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CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 17

LOCATION MONTREAL RD. TO ROCKLAND

No OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

ENGINEERING MATERIALS OFFICE
SOIL MECHANICS SECTION

WP 911-73-00 DIST 9

HWY 17 STR SITE

Feasibility Study of Hwy. 17
Montreal Road (Ottawa)
Easterly to Rockland

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GEOCRES 31G5-131

DATE JUL 17 1978

FOUNDATION INVESTIGATION REPORT

For

Feasibility Study of Hwy. 17
Montreal Road (Ottawa)
Easterly to Rockland
District 9, Ottawa
W.P. 911-73-00

INTRODUCTION

This report contains the results of a foundation investigation carried out at the location of the above project. The fieldwork was carried out from April 17 to May 3, 1978 and consisted of a total of 8 sampled boreholes advanced by means of hollow stem continuous flight augers or by washboring techniques to depths of 5.8 to 54.0 metres below the ground surface. Bedrock was proven by obtaining up to 1.5 metres of BXL or NXL size rock core in four of the borings, elsewhere the bedrock elevation was taken to be at the elevation of refusal to tri-coning.

STUDY AREA DESCRIPTION AND GEOLOGY

The study area is located along existing Hwy. 17 immediately east of Ottawa from Montreal Road easterly some 6.1 kilometres to Champlain Street, Orleans, in the Regional Municipality of Ottawa, Carleton.

The surrounding terrain is generally flat, however, Green's Creek and other small creeks have cut deep valleys. At Green's Creek the valley banks are as high as 7 metres and as steep as 45 degrees to the horizontal where past erosional slope failures have occurred. In some areas Green's Creek meanders through flood plains as wide as one kilometre. To the south of the highway and roughly parallel to it there exists an escarpment as high as 30 metres,being in some placesimmediately south of the highway and elsewhere being as much as one kilometre south of the highway. Rock outcrops were observed along this escarpment.

From Champlain Street to some 3.1 kilometres west of Champlain Street, land outside the highway right of way is presently being developed for residential purposes. The extent of residential development is shown on Figure 1 of the Appendix. The land use in the remainder of the study area is within the Ottawa Greenbelt and is presently being leased for farming purposes.

The existing Hwy. 17 is a four lane divided highway with grade separations from the junction of Hwy. 417 to just east of Montreal Road. The remainder of the existing highway in the study area is a two lane highway with at grade intersections. The grades of the highway are generally in the order of 2 to 3 metres above the average ground surface. However, grades are as much as 14 metres above the water level and 10 metres above the average ground surface at Green's Creek and Montreal Road respectively.

The proposed highway reconstruction is wholly located within the Ottawa Valley Clay Plains. During the time that the Champlain Sea was at its maximum only the land now above about elevation 200 metres remained dry. In the submerged areas soft, silty marine clay accumulated. When marine levels were at about elevation 100 metres, the salinity of the water changed, accompanied by the development of river currents capable of eroding terraces and scarps. The change in salinity resulted in a fresh water lacustrine deposit of oxidized rust mottled clay laid over the unconsolidated marine deposits. The river currents eroded the marine deposits from the bluff south of the escarpment shown on Figure 1. Finally, the Ottawa River in its high ancestral stages deposited alluvial sands in channel, bar delta and spit deposits on this bluff.

SUBSOIL CONDITIONS

General

Subsoil conditions were found to be quite uniform across the study area. The surficial deposit in the vicinity of Green's Creek is a 2.8 to 3.4 metre thick deposit of very stiff silty clay to clay. Underlying this cohesive deposit and elsewhere extending from the existing ground surface being 26.4 to 44.5 metres thick, is a firm to hard deposit of sensitive clay. Underlying this extensive cohesive deposit is a glacial till up to 10 metres thick having a hard consistency where cohesive and a very dense relative density where granular. Immediately below this glacial till is shale and or dolomite bedrock.

One boring was put down through the floodplain of the creek at about Sta. 20+700. Subsoil conditions here are somewhat variable. Recent slides of the creek banks adjacent to the highway show the dominant surficial deposit to be the sensitive clay deposit mentioned above. However, bedrock at this location is observed to outcrop in the creek bed both upstream and downstream of the existing Hwy. 17 crossing and also outcrops within the highway cut some 100 metres east of the creek. In certain localized areas bedrock is covered by shallow flood plain deposits. The borehole put down in the flood plain area revealed the presence of up to 3.5 metres of a compact to very dense deposit of gravelly sand with clay, silt and random cobbles overlying bedrock. In this location the upper 0.6 metres of the bedrock is weathered.

The boundaries between the various subsoil types are shown on the Record of Borehole Sheets. The locations of the boreholes are shown on a sketch, Figure 1. The various subsoil types are described as follows.

Silty Clay to Clay

This surficial deposit was encountered in most locations extending to a depth of 2.8 to 3.4 metres. The deposit is rust mottled in colour and is oxidized. Laboratory and in-situ testing performed on representative samples from this deposit gave the following results.

Geotechnical Properties			Range	Average
Natural Moisture Content	(w)	%	23-43	32
Liquid Limit	(w_L)	%	40-65	50
Plastic Limit	(w_p)	%	20-26	22
Plasticity Index	(I_p)	%	14-40	27
Liquidity Index	(I_L)		0.1-0.5	0.3
Bulk Unit Weight	(γ)	kN/m^3	17.5-19.4	18.2

Shear Strengths (c_u) kPa		Range
Field Vane Test	(5 tests)	>95
Unconfined Compression Test	(2 tests)	62-108

Consolidation Testing	(1 test)	
Initial Void Ratio	(e_o) %	0.61
Coefficient of Consolidation	(C_c)	0.11

The results of Atterberg Limit testing are plotted on Figure 3 and indicate that the deposit is inorganic and of medium to high plasticity. The deposit may be described as silty clay to clay. Based on the undrained shear strength as measured by the field vane test being greater than 95 kPa, the consistency of the deposit is estimated to be very stiff.

Sensitive Clay

This deposit was encountered immediately below the clayey silt to clay deposit or immediately below the ground surface and is estimated to have a thickness ranging from 26.4 to 44.5 metres. The deposit is grey in colour. Laboratory and in-situ testing performed on representative samples from this deposit gave the following results.

Geotechnical Properties			Range	Average
Natural Moisture Content	(w)	%	45-71	63
Liquid Limit	(w_L)	%	42-65	62
Plastic Limit	(w_p)	%	21-27	25
Plasticity Index	(I_p)	%	34-40	37
Liquidity Index	(I_L)		0.9-1.2	1.1
Bulk Unit Weight	(γ)	kN/m^3	15.8-16.2	15.9

Shear Strengths (c_u) kPa	Range	Sensitivity
Field Vane Test	44 to 95	5 to 18
Unconfined Compression Test	32 to 89	

Consolidation Testing (5 tests)	Range	Average
Degree of Preconsolidation ($\sigma_p' - \sigma_{vo}'$) kPa	125-260	210
Initial Void Ratio (e_o) %	1.74-1.98	1.87
Coefficient of Consolidation (C_c)	1.20-1.96	1.52

In addition, grain size distribution testing was performed on two representative samples from this deposit and the results are plotted on Figure 2 of the Appendix. The results of Atterberg Limit Tests are shown on the Plasticity Chart, Figure 3 and indicate that the deposit is inorganic and of medium to high plasticity, being generally of high plasticity. Furthermore, the Liquidity Index, being generally greater than 1, indicates that the deposit is sensitive to remoulding as confirmed by the high sensitivities, 5 to 18, as measured by the field vane testing. The deposit may be described as a sensitive clay.

Five consolidation tests were performed on samples from this deposit and are summarized on Figure 4 and 5. The consolidation testing indicates that the deposit has been preconsolidated by a pressure of 125 to 260 kPa in excess of the existing overburden pressure. The high initial void ratio, 1.7 to 2.0, together with the high coefficient of consolidation, 1.2 to 2.0, indicates that upon loading in excess of the preconsolidation pressure, the deposit will undergo significant consolidation.

In-situ vane testing indicates that the undrained shear strength decreases from greater than 95 kPa to as low as 44 kPa in the top 2 to 4 metres and thereafter increases to greater than 95 kPa at a depth of about 10 metres (a summary of shear strength verses depth is given on Figure 6 for the vicinity of Green's Creek and on Figure 7 for Sta. 16+600 to Sta. 17+700). The results indicate that the upper portion of the deposit has a very stiff to firm consistency decreasing with depth while the lower zone has a firm to very stiff consistency increasing with depth.

Furthermore, one consolidated undrained triaxial test with pore pressure measurements was performed on a sample from this deposit to estimate the effective strength parameters of this deposit. The test results are shown on Figure 8 in the Appendix. Further discussion on the effective strength characteristics of this subsoil is given in the section on "Slopes in the Vicinity of Green's Creek".

Gravelly Sand With Clay and Silt and Random Cobbles

This deposit was encountered in one borehole located within the floodplain of the creek at Sta. 20+700 extending from the ground surface to a depth of 3.4 metres below the ground surface. Grain size distribution testing performed on two representative samples from this deposit are shown on Figure 2 of the Appendix.

The deposit may be described as gravelly sand with clay and silt. Furthermore, the augering operation indicated the presence of cobbles and boulders within the deposit.

Based on Standard Penetration Test 'N' values being 8 and 10 blows per 0.30 metres, the relative density of this granular deposit may be described as loose to compact.

Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)

This cohesive glacial till was encountered in two borings immediately below the sensitive clay deposit. This deposit is estimated to be between 1.5 and 9.5 metres thick and is composed of a heterogeneous mixture of clayey silt, sand and gravel. Based on a Standard Penetration Test 'N' values of 17 blows per 0.30 metre and the nature of the washboring operation, the consistency of the deposit is estimated to be hard.

Heterogeneous Mixture of Silt, Sand and Gravel (Glacial Till)

This granular deposit was encountered in two borings in the vicinity of Green's Creek immediately below the sensitive clay stratum or below the cohesive glacial till deposit. The deposit is estimated to be between 4.0 and 4.9 metres thick and is composed of silt, sand and gravel with cobbles and boulders. Based on a Standard Penetration Test 'N' values of 70 blows per 0.15 metres and the nature of the washboring operation, this granular glacial till is estimated to have a very dense relative density.

Bedrock

Immediately below the glacial till deposit is bedrock. In four boreholes the bedrock was proven by obtaining up to 1.5 metres of BXL or NXL size rock core, whereas in other boreholes the bedrock surface was established at the point of refusal to tri-coning. The depth to bedrock below the ground surface varies widely from 3.35 to 53.95 metres which corresponds to elevation 1.98 to 46.48.

The bedrock is composed of either hard, fine textured, black shale bedrock or hard, fine textured, medium grey dolomite or comprised of layers of the above.

Groundwater Conditions

Groundwater level observations were carried out at the time of the field investigation by measuring in the open borehole. The groundwater was found to vary from 0.5 metres to 4.6 metres below the existing ground surface which corresponds to elevation 41.9 to 52.3. In addition, artesian conditions were encountered in one borehole put down in the floodplain of Green's Creek within the deposit of a heterogeneous mixture of silt, sand and gravel at a depth of 34.1 metres. The artesian head was found to stabilize at elevation 46.4 metres or 2.1 metres above the ground surface. Green's Creek was found to be at elevation 42.7 metres.

DISCUSSION AND RECOMMENDATIONS

It is proposed to reconstruct and upgrade Hwy. 17 immediately east of Ottawa from Montreal Road easterly 6.1 kilometres to Orleans. The existing facility throughout most of the proposed reconstruction area is a two lane undivided highway. Present reconstruction proposals call for a four lane divided highway with a median width of 20 metres to accommodate future transportation development within the corridor. Two lanes are to be constructed parallel to and some 25 metres north of the existing highway to serve as the future westbound lanes. It is understood that there will be no significant grade differences between the existing Hwy. 17 and the four lane facility.

The proposed structure locations and alternatives under consideration at this time are listed below.

Structure	Alternatives
Farmers Underpass	A/ Sta. 17+200 B/ Sta. 17+410 C/ Sta. 17+600
Eastern Parkway	A/ Under Hwy. 17 at Green's Crk. B/ Under Hwy. 17 at Sta. 16+510 C/ Over Hwy. 17 at Sta. 16+700
Hwy. 17 Over Green's Creek	A/ Construct Structure B/ Extend Existing Pipe Culvert
Hwy. 17 Over Culvert Sta. 20+700	Extend Concrete Box Culvert

The preliminary recommendations for the design and construction of the proposed structure or extension and their associated earthworks are discussed in the sections to follow.

The major geotechnical consideration in this area is the stability of slopes adjacent to Green's Creek. The banks of Green's Creek are composed of sensitive clay and are reknown for their erosional slope instability. The recommendations for cut and natural slopes in this area are applicable to some alternatives and in fact govern the geotechnical preference of alternatives. In view of the above, the stability of cut and natural slopes adjacent to Green's Creek will be discussed first.

Stability of Slopes in Vicinity of Green's Creek

The longterm stability of natural slopes and cut slopes as determined by using the total stress approach or the undrained shear strength of the cohesive strata (ϕ = o case) does not necessarily represent the most critical condition , since in this method of analysis the effect of pore pressure changes are not considered. Research has shown that the factor of safety computed by using the ϕ=o method may be as high as 4 to 10 for natural or cut slopes which actually failed. The pore pressure conditions within the natural or cut slopes will change with time due to the lateral seepage toward the slope faces. Therefore, only the effective stress approach was used to analyse the longterm stability of cut slopes along Green's Creek.

To estimate the shear strength parameters for this area, one consolidated undrained test with pore pressure measurements (Reference Figure 8) was carried out on a representative sample. The results were compared with a number of consolidated undrained tests with pore pressure measurements carried out on clay samples for a previous job in the nearby area (Contract 74-121, Hwy. 17 Slope Improvement East of Cumberland). It is important to note that Skempton * observed that "from the analysis of actual slips in clay, the values of the shear strength parameters as determined by conventional tests do not necessarily bear any relation to the values which must have been operative at the time of failure". In view of this a study was made of the published research and the Ministry's documents on slope failures that have occurred in this general area. The study has indicated that the following parameters may be used for the design of natural or cut slopes in this area.

$r_u = 0.62$ where $r_u = \frac{u}{\gamma H}$ and u is the excess pore water pressure at any point along the potential failure surface; h is the depth of the soil mass below the slope surface and γ is the bulk unit weight of the soil.

c' = 10-12 kPa

ϕ = 23-25

Skempton A.W. "Opening Address" Proceedings European Conference, Stability of Earth Slopes, Stockholm Vol. 3, pp 16-20

Based on our analysis (Refer to Figure 10) the following are our recommendations.

1. Cuts or natural slopes up to 5 metres deep will be stable with side slopes of 2 horizontal to 1 vertical.

2. To ensure the stability of cut or natural slopes up to 9 metres deep with side slopes of 2 horizontal to 1 vertical, it will be necessary to provide a mid height counter balancing berm 5 metres wide with the outer 2.5 metre width being rock fill keyed 1.5 metres into the subsoil (Refer to Figure 10).

The extent of the recommended stabilization of the creek banks is Sta. 16+400 to Sta. 16+500.

In order to minimize the effects of the creek banks on the proposed highway, it will be advantageous to shift the alignment of the westbound lanes away from the creek banks toward the existing highway in the vicinity of Green's Creek.

Farmer's Underpass

The alternate locations identified for this crossing at the approximate mid point of the Greenbelt (Refer to Figure 1) are as follows.

Location	Reference Borehole	Bedrock Elevation (m)
Sta. 17+200	5	17.56
Sta. 17+410	7	24.18
Sta. 17+600	6	17.34

Subsoil at these locations consists of an extensive deposit of stiff to very stiff sensitive clay overlying a thin veneer of glacial till overlying shale and/or dolomite bedrock. Due to the similarity of subsoil conditions at these locations the recommendation for the design and construction of the structure and associated approach fills are similar and are as discussed below.

Approach Fills

Analysis in terms of total stress have been carried out to determine the stability of fills immediately after construction. In this method of analysis, stability is governed by undrained shear strength properties of the foundation and fill materials. The following data and values were used in carrying out the stability analysis.

Fill Material

Locally Available Granular Type Fill $\gamma = 22\,{}^{kN}/m^3$ $\phi = 30^{\circ}$ $c_u = 0$

Subsoil Foundation Material

Elevation (Ft.)	γ (kN/m^3)	γ' (kN/m^3)	ϕ	Cu (kPa)
53-51	17.6	7.9	0	95
51-49	15.7	6.0	0	43
49-43	15.7	6.0	0	60
43-40	15.7	6.0	0	74

The assumed subsoil stratigraphs, critical circle and factor of safety are shown on Figure 9.

Based on the analysis fill heights up to 10 metres will be stable with side slopes of 2 horizontal to 1 vertical if constructed of compacted granular type fill.

Furthermore, at each of the proposed underpass locations consolidation of the underlying compressible clay stratum will settle as a result of consolidation under the weight of the embankment. Preliminary settlement analysis indicated that the maximum settlement due to consolidation of the clay deposit beneath a 10 metre high fill will be in the order of 0.40 metres. Based on time rate settlement analysis it is estimated that 80% of this will occur within a period of 18 to 24 months after construction. To minimize post construction settlements (total as well as differential) between the approach fills and the approach slabs, it is recommended that if possible paving of the approaches is to be delayed for a period of up to 2 years after completion of the approach fills.

Structure Foundation: End Bearing Piles

The structure piers and abutments should be supported on end bearing steel 'H' piles driven to the bedrock surface. Considerable negative skin friction loads may be imposed on the piles supporting the abutments and end pier due to settlement of the roadway embankment. In view of this it is recommended that the pile capacities for the abutments and end piers should be reduced by 15%; i.e. 85% of their maximum allowable loads. For example, a HP 12X74 steel H pile may be designed for 750 kN per pile. Piles elsewhere may be designed for their maximum allowable load, i.e. a HP 12X74 steel 'H' pile may be designed for 900 kN per pile.

In addition to the negative skin frictional forces, movement of subsoil due to strain imposed by the embankment loading, will generally tend to displace the piles laterally and can cause rotation of the abutments. In view of this, we recommend that consideration be given to supporting the extreme ends of the wing walls on end bearing piles founded as aforementioned. It is considered that this will improve the stability in the longitudinal direction. No bouldery or rock fill should be placed in areas where piles are to be driven.

Structure Foundation: Friction Piles

As an alternative to end bearing piles the abutments and piers can be founded on piles located within the clay stratum. Such piles would primarily derive their capacity from the adhesion between the foundation soil and the shaft of the pile. The allowable pile load would be dependent on the pile type and section chosen - for example, No. 14 timber piles driven 15 metres into original ground could be designed for an allowable pile capacity of 150 kN.

The structure units founded on friction piles will undergo settlement due to the consolidation of the foundation soil under application of load. The actual magnitude of the settlement at the various locations will be dependent on a number of factors, including

i) Pile type and length
ii) Configuration of the pile group
iii) Applied load
iv) Influence of approach fills

A true settlement analysis, therefore, can only be carried out once the structure details have been finalized. A few qualitative points, however, can be made as follows.

a) The settlement of the piles at the abutment and the pier locations will be influenced by the approach fills. Differential settlements, therefore, can be expected between the abutments and end piers. This being the case, it is recommended that the end spans of the structures be simply supported.

b) If friction piles are employed to support the structural elements, then it is recommended that the allowable pile load be determined by carrying out full scale pile loading tests at this site.

Pile caps should be founded at sufficient depth below finished grade so as to ensure adequate frost protection.

Alternate Comparisons

Based on geotechnical considerations the location at Sta. 17+600 (adjacent to the existing D.N.D. pumping stations) is most favourable because of the relatively shallower depth of overburden (thus the relatively shorter pile lengths).

Eastern Parkway Crossing

The alternate locations identified for this crossing by N.C.C. are as follows.

Location	Reference Borehole	Bedrock Elevation (m)
Sta. 16+700	4	1.98
Ravine Sta. 16+510	3	3 (estimated)
Green's Creek	2, 1	5.32

Subsoil at these locations consists of up to 44.5 metres of firm to very stiff clay overlying up to 19.5 metres of a hard heterogeneous mixture of clayey silt, sand and gravel or a very dense heterogeneous mixture of silt, sand and gravel. Although the subsoil conditions at these locations are similar, the natural topography and proximity of the Eastern Parkway alignment to the slopes of Green's Creek dictate different construction and design techniques and procedures to be employed at each location. The recommendations are discussed separately for each location in the paragraphs to follow.

A. Eastern Parkway Underpass located at Sta. 16+700
This location is by far the best alternative from a geotechnical point of view because of the remoteness from the slopes of the banks of Green's Creek.

The structure can be supported on end bearing steel 'H' piles or friction piles. The recommendations would be as outlined in the preceeding section.

Preliminary stability analysis indicate that fill heights up to 10 metres high will be stable with side slopes of 2 to 1 if constructed in accordance with current Ministry procedures.

Settlement will occur due to the consolidation of the underlying compressible clay deposit. Preliminary settlement analysis indicate that the maximum settlement due to consolidation of the sensitive clay deposit beneath a 10 metre fill height will be in the order of 0.5 metres. It is estimated that 80% of this settlement would occur within the first 18 to 24 months after construction. The recommendations outlined in the previous section for minimizing post construction maintenance are applicable at this location also.

B. Eastern Parkway located under Hwy. 17 at Green's Creek and
C. Eastern Parkway located under Hwy. 17 at Ravine Sta. 16+510.

These locations are by far the worst alternatives from a geotechnical point of view because of the proximity of the Parkway alignments to Green's Creek. The banks of Green's Creek are composed of sensitive firm to very stiff clay and are reknown for their erosional instability. Any construction on or adjacent to these river banks should be discouraged. At the time of writing this report, full details of the alignment were not known and detailed recommendations cannot be made at this time. However, for all natural or cut slopes it will be necessary to follow the recommendations similar tc those discussed under "Stability of Slopes in the Vicinity of Green's Creek. In addition, diversion of Green's Creek may also be required depending on the scheme.

Even if the above measures are carried out there is no guarantee that future meandering of the creek will not result in the reoccuring need for extensive maintenance procedures.

Structures constructed at these locations could be supported on end bearing steel 'H' piles or friction type piles. The recommendations for pile types and loading would be as described above.

Hwy. 17 Over Green's Creek (Reference B.H. 1, 2)

Two alternatives are available for this location:

A. Extending the four existing structural plate culverts or
B. Constructing a structure at this location.

Subsoil here consists of 3 metres of very stiff clayey silt to clay overlying up to 29 metres of stiff to very stiff sensitive clay. This extensive cohesive deposit overlies 7 metres of hard or very dense glacial till overlying dolomite and/or shale bedrock.

Structure

A structure at this location could be supported on end bearing or friction piles. The recommendations for piles are as described elsewhere.

It is understood that there will be no significant grade differences between the existing highway and the proposed highway. However, the construction of a structure would require excavation of the roadway foundation for the creek channel. It is believed that the required excavation will encroach upon the natural creek banks west, and possibly east, of the creek. If it is decided to proceed with the structure option it will be necessary to carry out borings through the roadway foundation in the area of the channel cut to determine the subsoil conditions and hence design a stable slope geometry. However, for the purposes of the preliminary design the recommendations for the required channel cut slopes are as discussed under "Stability of Slopes in Vicinity of Green's Creek".

Furthermore, east of the creek the proposed construction requires widening of the roadway embankment to the north. The following procedures should be adopted for such a widening:

1. All topsoil is to be removed within the plan limits of the proposed widening.
2. Transverse slopes not to exceed 2:1
3. Longitudinal slopes to correspond with creek channel slopes
4. Additional fill material to be carried out in accordance with M.T.C. Standard Benching of Earth Slopes (DD-414)

Settlement of the entire embankment will occur due to the consolidation of the underlying compressible clay deposit upon loading from the widening. Excessive differential settlements will occur between the widened portion and the existing embankment. Furthermore, differential settlements will occur between the widened portion and the approach slabs. In order to minimize post construction maintenance costs due to differential settlements it is recommended that if possible the paving of the widened portion of the embankment be delayed for a period of up to 1 year.

Extension to Existing Structural Plate Culverts

This structural plate culvert should be provided with a minimum 0.3 metres of granular pad beneath the culvert extension. Additional fill material should be designed and constructed as discussed under "Structure" i.e. removal of topsoil and benching of slopes, etc.

Settlement of the highway embankment will occur due to the consolidation of the underlying subsoil and differential settlement is anticipated between the existing roadway and the widened portion. Thus, it will be necessary to provide the extension with a camber to accommodate differential settlements along the length of the culvert. In order to further minimize post construction settlement problems it is desirable to delay the paving operations of the new portion of the highway in this area for a period of up to 1 year upon completion of the approach fill construction.

Water Crossing Creek Sta. 20+700 (Reference H.E. 8)

Hwy. 17 crossing this creek is presently accomplished by a closed type concrete box culvert. Reconstruction proposals call for the northerly extension of this culvert to accommodate the additional lanes.

MISCELLANEOUS

The fieldwork for this report was carried out under the supervision of Mr. M. MacLean, Project Engineer, using equipment rented from Dominion Soil Investigation Ltd., Toronto

This report was prepared by Mr. M. MacLean and reviewed by Mr. M. Devata, Supervising Engineer

M MacLean

M. MacLean, P. Eng.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
M. D. MacLEAN
PROVINCE OF ONTARIO

M. Devata

M. Devata, P. Eng.
Supervising Engineer

July, 1978

APPENDIX

RECORD OF BOREHOLE No 1

W P 911-73-00 LOCATION Sta. 16+276.0; O/S 21.00 m Lt. ¢ Hwy 17 W.B.L. ORIGINATED BY M.M.
DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Augers & Cone Test & Washboring COMPILED BY M.M.
DATUM Geodetic DATE April 17-19, 1978 CHECKED BY R S

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH KPa	WATER CONTENT (%)	UNIT WEIGHT γ KN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
44.33	Ground Level				Head					
0.00	Silty Clay Rust Mottled and Oxidized Very Stiff	1	SS	4			+> 95 KPa			
							+> 95 KPa			
		2	TW	PH	▼		● ; +> 95KPa		19.43	e_o=0.6 C_c=0.11
41.33 / 3.00	Stiff	3	TW	PH			5 +; 4 +			
	Stiff to Very Stiff	4	TW	PH		40.00	o; 6 +; 5 +		15.80	0 0 26 74
	Sensitive	5	TW	PH			6 +; 6 +			
	Clay	6	TW	PH	Artesian Water		11 +; +> 95 KPa			
	Grey	7	TW	PH		35.00	+> 95 KPa			
		8	SS	3			+> 95 KPa			
		9	TW	PH			o; +> 95 KPa		16.19	e_o=1.74 σ_p' =335 kPa C_c=1.19
		10	SS	10		30.00				
		11	TW	PH			6 +; +> 95 KPa			
	With Inclusions or Pockets of Organics									
		12	SS	7			+> 95 KPa			
		13	TW	PH		25.00			17.27	
						20.00				
		14	SS	10						
14.33 / 30.00						15.00				

Continued

$+^3, x^5$: Numbers refer to Sensitivity

20 / 15 -O- 5 / 10 (%) STRAIN AT FAILURE

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH KPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION – ENGINEERING MATERIALS OFFICE – SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 1 Continued

W P 911-73-00 LOCATION Sta. 16+276.0, O/S 21.00 m Lt. ₵ Hwy. 17 W.B.L. ORIGINATED BY M.M.
DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Augers, Cone Test & Washboring COMPILED BY J.A.
DATUM Geodetic DATE April 17-19, 1978 CHECKED BY RS

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
14.33	continued										
30.00	Sensitive Clay										Method of Boring 0-27.5 m Auger 27.5-39.0 m Tri-cone Ahead 39-39.5 m BX Casing
11.72											
32.61	Heterogeneous Mixture Clayey Silt, Sand and Gravel, Hard		15	SS	17	Encountered					
10.19											
34.14	Heterogeneous Mixture Silt, Sand & Gravel With Cobbles, Diameter 50-150 mm, Every 450 mm, Very Dense (Glacial Till)		16	SS	70/0.15 m		10.00				
5.32											
39.01	Black Shale Bedrock Sound		17	RC BXL	REC 100% RQD 90 %		5.00				
3.30											
41.03											

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

15 -○- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2

W P 911-73-00 LOCATION Sta. 16+332.3; o/s 7.00 m Rt. ¢ Hwy. 17 W.B.L. ORIGINATED BY M.M.

DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Auger (0-15 m); Washboring (15-41m) COMPILED BY M.M.

DATUM Geodetic DATE April 20, 1978 CHECKED BY RS

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
49.14	Ground Level
0.00	Silty Clay Rust, Mottled Oxidized Very Stiff
45.78 3.36	Firm Sensitive Clay Grey Stiff Very Stiff
11.96 37.18	Heterogeneous Mixture Silt, Sand & Gravel Very Dense (Glacial Till) With Boulders up to 125 mm Diameter
7.99 41.15	End of Borehole Refusal to Tricone Probable Bedrock

SAMPLES

NUMBER	TYPE	'N' VALUES
1	SS	8
2	TW	PH
2A	SS	3
3	TW	PH
4	SS	2
5	TW	PH
6	SS	1
7	SS	1/0.45 m
8	TW	PH

GROUND WATER CONDITIONS

ELEVATION SCALE: 45.00, 40.00, 35.00, 30.00, 10.00

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

Sensitivity values plotted: 95, 95, 4, 5, 18, 7, 5, 5, 8, 8, 6, 6, 7, 6, 95

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ KN/m^3: 17.56, 15.29, 15.70

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

20 / 15 ✧ 5 / 10 (%) STRAIN AT FAILURE

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Ministry of Transportation and Communications
Ontario

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 3

W P 911-73-00 LOCATION Sta 16+492.0; o/s 0.20 m Rt. ℄ Hwy. 17 W.B.L. ORIGINATED BY M.M.

DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.M.

DATUM Geodetic DATE April 24, 1978 CHECKED BY

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (Sensitivity)	WATER CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
49.83 / 0.00	Ground Level; Sensitive Clay Grey, Very Stiff	1	SS	4			7, 5			
		2	TW	PH			7, 5		16.80	
	Stiff	3	SS	2/0.45 m	▼		9, 7			
		4	TW	PH		45.00	9, 8		15.80	e_o=1.98, σ_p'=240 kPa, C_c=1.96
		5	SS	1/0.45 m			7, 4			
		6	TW	PH			9, 5			
	Very Stiff	7	SS	1/0.45 m		40.00	95			
		8	TW	PH			95		15.70	e_o=1.94, σ_p'=320 kPa, C_c=1.54
		9	SS	2/0.45 m		35.00	95			
30.48		10	SS	2			95			
19.35	End of Borehole									

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, 'N' VALUES); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; GR SA SI CL

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

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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RECORD OF BOREHOLE No 4

W P 911-73-00 LOCATION Sta. 16+697.7: o/s 3.30 m Rt. ¢ Hwy. 17 W.B.L. ORIGINATED BY M.M.

DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Auger (0-30 m) Washboring (30-54 m) COMPILED BY M.M.

DATUM Geodetic DATE April 24, 1978 CHECKED BY RS

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	FIELD VANE (+) shear strength / sensitivity	REMARKS
51.97	Ground Level						
0.00	Sensitive Clay Grey, With Organic Inclusions Below Elevation 35.00	1	SS	8		+> 95	
	Very Stiff	2	TW	PH	50.00	9, 6	
	Firm	2A	SS	1/0.45m		8, 7	
	Stiff	3	TW	PH		9, 7	e_o=1.87, σ'_p=255 kPa, C_c=1.20
		4	SS	1/0.45 m	45.00	8, 7	
		5	TW	PH		+> 95	e_o=1.84, σ'_p=265 kPa, C_c=1.71
		6	SS	1/0.30 m		5, 6	
		7	TW	PH	40.00	7, 5	
	Very Stiff	8	SS	1/0.30 m	35.00	+> 95	
7.47 / 44.50	Some Cobbles; Heterogeneous Mixture Clayey Silt, Sand and Gravel; Hard (Glacial Till)				5.00		
					0.00		
1.98 / 53.95	End of Borehole, Refusal to Tricone, Probable Bedrock						

$+^3$, x^5 : Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 5

W P 911-73-00 LOCATION Sta 16+256.9; o/s 13.70 m Lt. Hwy 17 W.B.L. ORIGINATED BY M.M.

DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Auger (0-30 m), Washboring (30-35 m) COMPILED BY M.M.

DATUM Geodetic DATE April 27, 1978 CHECKED BY R S

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
53.07	Ground Level										
0.00	(Probable) Sensitive Clay Grey Stiff to Very Stiff						50.00 45.00 40.00 20.00				
17.56 / 35.51	Grey Dolomite Bedrock Sound		1	NXL RC	REC 100% RQD 80%						
16.04 / 37.03	End of Borehole Note: Water Level Not Established										

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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RECORD OF BOREHOLE No 6

W P 911-73-00 LOCATION Sta. 17+612.0; O/S 7.00 m Rt ₵ Hwy 17 W.B.L. ORIGINATED BY M.M.

DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Auger (0-12 m) Tricone (12-36 m) COMPILED BY M.M.

DATUM Geodetic DATE April 28, 1978 CHECKED BY R.S.

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: 'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH: O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
53.15	Ground Level										
0.00	(Probable) Sensitive Clay Grey Stiff to Very Stiff						50.00				
							45.00				
							20.00				
17.34											
35.81	End of Borehole Refusal to Tricone Probable Bedrock Note: Water Level Not Established										

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

15 ◆ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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RECORD OF BOREHOLE No 7

W P 911-73-00 LOCATION Sta. 17+404; o/s 16.00 m Lt. ¢ Hwy. 17 W.B.L. ORIGINATED BY M.M.
DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Auger (0-14 m) Washboring (14-29 m) COMPILED BY M.M.
DATUM Geodetic DATE May 1, 1978 CHECKED BY RS

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH KPa (+ FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
53.44	Ground Level									
0.00	Silty Clay to Clay Rust - Mottled Oxidized Very Stiff	1	SS	5	▼		+95			
		2	TW	PH			+95	Wp–W–WL	17.56	
50.59 / 2.85	Stiff to Very Stiff Sensitive Clay Grey	3	SS	2		50.00	+10, +5			
		4	TW	PH			+95			
						45.00				
		5	SS	1/0.45 m			+95			
		6	SS	1/0.45 m			+95	Wp–W		
						40.00				
						25.00				
24.18 / 29.26	Layers of Black Shale and Dolomite Bedrock Sound	1	RC NXL	REC =82% RQD =73%						
22.50 / 30.94	End of Borehole									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH KPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL — WATER CONTENT (%) 20 40 60

+3, ×5 : Numbers refer to Sensitivity

15 ◇ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

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HIGHWAY ENGINEERING DIVISION-ENGINEERING MATERIALS OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE No 8

W P 911-73-00 LOCATION Sta. 20+707.3; o/s 14.00 m Lt. ₵ Hwy. 17 W.B.L. ORIGINATED BY M.M.
DIST 9 HWY 17 W.B.L. BOREHOLE TYPE Hollow Stem Auger COMPILED BY M.M.
DATUM Geodetic DATE May 3, 1978 CHECKED BY R S

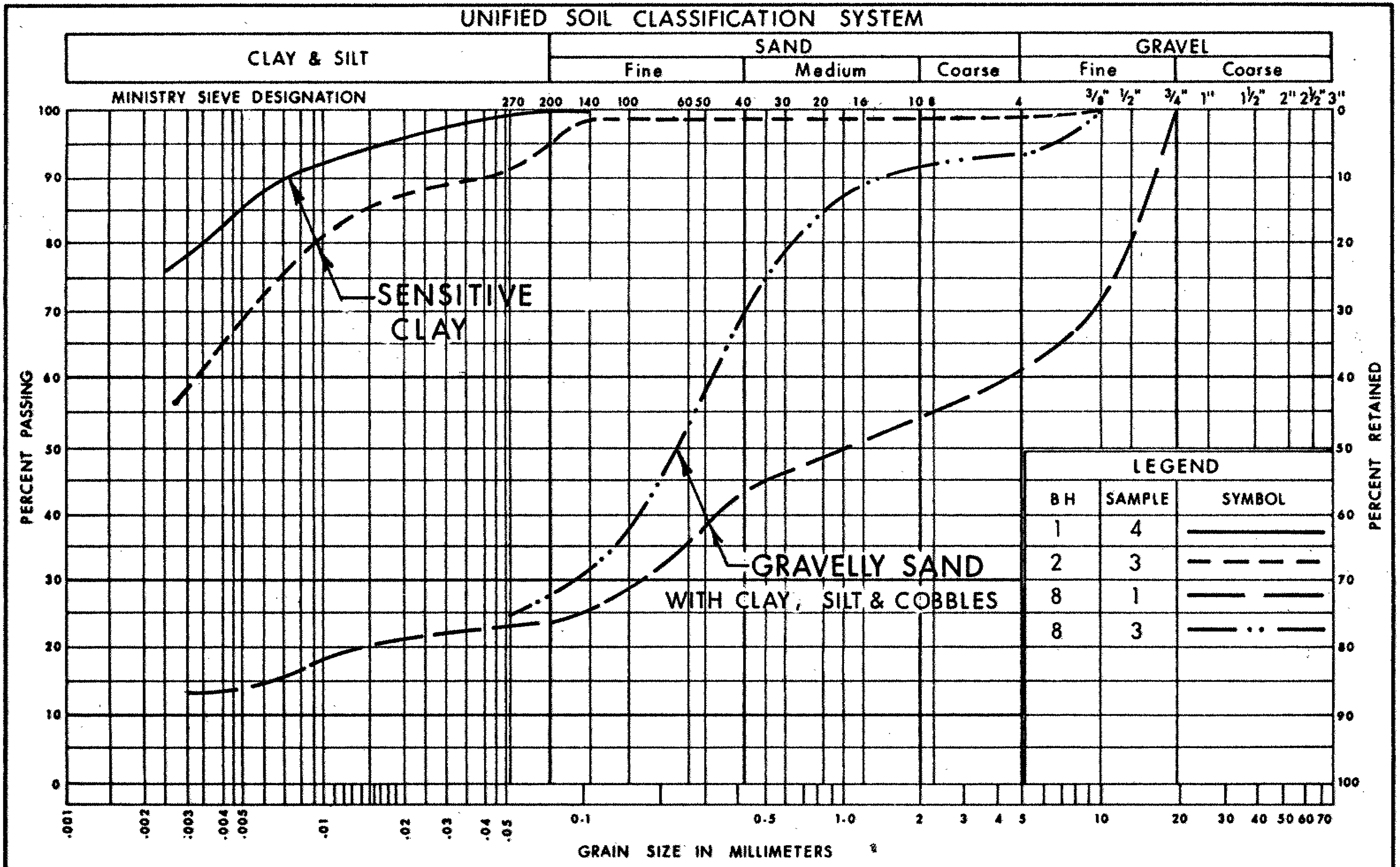
ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
50.83	Ground Level										
0.00	Gravelly Sand With Clay, Silt and Random Cobbles Loose to Compact		1	SS	10	▼	50.00				39 38 10 38
			2	SS	8						
46.48 / 3.35	Weathered		3	SS	30/0.075 m						6 66 (28)
	Sound Grey Dolomite Bedrock		4	RC BXL	REC =90% RQD =90%						
45.04 / 5.79	End of Borehole						45.00				

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

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Oct 75, FF-S-22



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GRAIN SIZE DISTRIBUTION

FIG No 2

W P 911 - 73 - 00

VOID RATIO – PRESSURE CURVES

WP 911-73-00

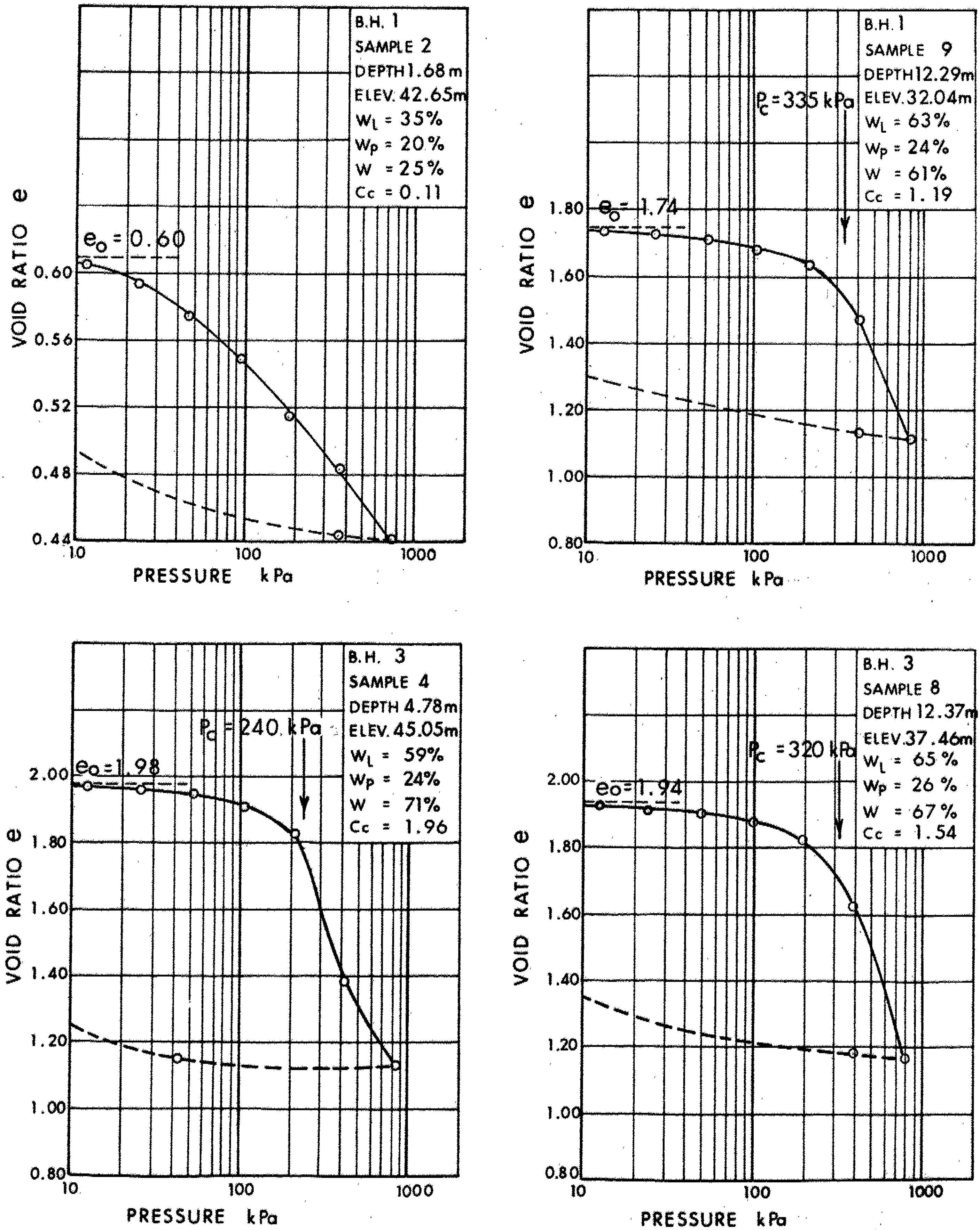


FIG. 4

VOID RATIO – PRESSURE CURVES

WP 911-73-00

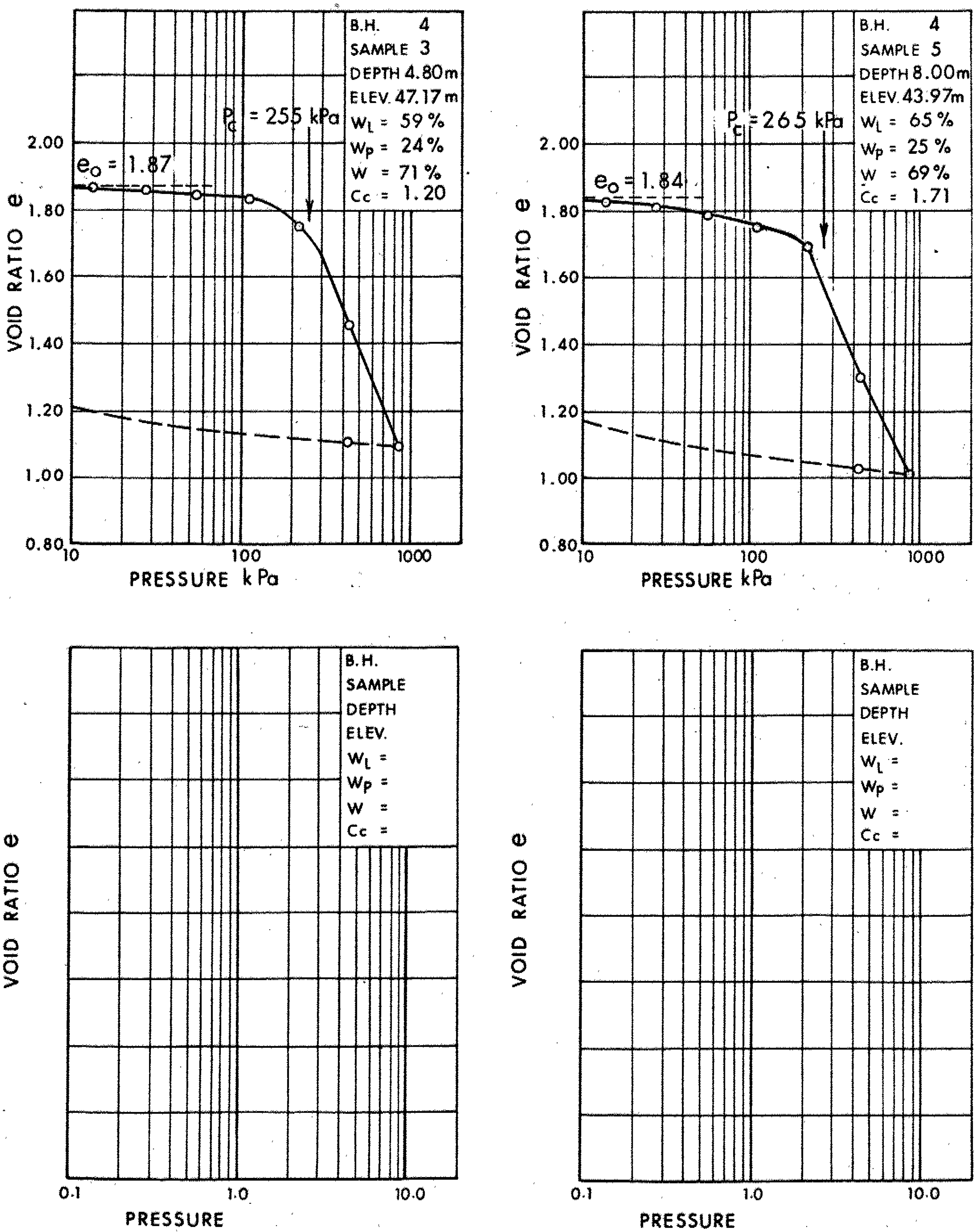
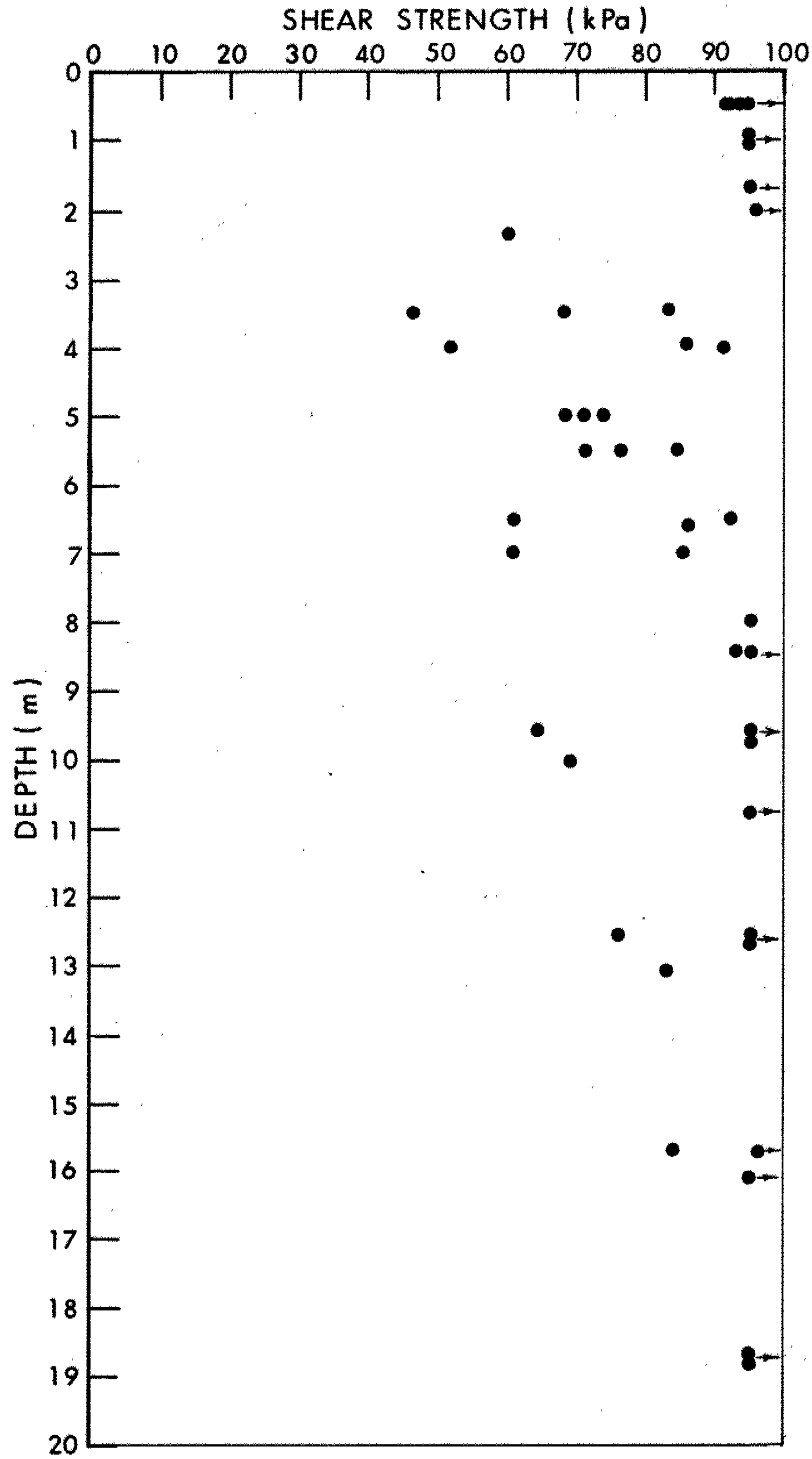


FIG. 5

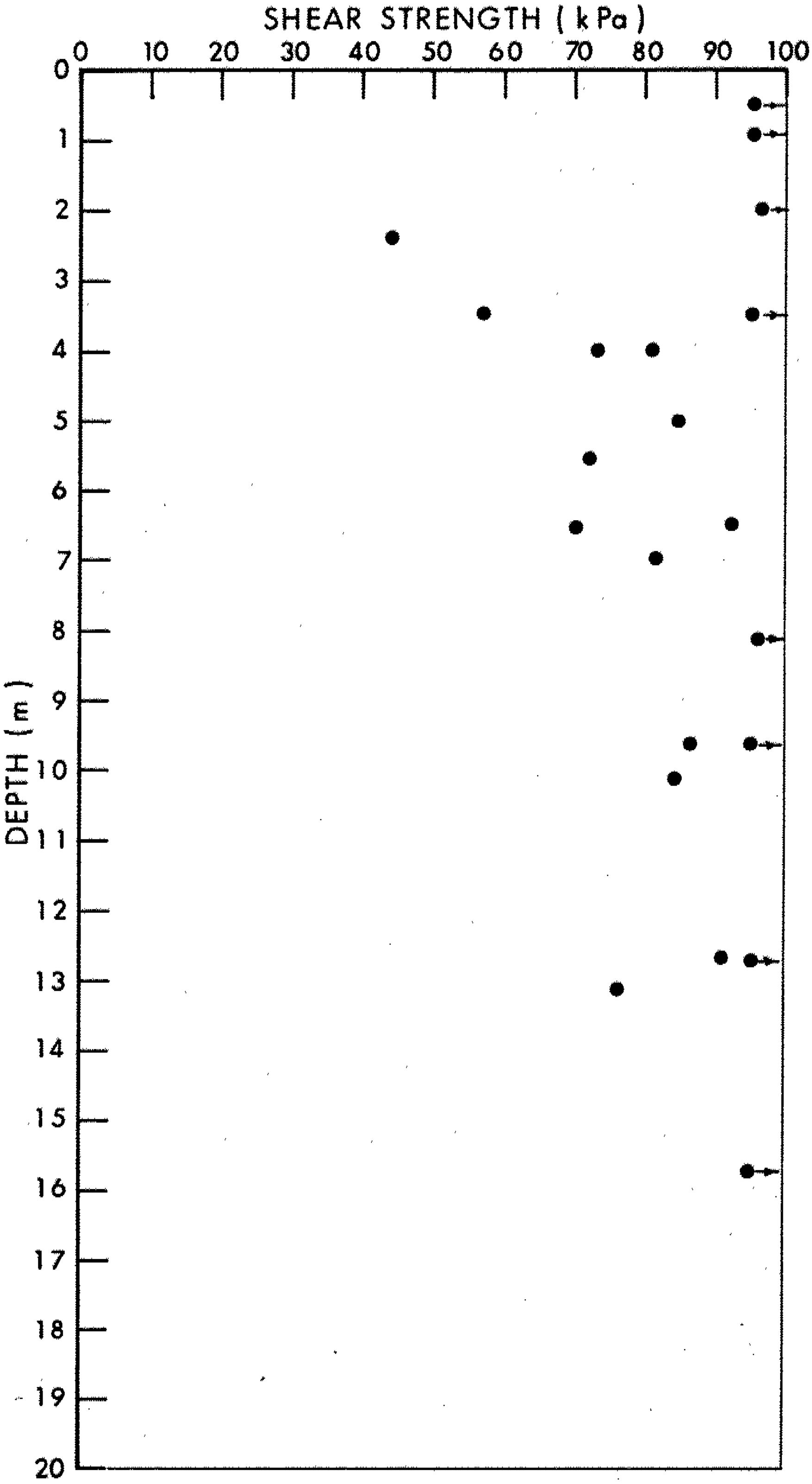


SHEAR STRENGTH vs DEPTH SUMMARY

VICINITY OF GREEN'S CREEK

WP 911-73-00

FIG No 6

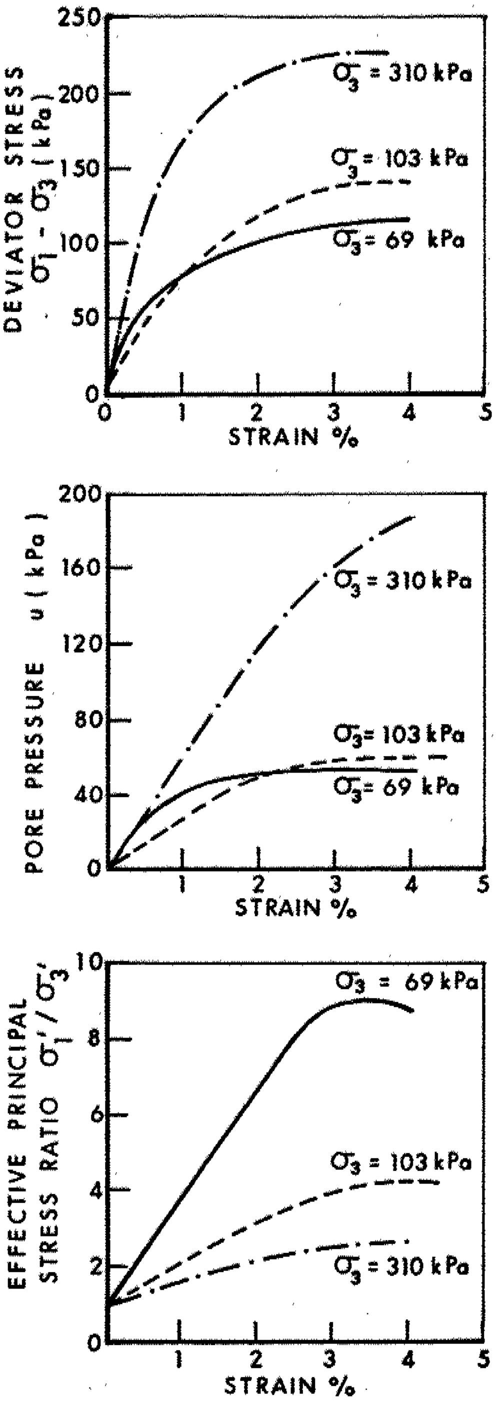


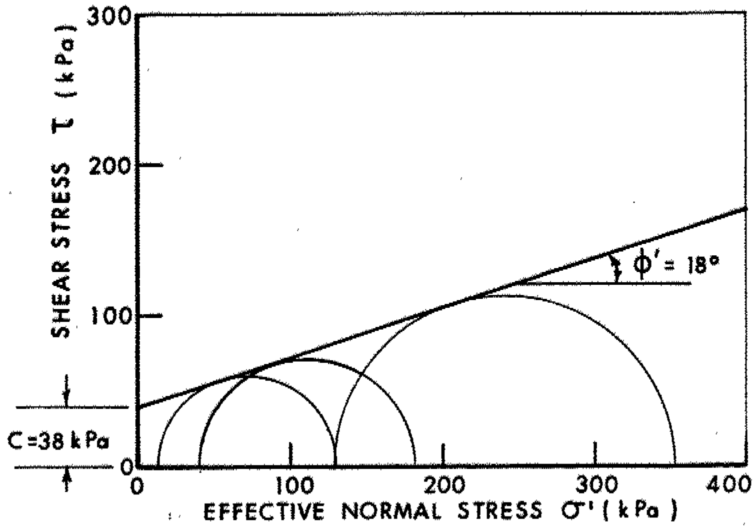
SHEAR STRENGTH vs DEPTH SUMMARY

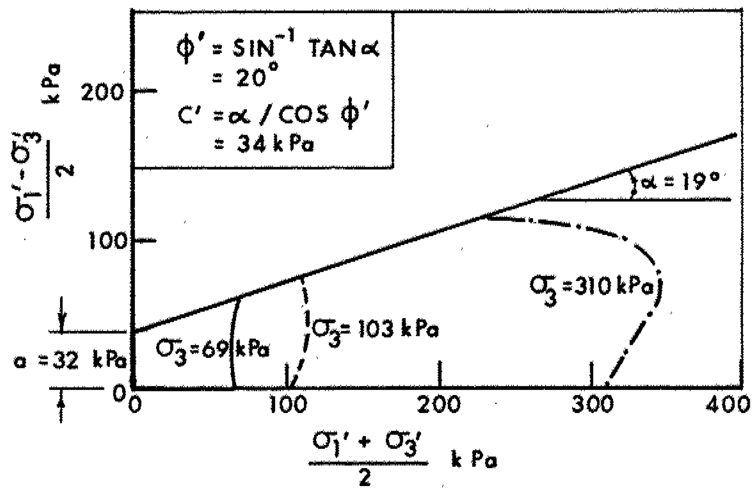
STA 16+600 to STA 17+700

WP 911-73-00

FIG No 7







Bore Hole No 7

Sample No 4

Rate of Strain = 0.0122 mm / min

Sample Size = 50.8 mm ϕ × 101.6 mm high

		σ CONFINING PRESSURE KPa		
		10	15	45
MID SAMPLE DEPTH (m)		6.159	6.337	6.501
MOISTURE CONTENT W%	INITIAL	68.4	69.8	66.9
	FINAL	67.1	68.0	57.6
BULK DENSITY γ kN / m^3	INITIAL	15.72	15.72	15.87
	FINAL	15.87	15.87	16.50
PLASTIC LIMIT Wp %		26		
LIQUID LIMIT W_L %		62		

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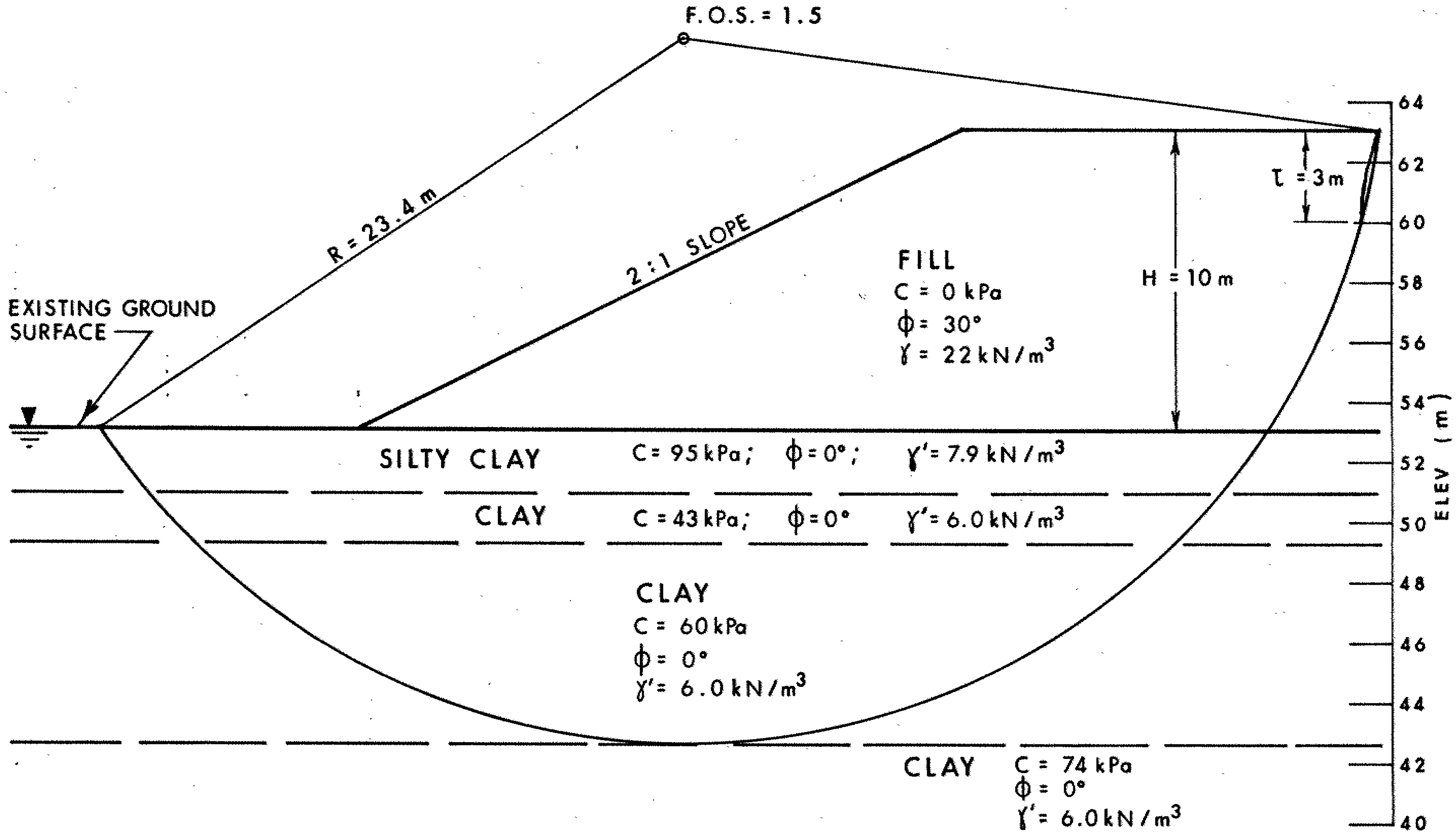
FIG No 8



TOTAL STRESS ANALYSIS

ASSUMED SUB-SOIL STRATIGRAPHY, CRITICAL CIRCLE & FACTOR OF SAFETY FOR FILLS STA 16+600 TO STA 17+700

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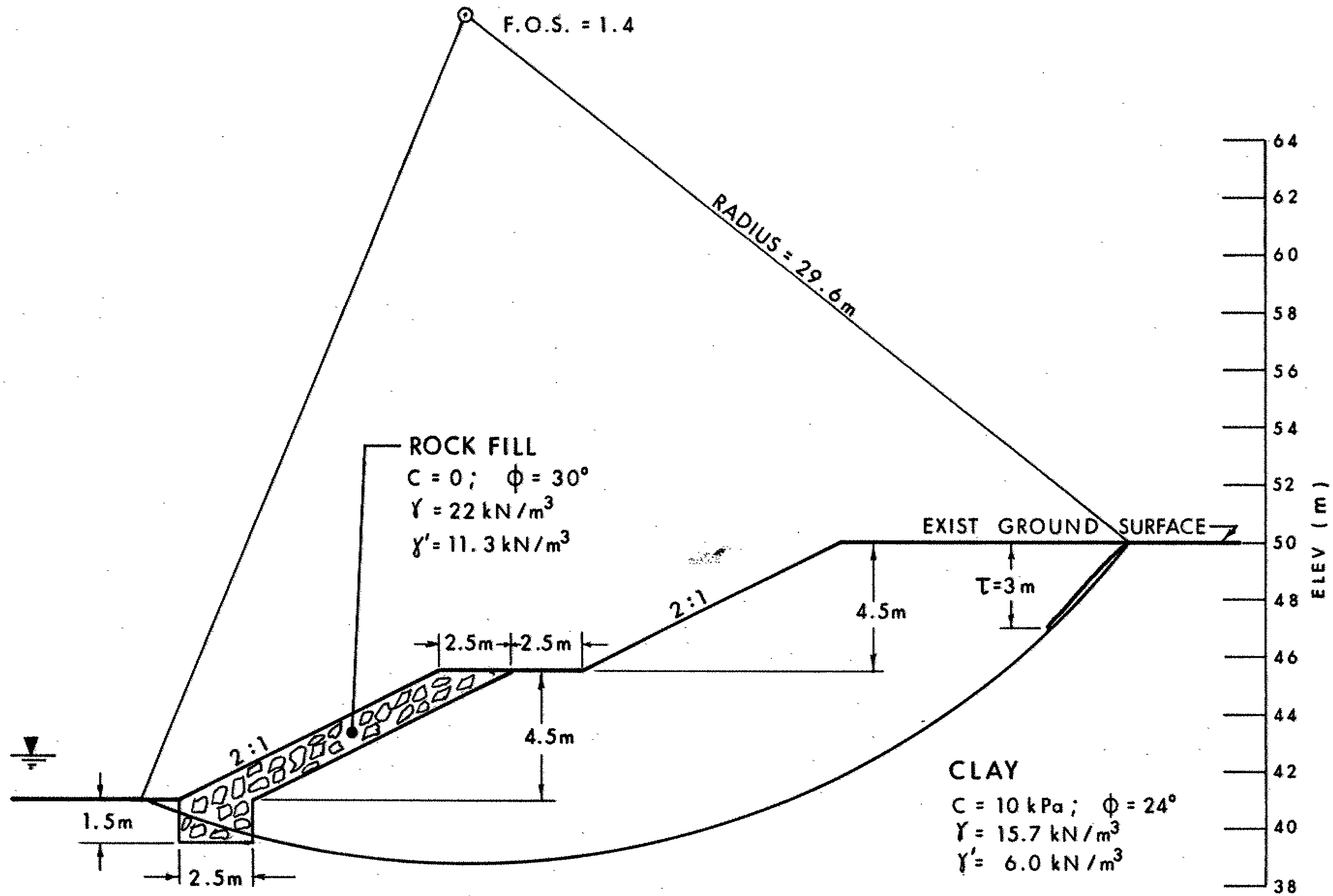
FIG No 9



EFFECTIVE STRESS ANALYSIS

STABILITY OF SLOPES IN VICINITY OF GREEN'S CREEK

FIG No 10

WP 911-73-00

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/0.3 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	-	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	1,%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	1,%	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	1,%	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	-	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	-	WATER CONTENT
S_r	1,%	DEGREE OF SATURATION
w_L	-	LIQUID LIMIT
w_P	-	PLASTIC LIMIT
w_S	-	SHRINKAGE LIMIT
I_P	-	PLASTICITY INDEX $= w_L - w_P$
I_L	-	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	-	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1,%	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Table 1
Summary of Borehole Data

	BH #	Location	Ground Elevation	Subsoil Description	Ground Water Elev.
For Green's Creek	1	Sta 16+276.0 WBL o/s 21.00m Lt	44.30	0-32 metres Stiff to V.Stiff silty clay to clay	41.9
				32-40 metres Glacial Till Hard or V. Dense	ARTESIAN CONDITIONS ENCOUNTERED AT ELEV. 10.2 with 2 meter head above GROUND SURFACE
				40 + Limestone Bedrock	
	2	Sta 16+332.3 WBL o/s 7.00 m Rt	49.14	0 - 37 metres Firm to V. Stiff silty clay to clay	44.6
				37 - 41 metres Glacial Till Hard or V. Dense	
				41 + Limestone Bedrock	
For Eastern Parkway	3	Sta 16+492.0 WBL o/s 0.20 m Rt	49.83	0 - 18+ metres Stiff to V.Stiff silty clay to clay	46.8
	4	Sta 16+697.0 WBL o/s 3.30 m Rt	51.97	0 - 45 metres Firm to V.Stiff silty clay to clay	51.1
				45 - 54 metres Glacial Till Hard or V. Dense	
				54 + Limestone Bedrock	
For Farmer's Overpass	5	Sta 17+256.9 WBL o/s 13.70 m Lt	53.07	0-36 metres Firm to V. Stiff silty clay to clay	NOT ESTABLISHED
				36 + Limestone Bedrock	
	6	Sta 17+404.1 WBL o/s 16.00 m Lt ↕ WRONG LOCATIONS STR. OFFICE NOTIFIED	53.44	0-35 metres Firm to V. Stiff silty clay to clay	NOT ESTABLISHED
				35 + Limestone Bedrock	
	7	Sta 17+612.0 WBL o/s 7.00 m Rt	53.15	0-30 metres Firm to V. Stiff silty clay to clay	52.7
				30 + Limestone Bedrock	
Culvert Extension	8	Sta 20+707.3 WBL o/s 14.00 m Lt	50.83	0-3.5 metres Compact to Dense silt, sand and gravel with occassional cobbles and some Boulders	50.3
				3.5-4.1 Weathered Bedrock	
				4.1 + Sound Limestone Bedrock	