no shear strength was assumed for the existing friction piles where they intersected theoretical slip planes. The minimum computed Factor of Safety for an assumed circular slip surface passing through the silty clay stratum was about 1.4 as shown on Figure 12, for Circle 1.

During driving of piles for the new Bridge, a temporary reduction in shear strength will occur in the vicinity of the piles due to increase in pore water pressure in the clay subsoil and remoulding of same. The extent of strength reduction is dependent on such factors as pile spacing and sensitivity of the clay. With time, there will be strength regain due to pore pressure dissipation, and thixotropic and consolidation effects. As much as 50 percent dissipation of the excess pore pressure may take place between 1 and 10 days after driving, depending on the size and other details of the pile group⁽¹⁹⁾. The pattern

(18) Bailey, W.A. and Christian J.T., 1969 "ICES-LEASE I, A Problem Oriented Language for Slope Stability Analysis, Users Manual". M.I.T. Research Report R69-22, Department of Civil Engineering, pp. 567

(19) Blanchet, R.; Tavenas, F. and Garneau, R. 1980. "Behaviour of Friction Piles in Soft Sensitive Clays". Canadian Geotechnical Journal, Vol. 17, No. 2, National Research Council of Canada.

5.0 DISCUSSION (continued)

5.4 Stability Considerations (continued)

of pore-pressures set up in clay by piles driven into it depends on such factors as the type of pile (i.e. whether displacement or non-displacement type) the pile material (whether wood, steel or concrete), on pile spacing as well as the characteristics of the clay. This pattern is difficult to predict precisely, and for present purposes the simplifying assumption was made that during construction, there would be a 50 percent reduction in shear strength in the soil below the new abutments in the immediate vicinity of the pile group.

The stability of the existing slope during driving of the original bridge piles (circa 1937) was similarly estimated assuming a 50 percent strength reduction in the zone of soil affected by piling. The minimum computed Factor of Safety so obtained was about 1.3 for Slip Circle 1 on Figure 12.

The geometry of the embankment was then altered to represent conditions during construction, assuming temporary excavations would be required for pile driving at the new abutments as shown on Figure 12. Piles were assumed driven at the location of the proposed abutment i.e. 15 metres inland of the proposed river piers. Within the pile group, the soil was assumed disturbed with an undrained shear strength immediately after driving, of 50 percent of the natural strength. The strength of the piles in shear was ignored, bridge loads were removed, and circular failure surfaces were considered. The minimum computed Factor of Safety for this case was 1.60, for Slip Circle 4 on Figure 12.

The next case studied assumed the new abutment to be constructed and backfilled to existing road level, 50 percent soil strength disturbance within the abutment pile group. The minimum computed Factor of Safety for this case decreased to 1.35 for Circle 2 shown on Figure 12. With complete strength regain, the

5.0 DISCUSSION (continued)

5.4 Stability Considerations (continued)

computed Factor of Safety increased to 1.43 for the same slip surface. By increasing the new approach span to 20 metres (by moving the abutment a further 5 m inland) and assuming the new Bridge abutment constructed, backfilling completed, and a 50 percent reduction in shear strength for the soil in the zone of piling, the minimum computed Factor of Safety increased to 1.5.

The above analyses are approximate in that a number of simplifying assumptions have been used including (i) a two dimensional case for the approach embankment at the abutment, (ii) an arbitrarily selected 50 percent reduction in shear strength due to pile driving, as shown on Figure 12, (iii) Bridge deck loads have not been included, and (iv) the strength of the piles in shear has been neglected. Nevertheless, considering that the existing Bridge has undergone deformation at the abutments, it is considered desirable to design the new Bridge for a higher computed Factor of Safety against instability at the abutments. In this respect, it is recommended that the approach spans of the new Bridge be increased by moving the abutments 8 m further from the river than the existing abutments, which would increase the proposed approach spans from 15 to 20 m.

5.5 General Design and Construction Considerations

- (i) As recommended, the proposed abutments would be located some 8.0 metres landward of the existing abutments. To facilitate construction of the new abutment, the existing abutments and ends of the approach embankments will have to be removed. Temporary slopes cut in the silt fill or clayey silt overburden of no steeper than 2 horizontal to 1 vertical are recommended.
- (ii) Removal of the existing piles may be required to facilitate driving of batter piles for the abutments, and perhaps also vertical piles for the new Piers. The specific locations of the new abutments and piers, should take such possible interference into account.

5.0 DISCUSSION (continued)

5.5 General Design and Construction Considerations (continued)

- (iii) On the basis of the water levels observed in Boreholes 1 and 4, some seepage is expected into the temporary excavations for abutment construction. This seepage could be handled by the procedure of pumping from filter-equipped sumps.
- (iv) The silt fill and clayey silt which will be exposed at the base of the excavations during construction is sensitive, and disturbance should be minimised by such means as placing a work pad of sand and gravel fill.
- (v) A minimum of 1.5 m of earth cover, or other suitable protective measures, should be provided for pile caps for protection against frost action.
- (vi) The abutment should be provided with engineered, non frost-susceptible, pervious backfill and effective drainage. For design, a coefficient of earth pressure at rest (K_o) of 0.50 is recommended in view of the pile support.
- (vii) Depending on the details of the pile support and particularly if friction piles are used, the Piers may require scour protection. This is however, a matter of design.
- (viii) Along the proposed highway detour north of the existing Bridge, temporary shallow approach cuts and timber crib abutments could be constructed to support the ends of the proposed Bailey bridge. The Bailey bridge would probably best be designed as a multiple span structure, in conjunction with approach cuts. Founding could be either on timber cribs, or timber friction piles, or a combination of the two. For design of cribs, the allowable bearing value should not exceed 50 kPa on the river bottom soils. Even at this pressure, significant settlements can be expected, and these could be corrected

5.0 DISCUSSION (continued)

5.5 General Design and Construction Considerations (continued)

(viii) (continued)

if required by periodic levelling. The basis for design of timber friction piles, if used, has been discussed earlier. In selecting abutment details, the design should maintain adequate stability of the existing river banks. For purposes of the necessary analyses, the strength parameters given in the report may be used.

In construction of the Bailey bridge, care should be taken to maintain existing drainage works north of the existing Bridge, on both sides of the river.

6.0 CLOSURE

The field work for this investigation was carried out by Messrs. M. Hudson and G. Parker under the supervision of Mr. R.C. Sansom, P.Eng. This report was written by Messrs. R.C. Sansom, P.Eng. and G. Parker and reviewed by Mr. M.A.J. Matich, P.Eng.

Some sections of the interpretive part of this report are necessarily general, and to be more specific would require further liaison with your Designers as the final scheme evolves for the new Bridge. Should additional geotechnical input be required from us in the form of such liaison, or if we can be of further service otherwise, we would be pleased to oblige.

In closing, may we express our appreciation for the cooperation extended to us by the M.T.C. Personnel involved, throughout all phases of this work.

Yours very truly,
GEOCON (1975) LTD.

G. Parker

G.F. Parker,
Geotechnical Engineer

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APPENDIX I

PROCEDURE AND FIELD EQUIPMENT

GEOCON

1.0 PROCEDURE AND FIELD EQUIPMENT

The field work for this investigation was carried out in the period November 10th to December 21st, 1980. A total of ten boreholes was put down to depths ranging from 1.8 m to 54.9 m for a cumulative depth of about 158 m. In addition, six uncased dynamic cone penetration tests (pentests) were carried out, five of which were adjacent to boreholes. The majority of the pentests were terminated at a refusal of greater than 100 blows per 0.3 m except in the case of Boreholes 5 and 6 where pentests were terminated at the limit of the portable equipment used.

The boreholes at the proposed Bridge approaches were put down using a CME 55 power auger drill and the shallow boreholes on the alignment of the proposed detour route and below the Bridge deck were put down using hand portable motorized tripod equipment. The CEM 55 advanced Boreholes 1 and 4 using hollow stem augers to a depth of approximately 23 m., and by washboring techniques in BW casing below this depth. Washboring techniques in BW casing were used in conjunction with the motorized tripod equipment to advance Boreholes 2, 5 and 6. Boreholes 8, 9, 10 and 11 were shallow boreholes put down along the detour route using the motorized tripod. These shallow boreholes were advanced by continuous sampling by 51 mm O.D. standard split spoon samplers.

Sampling in the deeper boreholes was generally carried out at intervals of not greater than 1.5 m, to depths of about 34 m in Borehole 1 and 23 m in Borehole 4. Below these depths, the sampling interval was increased to approximately 3 m within cohesive soils. In each of the surficial fill, the granular glacial drift (till) stratum which underlies the cohesive soils and, in some cases, the upper few metres of the near-surface silty stratum, samples were recovered using 51 mm O.D. split spoon samplers. Standard Penetration Tests were carried out in conjunction with the split spoon sampling. In the cohesive soils samples were obtained in 51 mm and 75 mm thin-walled Shelby type steel tube samplers.

1.0 PROCEDURE AND FIELD EQUIPMENT (continued)

In-situ shear strengths were determined using a four-bladed 51 mm diameter vane connected to a 12 mm diameter rod coupled to standard A sized drilling rods. The overall length of the vanes was 175 mm, and over the 25 mm from both ends of the vanes, there was a taper from 51 mm to 6 mm diameter. The vane was pushed into the soil 0.45 m below the depth previously reached by the thin-walled tube samplers. Torque was applied to the vane using a torque wrench. Failure generally occurred within one minute from start of application of the force. The field vane was then rotated a minimum of 10 revolutions and a second test carried out to obtain the soil shear strength in a remoulded condition. In addition, a Geonor miniature laboratory vane was used in the bottom of the thin-walled steel tube samplers for measurements of undrained shear strengths in the field subsequent to sampling.

For later observations of groundwater levels, one piezometer was installed in Borehole 1 and one in Borehole 2 and a perforated plastic standpipe was placed in Borehole 1.

Borehole ground surface elevations and locations were surveyed by M.T.C. personnel. Ground surface elevations are understood to be referenced to Geodetic Datum.

Work carried out on Highway 17 requiring traffic control was supervised by M.T.C. personnel.

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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_f	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 ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

APPENDIX II

FIGURES - LABORATORY TEST RESULTS

GEOCON

RECORD OF BOREHOLE No 1

METRIC

W P 145-80-01 LOCATION STA. 10+812.8 o/s 3.9m RT. & HWY. 17 ORIGINATED BY M.H.
DIST 13 HWY 17 BOREHOLE TYPE CME 55 TRUCK MOUNTED DRILL RIG - HOLLOW STEM AUGER - COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 11 12 TO 1980 12 01 BW CASING CHECKED BY R.C.S.

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 Y 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					
202.6	ROAD LEVEL																
202.0	Sand and Gravel 450																
0.6	Fill - Silt, Some Clay, Trace Sand, Gravel and Organics Loose to Compact		2	SS	21		WL	80	12	21							
			3	SS	5		WL	80	12	21							
			4	SS	6												
			5	TW	PH												
			6	TW	PH												
196.0	Grey and Brown																
6.6	Clayey Silt Occasional lamina of silt, trace organics.		7	TW	PH												
192.8	Firm Grey and Brown		8	TW	PH												
9.8	Silty clay, occasional pockets or lamina of silt and fine sand		9	TW	PH												
189.8	Firm Dark Grey		10	TW	PH												
12.8			11	TW	PH												
	Silty clay, clayey silt and silt-varved		12	TW	PH												
			13	TW	PH												
	Firm to Stiff		14	TW	PH												
			15	TW	PH												
	Light and Dark Grey Layers		16	TW	PH												
			17	TW	PH												
			18	TW	PH												
			19	TW	PH												
			20	TW	PH												
			21	TW	PH												
			22	TW	PH												
			23	TW	PH												
			24	TW	PH												
167.2																	
35.4	Silty clay, clayey silt and silt-varved		25	TW	PH												
			26	TW	PH												
	Stiff to Very Stiff																
			27	TW	PH												
	Light Grey, Dark Grey and Brown		28	TW	PH												
153.1			29	TW	PH												
49.5	Sand, trace gravel (Till)		30	SS	79												
150.6																	
52.0	END OF BOREHOLE																
	(a) Asphalt 150																

Note: Pentest location 15 metres
east of Borehole

SEE FIGURES
FOR
CONSOLIDATION
TEST
RESULTS

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

METRIC

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+847.1 o/s 3.2m RT. Ø HWY. 17 ORIGINATED BY G.F.P.
 DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING, BW CASING COMPILED BY M.C.Z.
 DATUM GEODETIC DATE 1980 12 12 to 1980 12 18 CHECKED BY R.C.S.

[illegible]

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



METRIC

LITTLE STURGEON RIVER BRIDGE

W P 145-80-01 LOCATION STA. 10+869.2 o/s 3.5m LT. Ø HWY. 17 ORIGINATED BY G.F.P.

DIST 13 HWY 17 BOREHOLE TYPE CME 55 TRUCK MOUNTED DRILL RIG - HOLLOW STEM AUGER - COMPILED BY P.A.D.

DATUM GEODETIC DATE 1980 12 19 BW CASING CHECKED BY R.C.S.

+3, x5: Numbers refer to Sensitivity



RECORD OF BOREHOLE No 4

METRIC

W P 145-80-01

LOCATION STA. 10+878.1 o/s LT. 19.6m W HWY. 17

ORIGINATED BY G.F.P.

DIST 13 HWY 17

BOREHOLE TYPE CME 55 BOMBARDIER MOUNTED DRILL RIG - HOLLOW STEM

COMPILED BY M.C.Z.

DATUM GEODETIC

DATE 1980 12 06 TO 1980 12 09

AUGER - BW CASING

CHECKED BY R.C.S.

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 Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100				
202.5	GROUND LEVEL														
199.7	Fill - sand and gravel Brown														
2.8			1	SS	5		80	12	21						
			2	TW	PH										
	Silt to Clayey Silt. Occasional lamina of silt and fine sand, trace of organics		3	TW	PM										
192.2	Firm Brown and Grey		4	TW	PM										
10.3	Silty clay, occasional pockets and thin lamina of silt and fine sand		5	TW	PM										
189.4	Firm Dark Grey		6	TW	PH										
13.1			7	TW	PM										
	Silty clay, clayey silt and silt - varved		8	TW	PM										
	Firm to Stiff		9ab	SS	-										
			10	TW	PH										
			11	TW	PH										
	Light and Dark Grey Layers		12	TW	PH										
			13	TW	PH										
			14	TW	PH										
			15	TW	PH										
166.5															
36.0															
	Silty clay, clayey silt and silt - varved		16	TW	PH										
	Stiff														
	Light Grey, Dark Grey and Brown		17	TW	PH										
			18	TW	PH										
153.7			19	SS	100/0.02m										
48.8			20	SS	51										
	Sand, some silt, traces gravel and clay (Till)		21	SS	90										
			22	SS	50										
			23	SS	17.5										
			24	WS	-										
147.6															
54.9	END OF BOREHOLE														

Note: 1. Increase in penetration resistance at approximately El. 187, Overnight delay.
2. Pentest location 2 metres east of Borehole.

+3, x5 : Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

METRIC

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+878.1 o/s LT. 19.6m @ HWY. 17 ORIGINATED BY M.H.
 DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
 DATUM GEODETIC DATE 1980 11 20 CHECKED BY R.C.S.

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					
198.3	GROUND LEVEL											
0.0	Clayey Silt. trace organics		1	TW	PM							
	Firm to Stiff		2	TW	PM							
			3	TW	PM							
	Grey and Brown		4	TW	PM							
			5	TW	PM							
191.1			6	TW	PM							
7.2	Silty clay		7	TW	PM							
187.8	Firm to Stiff Dark Grey		8	TW	PM							
10.5	END OF BOREHOLE											
						Note:	Pentest location 2 metres east of Borehole.					

+3, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 6

METRIC

W P 145-80-01 LOCATION STA. 10+832.2 o/s 17.0m LT. Q HWY. 17 ORIGINATED BY M.H.
 DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
 DATUM GEODETIC DATE 1980 11 11 to 1980 11 13 CHECKED BY R.C.S.

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	50 100 150 200 250					
197.7	GROUND LEVEL													
0.0	Clayey Silt. Firm to Stiff		1	TW	PM		195							
			2	TW	PM									
			3	TW	PM									
	Grey		4	TW	PM									
			5	TW	PM									
191.5	Silty clay		6	TW	PM		190							
6.2	Firm Dark Grey		7	TW	PM									
188.8	(a)		8	TW	PM									
8.9														
187.5	END OF BOREHOLE													
10.2	(a) Silty clay, clayey silt and silt - varved Firm Light and Dark Grey Layers													

+3, x5 : Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 8

METRIC

W P 145-80-01 LOCATION STA. 10+782.0 o/s 8.0m LT. @ HWY. 17 ORIGINATED BY M.H.
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 11 14 CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES								
202.4	(a) GROUND LEVEL		1	SS	1.5								
0.0	(b)		2	SS	1.5								
1.3	(c)		3	SS	1.5		200						
2.4	END OF BOREHOLE												
	(a) Topsoil 150												
	(b) Fill - silty sand and gravel, trace organics Loose to Compact Brown												
	(c) Silt Very Loose Brown to Grey												

RECORD OF BOREHOLE No 9

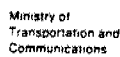
METRIC

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+800.7 o/s 10.6m LT. & HWY. 17
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING
DENUM GEODETIC DATE 1980 11 14
ORIGINATED BY M.H.
COMPILED BY M.C.Z.
CHECKED BY R.C.S.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
201.9	(a) GROUND LEVEL															
0.0	(b)	X	1	SS												
0.7	Sandy silt		2	SS												
	Very Loose Grey		3	SS												
			4	SS												
2.4	END OF BOREHOLE															
	(a) Topsoil 100															
	(b) Fill - silty sand, trace gravel and organics															

+3, x5 : Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



METRIC

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE
STA. 10+911.9 o/s 16.4m LT. @ HWY. 17 ORIGINATED BY M.H.
DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
DATUM GEODETIC DATE 1980 11 14 CHECKED BY R.C.S.

[illegible]

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 11

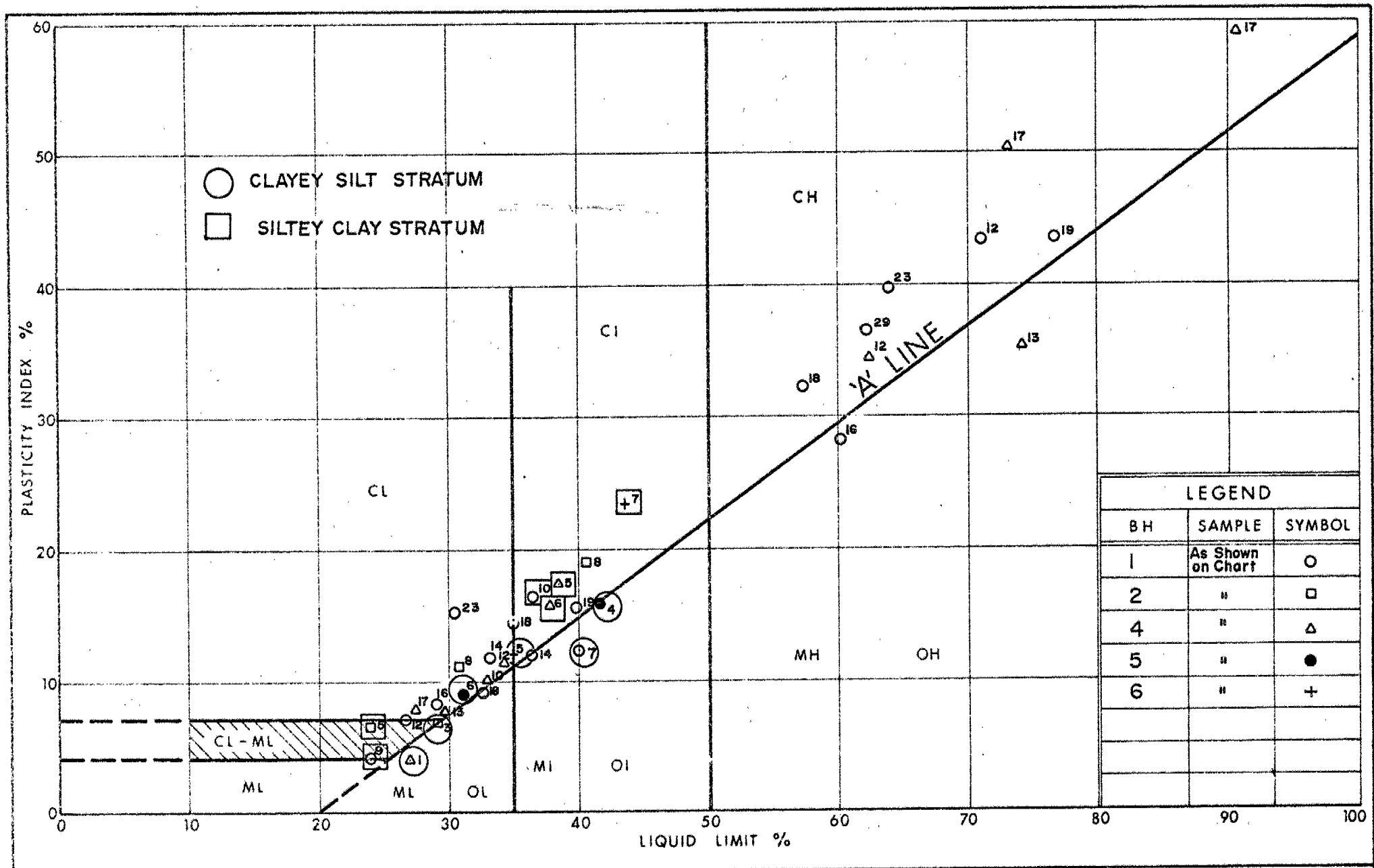
METRIC

W P 145-80-01 LOCATION LITTLE STURGEON RIVER BRIDGE ORIGINATED BY M.H.
 DIST 13 HWY 17 BOREHOLE TYPE MOTORIZED TRIPOD - WASH BORING - BW CASING COMPILED BY M.C.Z.
 DATUM GEODETIC DATE 1980 11 14 CHECKED BY R.C.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (EST)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH								
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				WATER CONTENT (%)						
200.0	GROUND LEVEL															
0.0	(a)	XX	1	SS	9											
0.8	(b)		2	SS	17											
1.8	END OF BOREHOLE															
	(a) Fill - sand and gravel															
	Loose															
	Brown															
	(b) Silt															
	Compact															
	Brown															

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



Ontario

Ministry of
Transportation and
Communications

PLASTICITY CHART

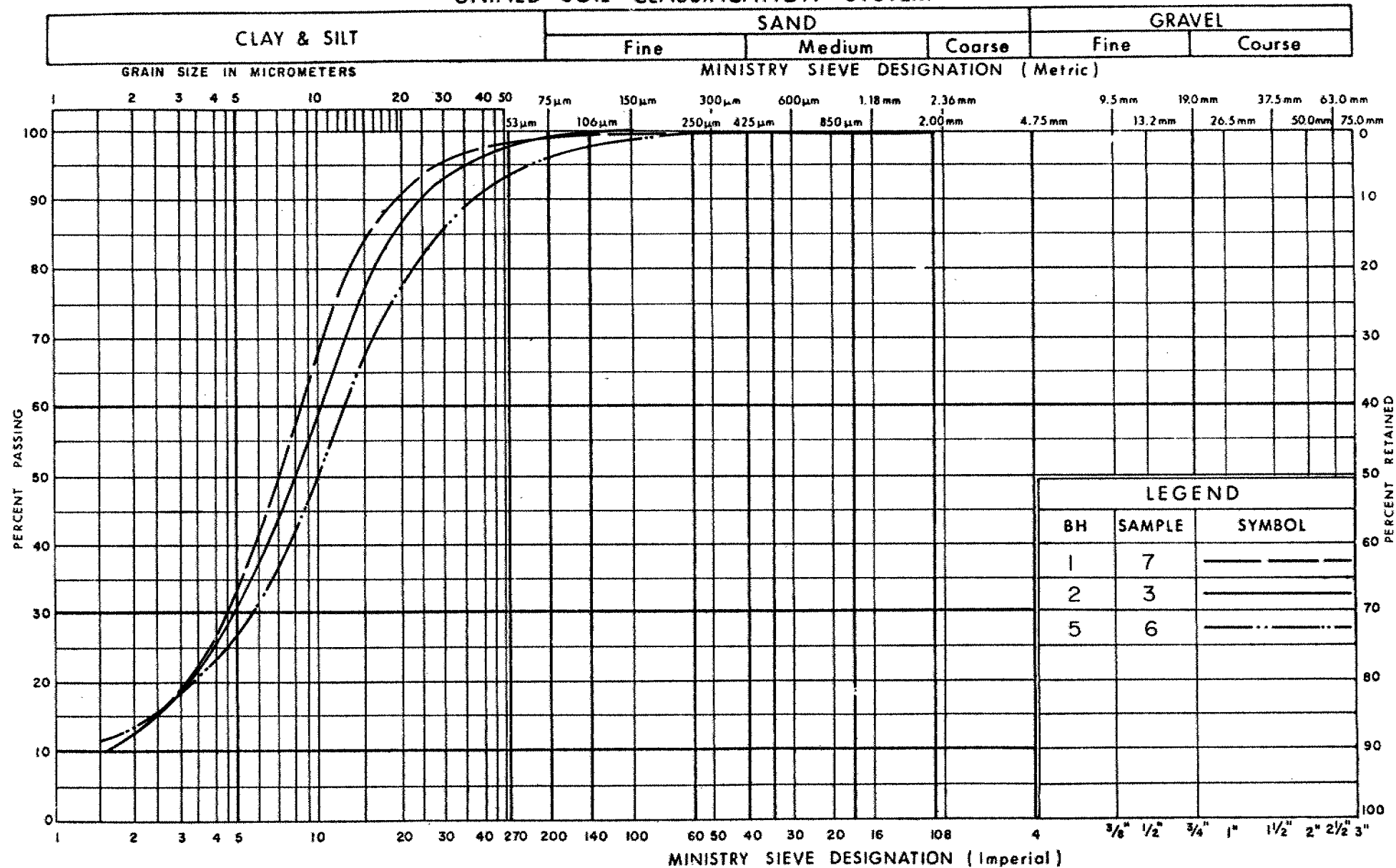
COMBINED TEST RESULTS ON SAMPLES OF COHESIVE STRATA

NOTE Where sample numbers are duplicated, tests carried out on separated varve layers.

FIG No 1

W P 145-80-01

UNIFIED SOIL CLASSIFICATION SYSTEM



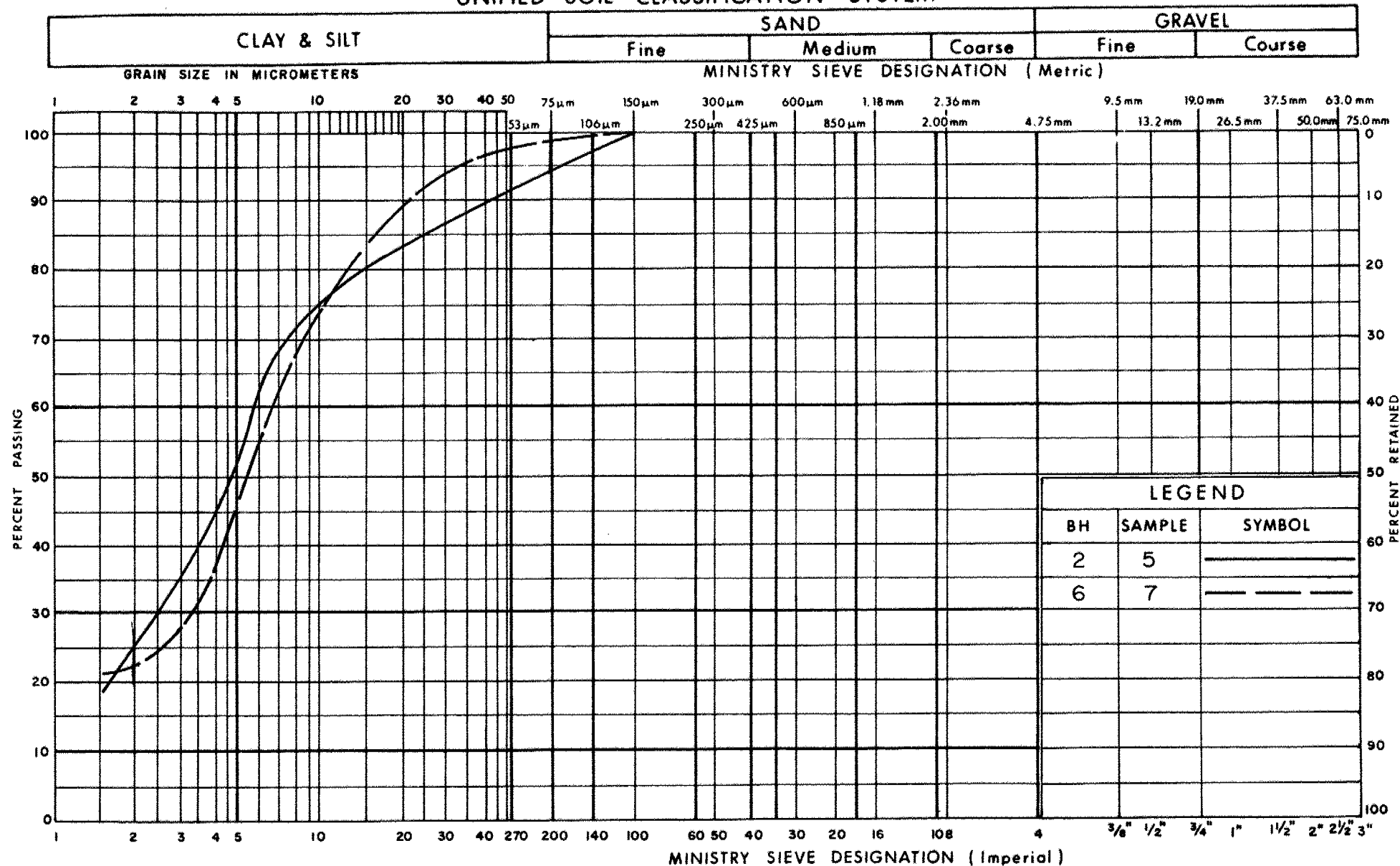
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
CLAYEY SILT
(CL-ML to CI)

FIG No 2

W P 145-80-01

UNIFIED SOIL CLASSIFICATION SYSTEM



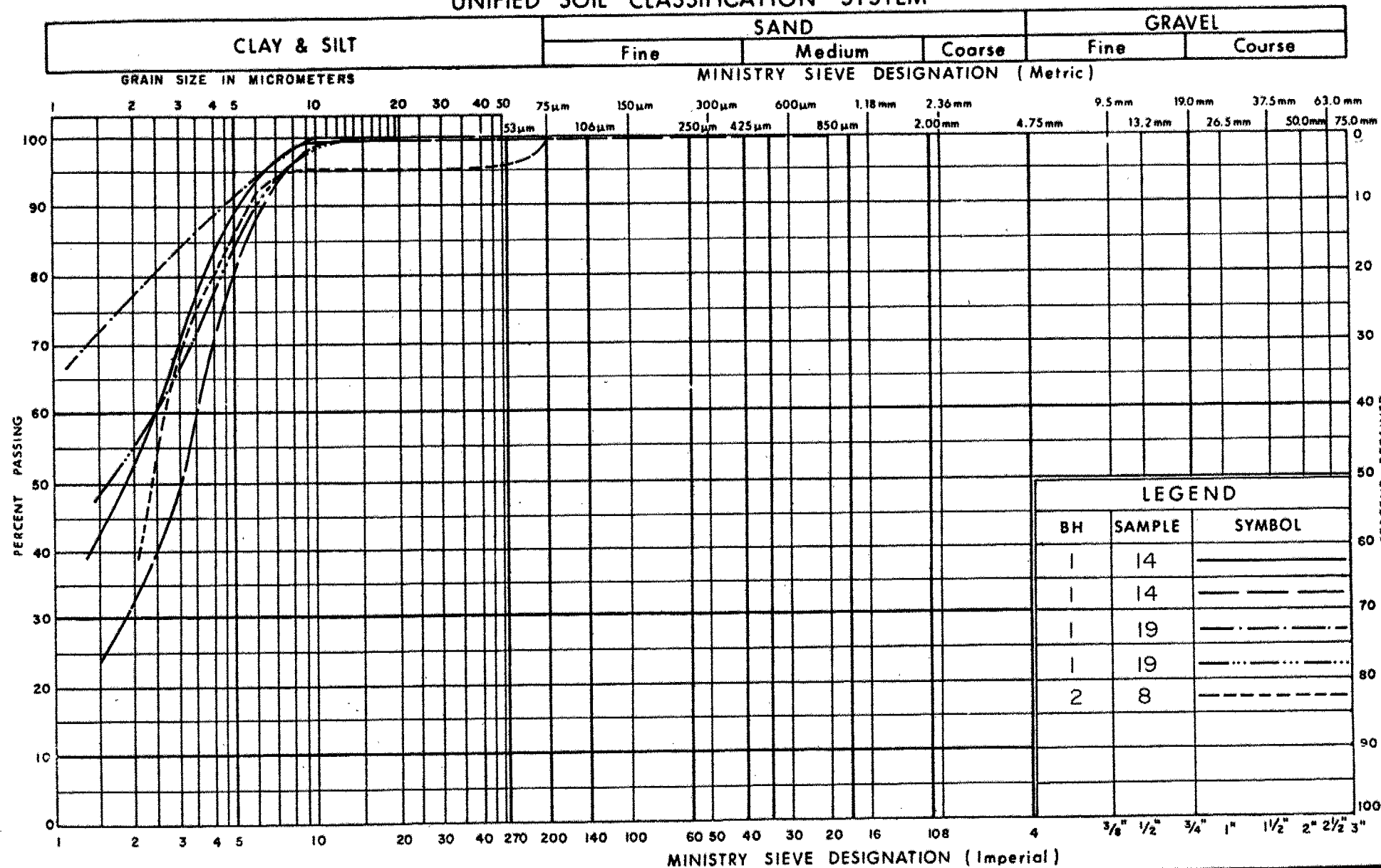
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAYEY SILT
(CI)

FIG No 3

W P 145 - 80 - 01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION

SILTY CLAY & CLAYEY SILT

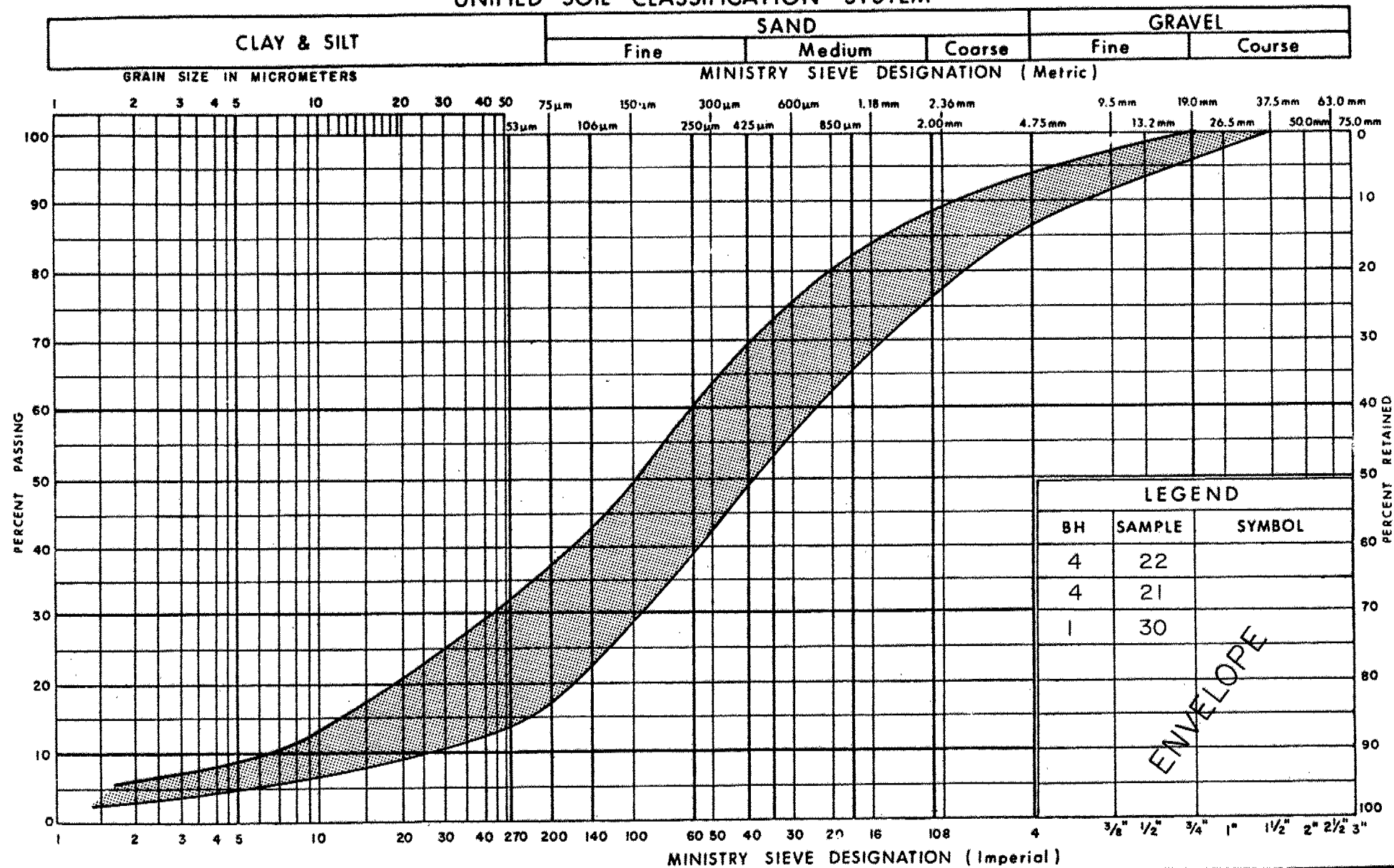
(CL, CH, ML)

NOTE: Tests carried on separated varve layers

FIG No 4

W P 145-80-01

UNIFIED SOIL CLASSIFICATION SYSTEM



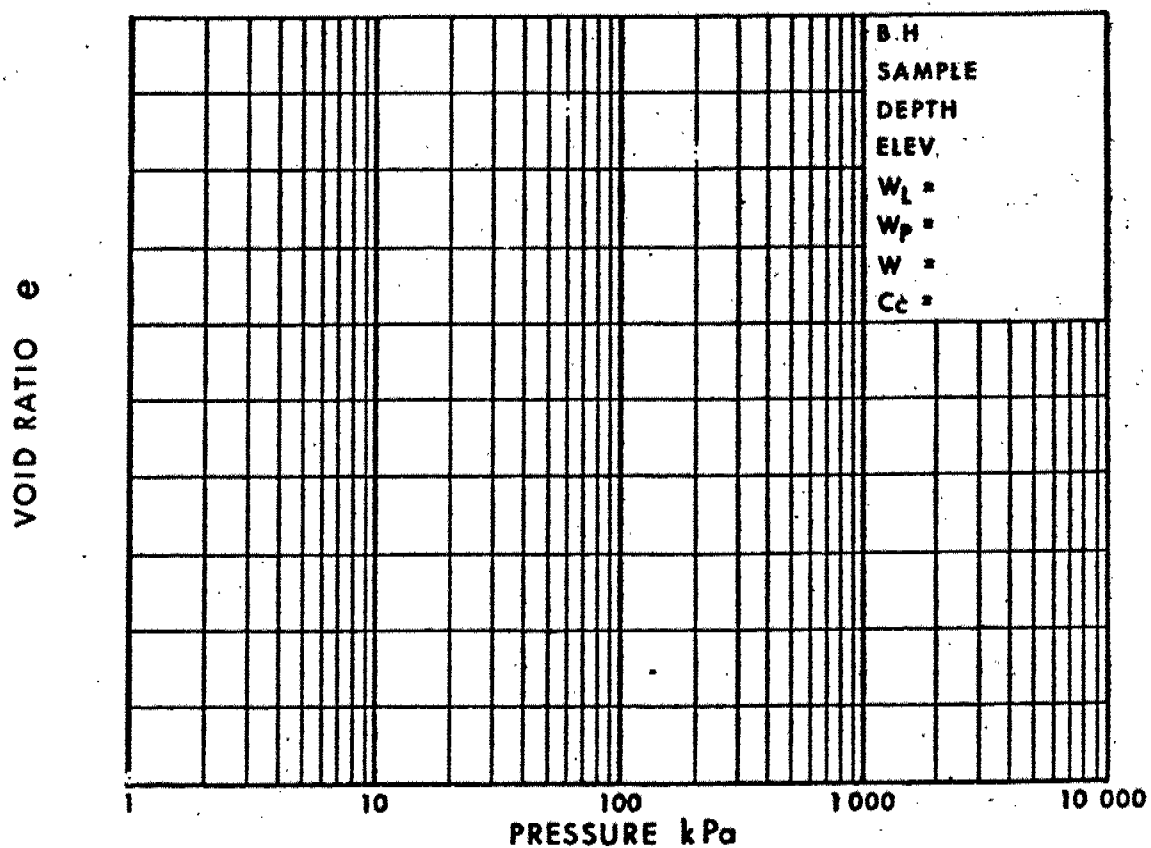
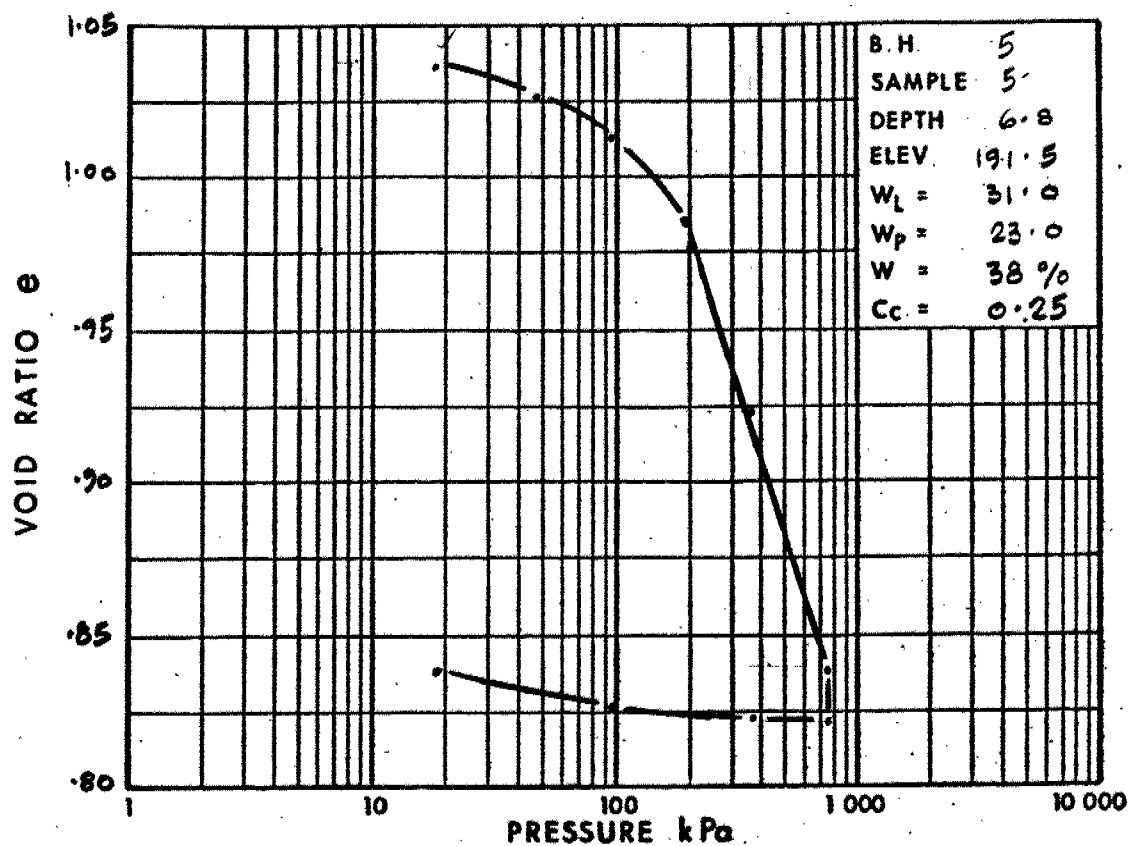
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SAND, TRACE TO SOME GRAVEL, SILT & CLAY (TILL)

FIG No 5

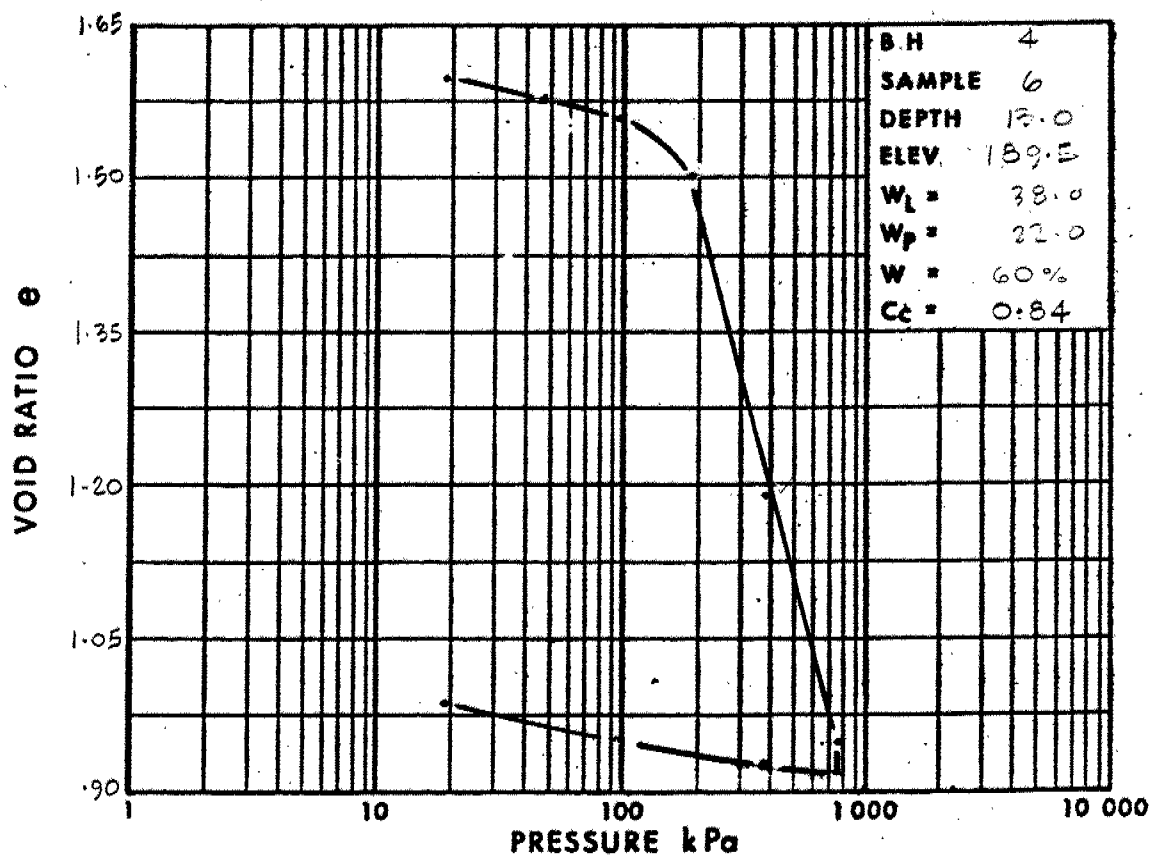
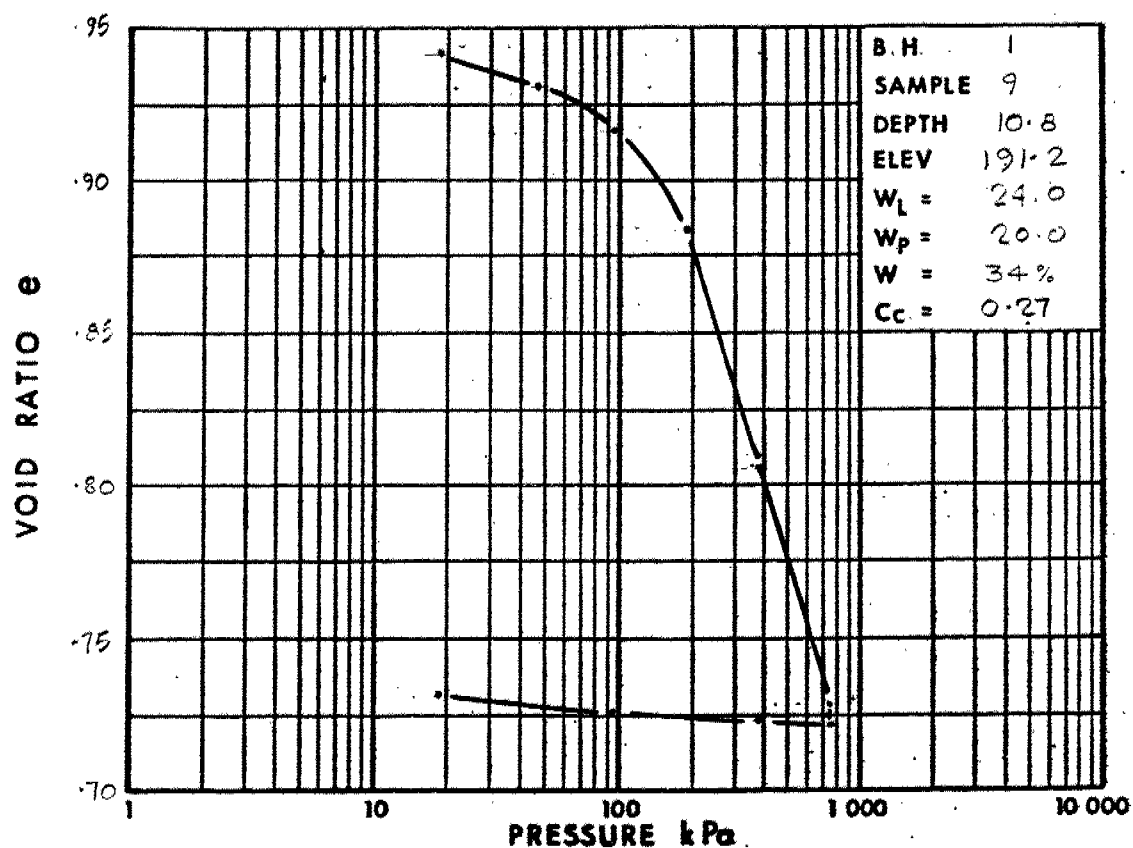
W P 145-80-01

VOID RATIO - PRESSURE CURVES



W P 145-80-01

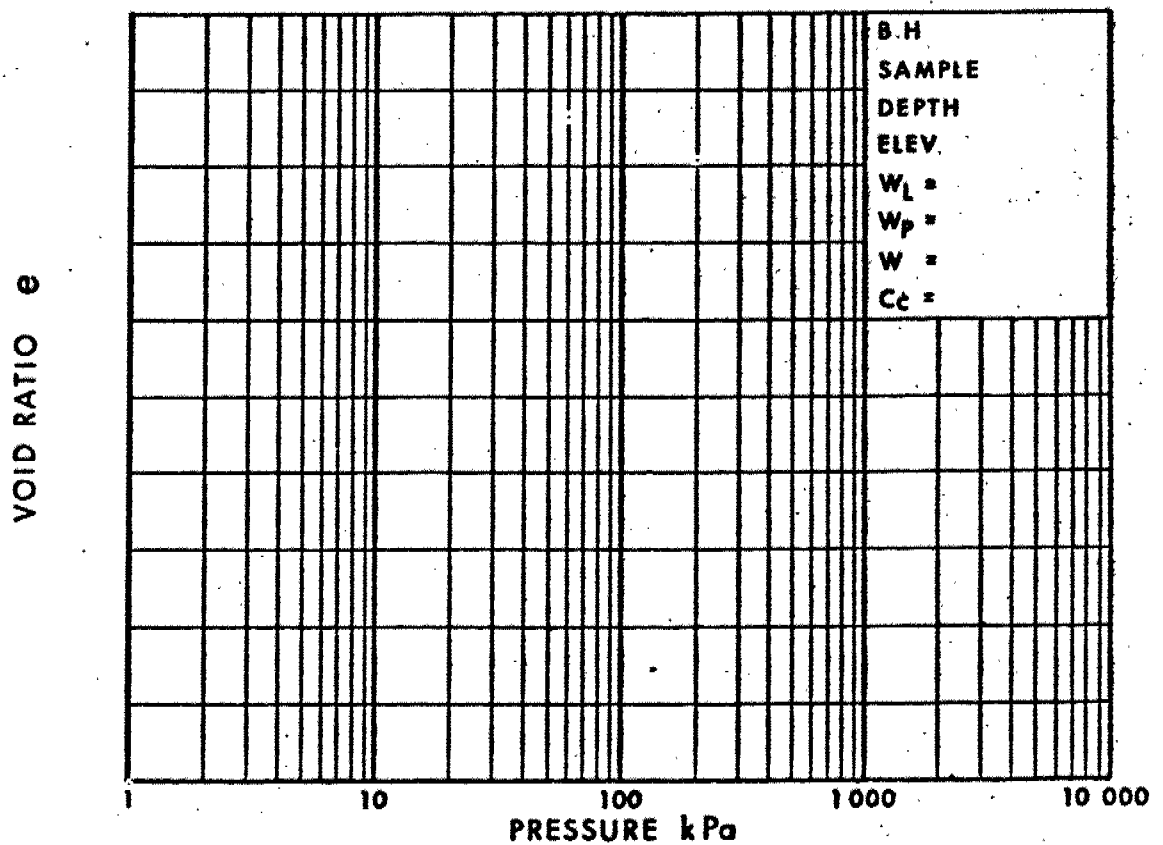
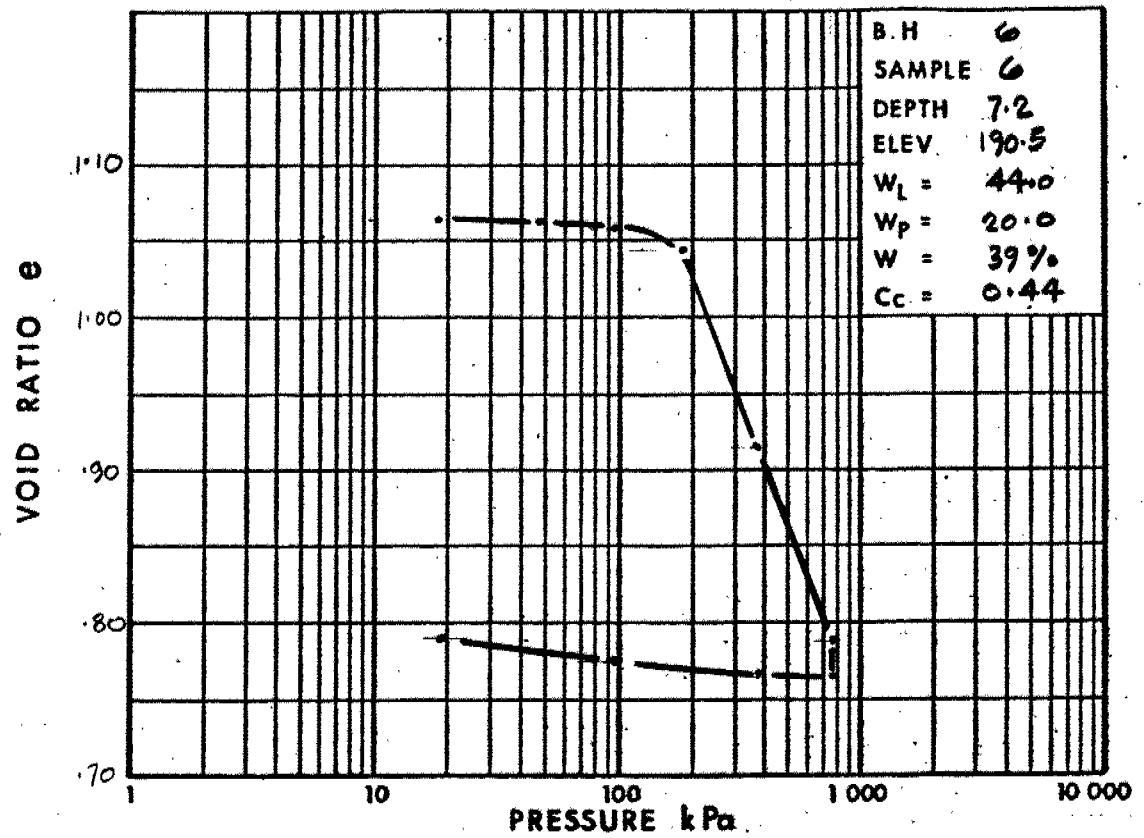
VOID RATIO - PRESSURE CURVES



W P 145-80-01

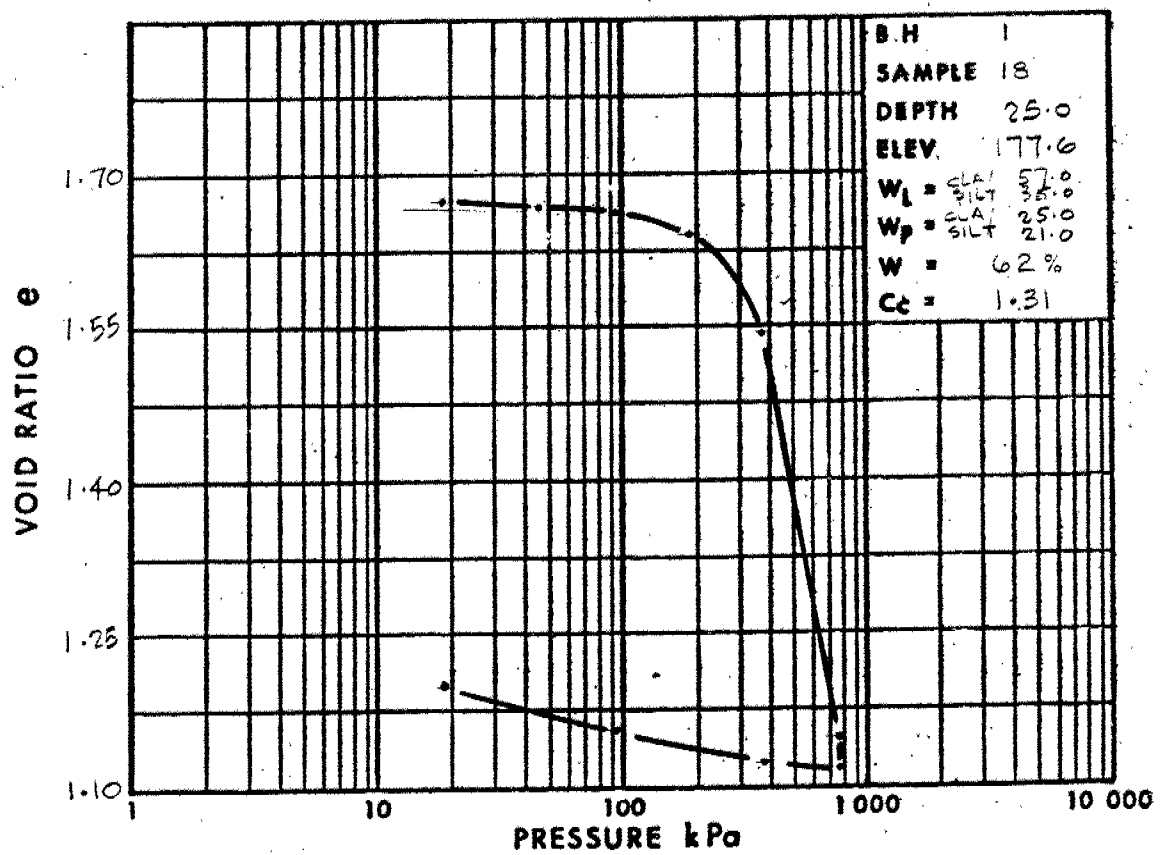
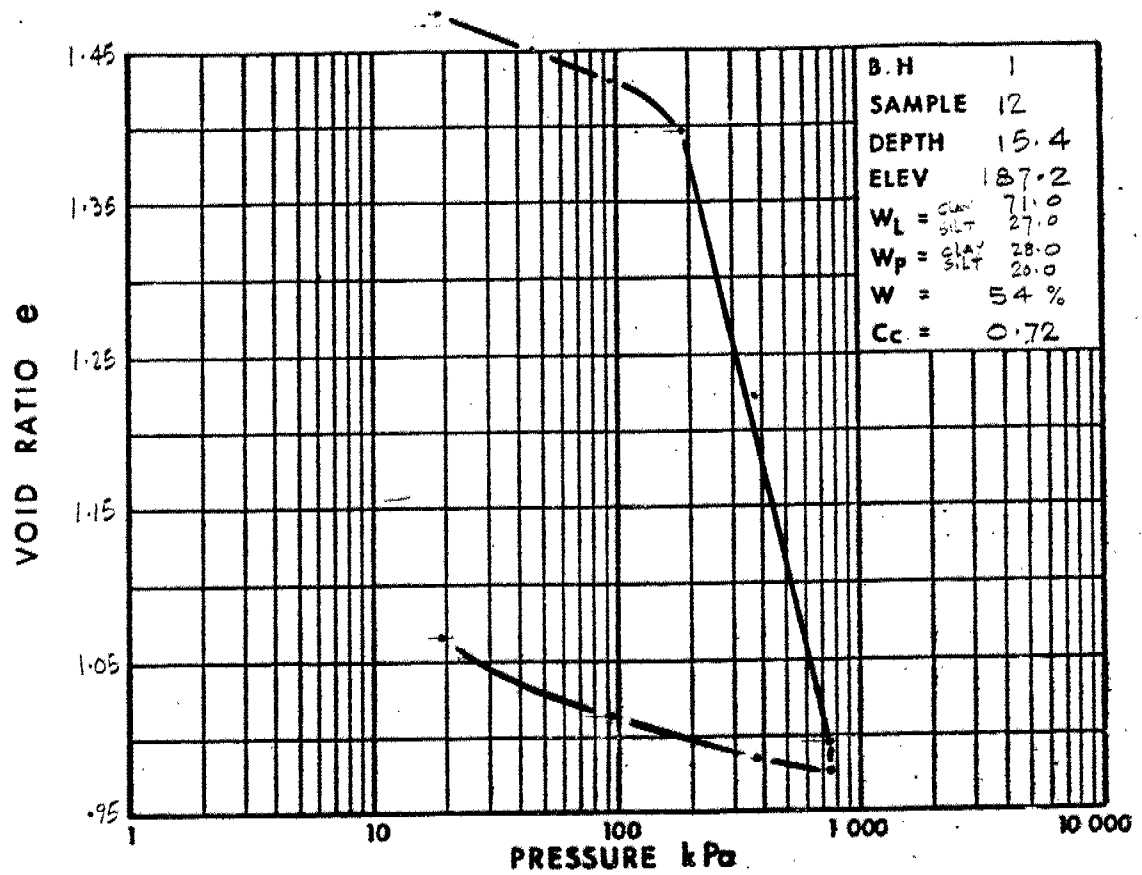
FIG No 7

VOID RATIO - PRESSURE CURVES



WP 145-80-01

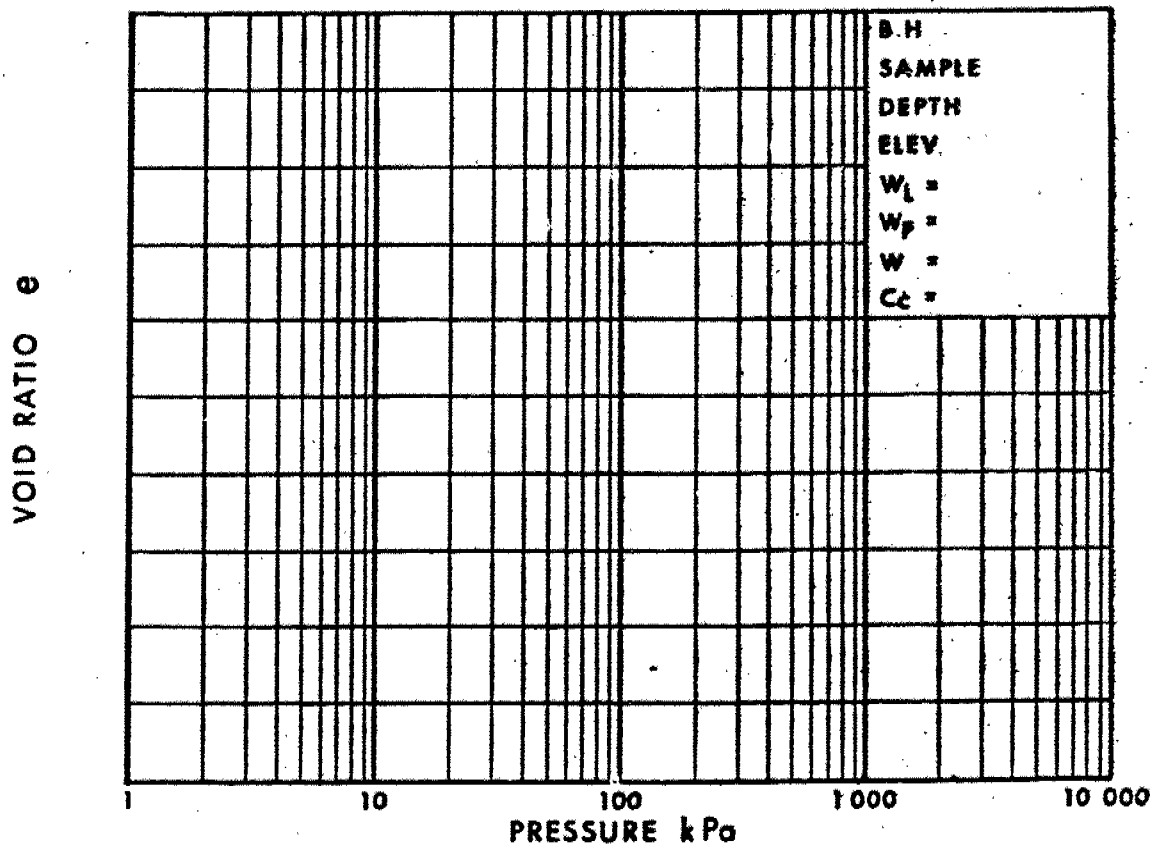
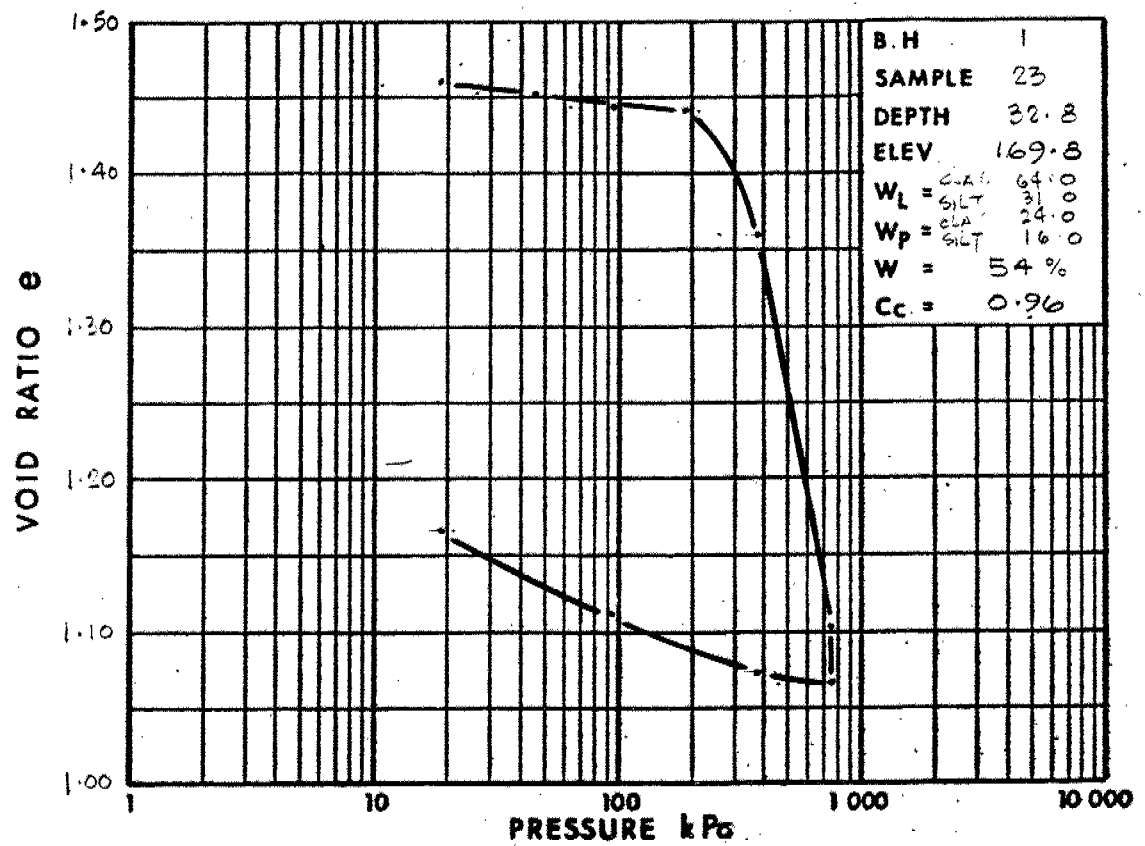
VOID RATIO - PRESSURE CURVES



W P 145-80-01

FIG No 9

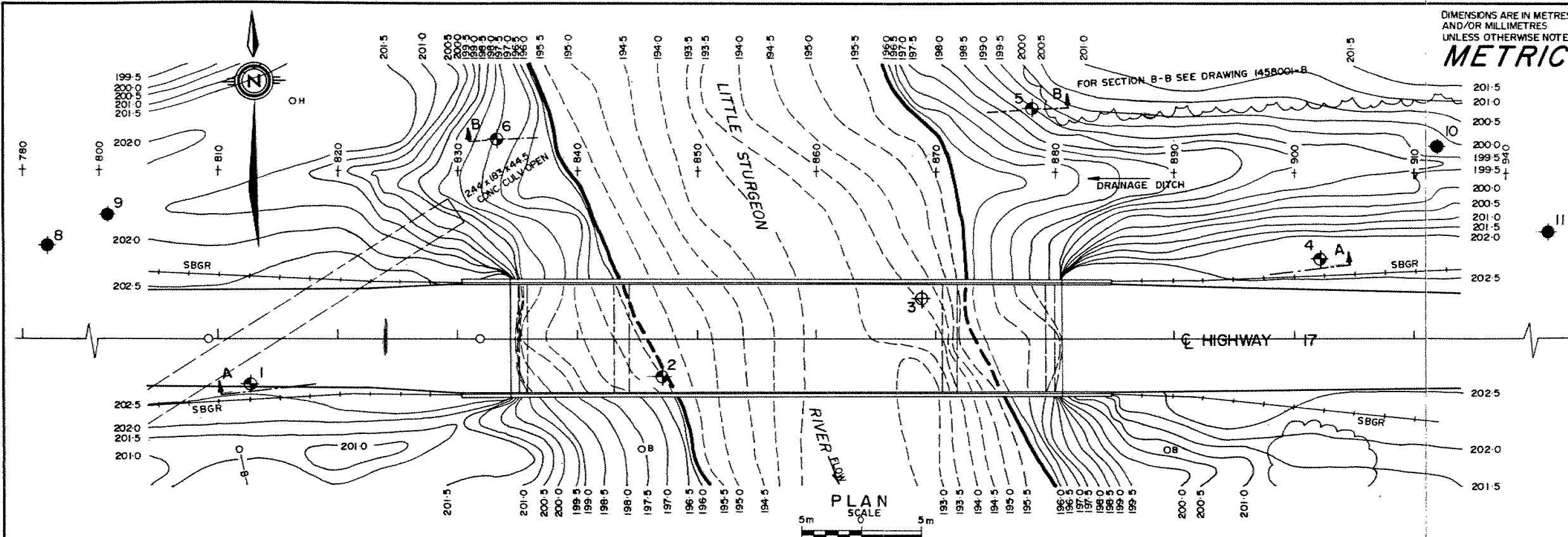
VOID RATIO - PRESSURE CURVES



W P 145-80-01

FIG No 10

OVERSIZE DRAWING

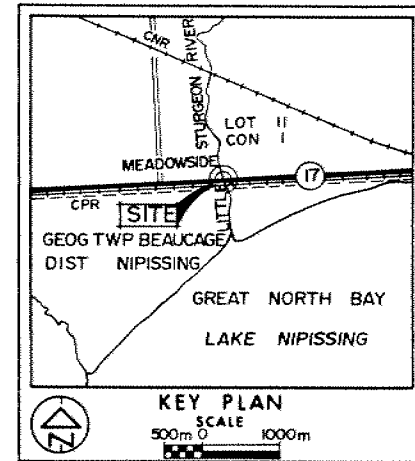


CONT No
WP No 145-80-01

LITTLE STURGEON RIVER
(25 km West of North Bay)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

GEOCON (1975) LTD.



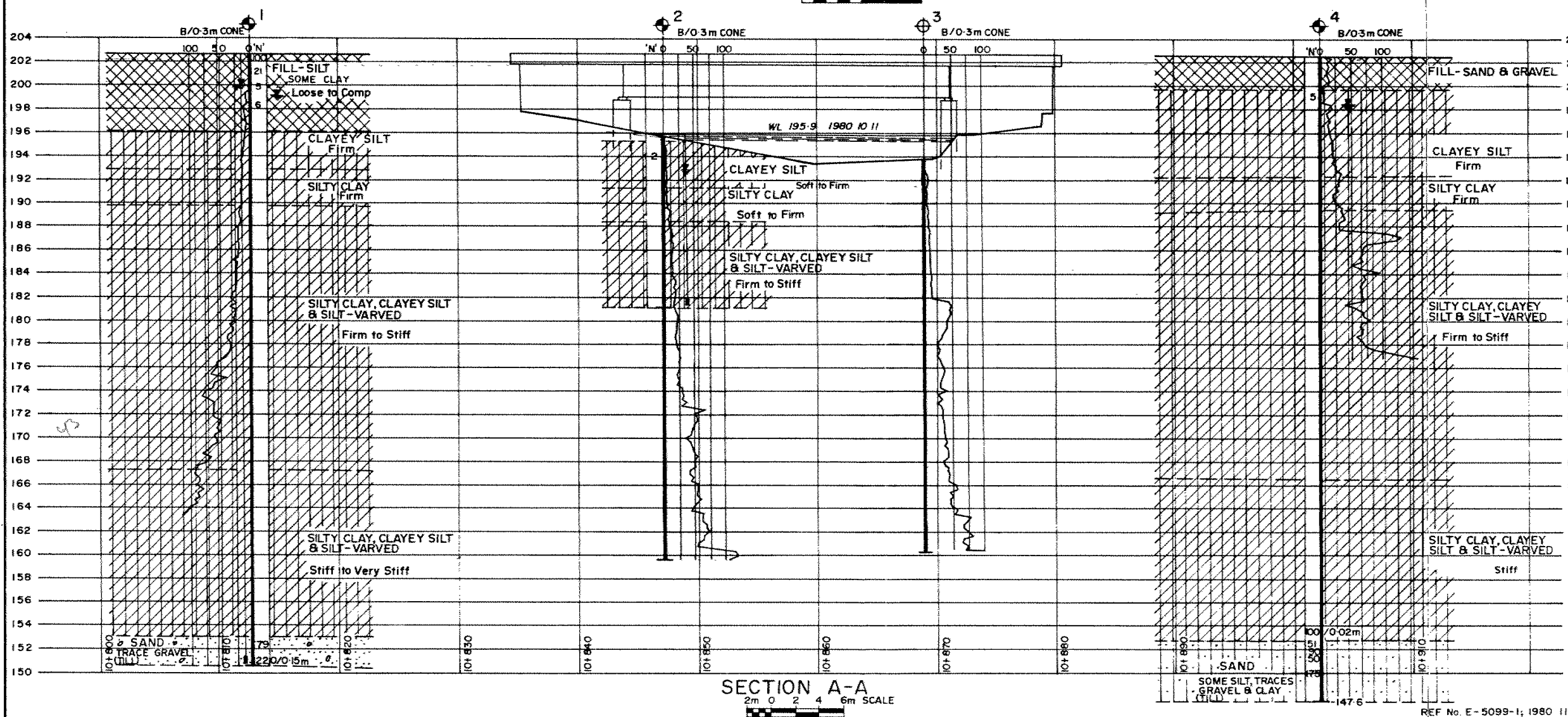
LEGEND

- 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 801221
- PIEZOMETER

No	ELEVATION	LOCATION	
		STATION	OFFSET
1	202.6	10+812.8	3.9 RT.
2	202.7	10+847.1	3.2 RT.
3	202.7	10+868.2	3.0 LT.
4	202.5	10+904.7	7.0 LT.
5	198.3	10+878.1	19.6 LT.
6	197.7	10+832.2	17.0 LT.
8	202.4	10+782.0	8.0 LT.
9	201.9	10+800.7	10.6 LT.
10	199.9	10+911.9	16.4 LT.
11	202.0 EST.	10+943.6	9.1 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.



DATE	BY	DESCRIPTION
1980 12 16	MAJ	314-50

Geocres No 314-50
HWY No 17 DIST 13
SUBM'D RCS [CHECKED] DATE 1980 12 16 SITE 43-64
DRAWN MCZ [CHECKED] APPROVED MAJ DWG 1458001-A

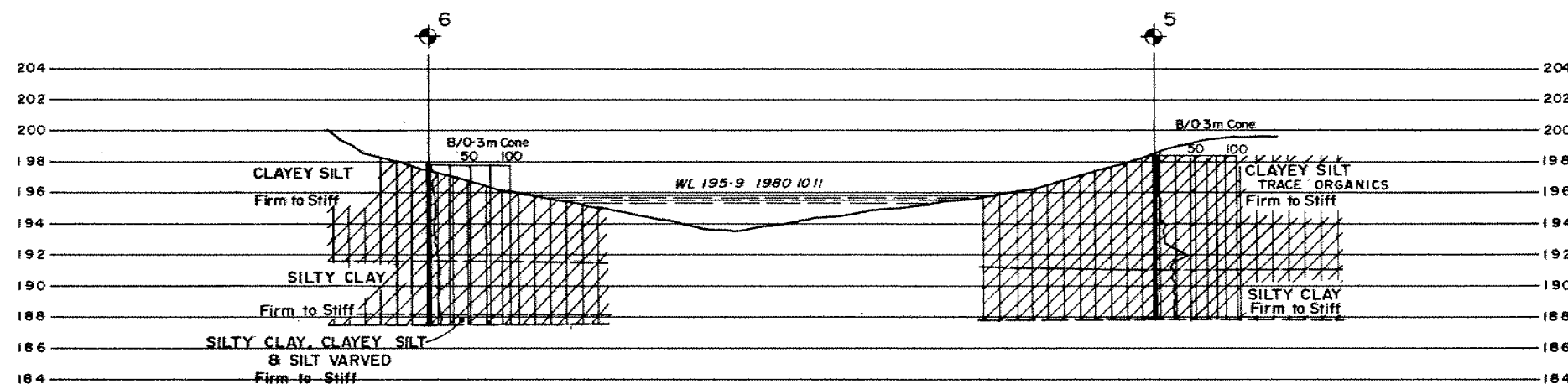
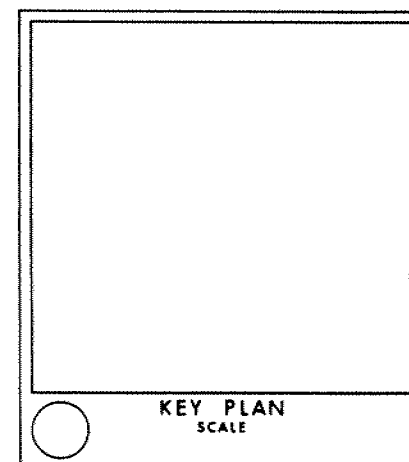
REF No. E-5099-1; 1980 11

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE NOTED

CONT No
WP No 145-80-01
LITTLE STURGEON RIVER
(25 km West of North Bay)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

GEOCON (1975) LTD.



SECTION B-B
SCALE
2m 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

No	ELEVATION		

NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No	
HWY No	17
SUBMITTALS CHECKED	DATE 1980 12 18
DRAWN MCZ	CHECKED MAJM
SITE	43-64
DWG	1458001-B

