G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31F-109

DIST. 9 REGION

W.P. No. 34-81-03

CONT. No. 93-31

W. O. No.

STR. SITE No. 3-569

HWY. No. 17

LOCATION Hwy 17 & McGee Rd. Underpass

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

Dominion Soil Investigation Inc.
Consulting Engineers

DOMINION SOIL

FOUNDATION INVESTIGATION
PROPOSED UNDERPASS STRUCTURE
HIGHWAY 17 (417) AND McGEE ROAD
SITE 3-569, W.P. 34-81-03
DISTRICT 9
TOWNSHIP OF WEST CARLETON, ONTARIO

CONT 93-31

REF. NO. 89-11-14

MAY 1990

PREPARED FOR:

MINISTRY OF TRANSPORTATION
FOUNDATION DESIGN SECTION
CENTRAL BUILDING
1201 WILSON AVENUE
DOWNSVIEW, ONTARIO
M3M 1J8

ATT: MR. M.S. DEVATA, P. ENG.

GEOCRES # 31F-109

DISTRIBUTION:

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104 Crockford Blvd.
Scarborough, Ontario
M1R 3C6
(416) 751-6565
Fax: (416) 751-7592

440 Phillip Street
Waterloo, Ontario
N2L 5R9
(519) 885-6220
Fax: (519) 885-3490

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London, Ontario
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(519) 681-2400
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P7C 5G8
(807) 623-2929
Fax: (807) 623-1792

17-1173 Michener Rd.
Sarnia, Ontario
N7V 4G5
(519) 337-5409
Fax: (519) 337-2514

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APPENDICES

ENCLOSURES

1.0 INTRODUCTION

Dominion Soil Investigation Inc., Consulting Geotechnical Engineers, was retained by the Ontario Ministry of Transportation to carry out a foundation investigation for a proposed underpass structure at Highway 17 (417) and McGee Road in the Township of West Carleton (west of Ottawa). The proposed bridge structure will have two equal spans of approximately 31 metres to support two 3 m lanes with 1 m wide shoulders.

The purpose of this investigation has been to determine the subsurface condition at the site; to establish the engineering properties of the substrata; and to provide recommendations for the design and construction of the proposed bridge from a geotechnical engineering standpoint.

The field work was carried out during the period of December 4 to 12, 1989 and consisted of ten sampled boreholes and three dynamic cone penetration tests (augered through dense strata). The plan locations of boreholes and cone tests, and stratigraphic sections are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including results of in situ testing, are presented on the Record of Borehole sheets. The results of field and laboratory work, including our interpretation and recommendations are presented in this report.

2.0 SITE DESCRIPTION & PHYSIOGRAPHY

The proposed underpass site is located approximately 30 km west of Ottawa, at the intersection of Highway 17 and McGee Road. The topography in the general area is flat, with a mild relief to the south. The existing road pavement is about 1.2 to 1.5 m higher than the adjacent general ground surface level. The south-western (construction north) quadrant of the site has a light bush cover. The remaining areas have tree growth, outside the right-of-way limits. At the time of our field work the general site area was snow covered.

In general, the Ottawa Region is known to be underlain by 2 to 4 m thick fine grained sand followed by sensitive marine clays. These clays, known as Champlain Sea or Leda clays, are believed to have been deposited during the late stages of the Wisconsin glaciation when the Champlain Sea invaded a significant portion of the South-eastern Ontario including the St.Lawrence lowlands and the Ottawa Valley. It is believed that while these clays were deposited in a marine environment, the influx of fresh waters from Lake Ojibway-Barlow, due to isostatic rebound, progressively decreased the salinity of the regressing sea, some 12 to 15 thousand years ago. This gave way to a leached, open, flocculated structure of the marine clay. Consequently, these clays are characterized by generally high sensitivity, low shear strength and a high compressibility beyond a threshold stress range. While the

.../...

thickness of these clays can be extensive at most areas, it can also vary significantly, generally depending on the elevation of the ground and bedrock. Relatively thin deposits occur west of Stittsville where bedrock outcrops and granular deposits are frequent. Between the bedrock and the clay a layer of glacial till deposit is also commonly encountered.

The bedrock in the Region is known to consist of a faulted sequence of limestones and shales of the Ordovician Period. Published information shows that at the intersection of Highway 17 and McGee Road the bedrock consists of interbedded sublithographic to fine crystalline limestone and calcarenite, known as the Bobcaygeon Formation of the Upper Ordovician Period of the Palaeozoic Era. The intersection however is very close to the interface of other limestone deposits known as Gull River and Verulam Formations. All of these formations are known to belong to the Simcoe (Trenton-Black River) Group.

3.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at ten borehole locations and were inferred at the locations of three dynamic cone penetration tests. The locations of the boreholes and cone penetration tests are shown on the Plan and Profile Dwg. No. 1.

Details of the stratigraphy encountered in the boreholes are given on the individual record of Borehole Sheets. The subsurface conditions can be summarized as follows:

The site is generally underlain, below a 1 to 2 m thick fill deposit, by a 0.2 to 0.7 m thick layer of topsoil and/or somewhat organic sand and silt which are in turn underlain by a deposit of stratified silty sand extending to depths ranging between 2.9 and 5.6 m below the ground surface. These surficial soils are underlain by a major deposit of silty clay with sand and silt interbeds. This deposit is 4.5 to 7.4 m thick and is underlain at some of the borehole locations by a 0.2 to 2.7 m thick gravelly sand deposit immediately above the bedrock. The surface of the limestone bedrock was contacted or inferred at depths ranging between 10.2 and 13.1 m below the ground surface or between Elevations 109.1 and 106.1 m indicating that it generally slopes down from west to east with an elevation difference of 3 m over a horizontal distance of about 100 m.

The individual strata are briefly described in the following paragraphs.

.../...

a) **Fill:** Pavement materials consisting of asphaltic concrete over a thin granular base were encountered in the majority of the boreholes. These were found to overlie a sub-base fill deposit of brown sand with some gravel extending to a depth of 1.1 to 2.1 m below the ground surface.

b) **Topsoil:** Beneath the granular fill Boreholes 1 and 4 contacted a 0.2 to 0.3 m thick layer of topsoil.

c) **Organic Sand and Silt:** Underlying the fill and/or topsoil a deposit of dark brown to grey sand or silt with organics was encountered in Boreholes 1, 101, 103, 4 and 5. At the borehole locations this deposit extends to depths ranging from 1.5 to 2.1 m below the ground surface and its thickness ranges from 0.2 to 0.4 m. From 'N'-values of between 7 and 50 blows/0.3 m, the deposit is described as loose to dense.

d) **Silty Sand:** Below the surficial fill and organic materials, a stratum of silty sand was encountered in all the boreholes ranging in thickness from 1.4 m (B.H.5) to 4.4 m (B.H.102), i.e. to depths ranging between 2.9 and 5.6 m below the ground surface, respectively.

These sands are believed to have been deposited by fluvial activity after the withdrawal of the Champlain Sea. The contact between the sand and the underlying clay is rather gradational and is not well defined.

The material was moist to wet and moisture content determinations carried out on samples from this material measured values ranging from 15 to 22%. Grain-size analyses carried out on selected representative samples showed that the material is comprised of 49 to 65% sand, 21 to 41% silt and 10 to 14% clay size particles. (Fig. No. 1).

Standard penetration resistances, 'N'-values, measured in this deposit randomly vary between 14 and 81 blows/0.3 m and based on these values this deposit is described as compact to very dense.

e) **Silty Clay:** A major deposit of silty clay to clayey silt, interbedded with silt and sand, was encountered in all the boreholes directly below the silty sand stratum. The thickness of this deposit, including the interbeds and lenses ranges from 4.5 m (B.H.102) to 7.4 m (B.H.103). These interbeds range in thickness from several millimetres to 3.4 m.

Atterberg Limits tests carried out in the laboratory gave the following index values:

Liquid Limit: 18 - 39%
Plastic Limit: 14 - 24%
Plasticity Index: 4 - 15

These values are characteristic of clayey soils of low to intermediate plasticity. The lower plasticity indices generally represent the silt and clayey silt zones in the deposit. The measured moisture contents range from 20 to 48% and are generally near or above the liquid limits.

The undrained in-situ shear strength of the silty clay as measured by field vane tests ranges from 17 to 179 kPa and are generally in the 25 to 50 kPa range. The variation of in-situ shear strength with elevation is plotted in Figures No. 9 and 10. Several undrained quick triaxial tests were also performed in the laboratory to determine the undrained shear strength of the soil but these are not considered to be representative of the actual field strengths due to the failure of the test samples through the random silt and sandy silt to silty sand zones and lenses that were present in the samples.

The unique properties of the Champlain Sea clay are its blocky, fissured structure and its extreme sensitivity to remoulding. The sensitivity of the soil, determined as the ratio of the peak shear strength to the remoulded shear strength of the in-situ vane test data ranges from 1.3 to 16.7 with an average value of 5.5. In general however the measured sensitivity ranges from 3.5 to 8.0 and these values indicate a generally low to sensitive clay.

The compressibility and consolidation characteristics of the clay were determined in the laboratory by conventional oedometer tests. The test results are shown in Fig. 6 and 7 which suggest that the clay is slightly preconsolidated and highly compressible.

The measured bulk unit weight of the soil ranges between 18 and 20 kN/m^3 .

The clay is frequently interlayered and interbedded with silt and sand seams or lenses of various thicknesses, ranging from several millimetres to several metres in thickness. In some instances the thicknesses of such zones showed great variations in between boreholes drilled close to each other (e.g. Boreholes 3 and 103) indicating random deposition modes. The grain size distribution of samples from these silt and sand zones are presented on Fig. Nos. 2, 3 and 4.

Standard Penetration resistances measured in these silt and sand interbeds gave 'N'-values of 7 to over 50 blows/0.3 m advance indicating a highly variable relative density ranging from loose to very dense. These zones/layers were found to be wet and water bearing.

f) **Silty Sand:** A 0.2 to 2.7 m thick lower sand stratum with silt and gravel content was contacted in the majority of the boreholes immediately overlying the bedrock.

A grain-size distribution analysis performed on a sample from this deposit showed 25% gravel, 51% sand, 20% silt and 4% clay size particles (Fig. No.5). This deposit was wet and from 'N'-values of 11 to 22 blows/0.3 m it is considered compact.

g) **Bedrock:** Bedrock was proven by diamond drilling and rock coring in Boreholes 1, 2, 3 and 103, and it was inferred from refusal to augering or dynamic cone penetration tests at the other exploration locations.

At the proposed abutment locations, the surface of the rock was contacted at depths ranging between 10.7 m below the ground surface (or at Elevation 108.7 m, at Borehole 101) at the west abutment location and 13.1 m (or Elevation 106.1 m, at Borehole 2) at the east abutment location. This indicates that the surface of the rock is relatively level with an elevation drop of 2.6 m from west to east over a horizontal distance of 62 m.

The rock was cored for a vertical distance of 3.0 m, 3.1 m, 1.2 m and 1.9 m at Boreholes 1, 2, 3 and 103, respectively. The core samples show that the rock consists of grey limestone with frequent highly argillaceous zones and thin shale seams. It is generally horizontally bedded and does not contain major fractures or solution cavities where it was cored. The percentage of recovery of the rock cores ranged from 89 to 97% and R.Q.D. values of between 13 and 40% were recorded. From these observations and high percentage of recovery, the rock is described as generally sound. The low R.Q.D. values however indicate that it is of poor quality mainly due to the presence of weak shale zones.

.../...

4.0 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. Standpipe piezometers were installed in Boreholes 6 and 101 to enable us to monitor the groundwater levels over a prolonged period of time without interference from surface water. In each of these boreholes two piezometers were installed, one at a depth of 3 m in the upper sand deposit and second one at 6 m depth in the underlying soil strata.

The recorded values, presented on the individual borehole log sheets, indicate that there is a perched water table in the upper sand which at the time of our investigation was generally at a depth of about 1.8 to 2.0 m below the ground surface. The water level in the underlying clay was generally 3 to 4 m below the ground surface.

5.0 DISCUSSION & RECOMMENDATIONS

Ontario Ministry of Transportation is planning the extension of the Highway 17 staged freeway from two to four lanes from the junction of Highway 17/417 to west of Highway 44. The freeway will necessitate the construction of a new bridge structure at the presently level crossing of McGee Road. The new underpass structure will have two equal spans of 31 metres. It will support two 3.0 m lanes with 1.0 m shoulders. The approach embankments for abutment construction of the new bridge will be 6.0 m high.

The boreholes have shown, beneath an approximately 1 to 2 m thick granular pavement and sand fill and in some of the boreholes a 0.2 to 0.7 m thick topsoil and/or somewhat organic soil, the presence of a deposit of silty sand extending to depths ranging between 2.9 and 5.6 m below the ground surface. Below these surficial fill and fluvial deposits, the site is underlain by a sensitive silty clay (Champlain Sea) deposit. From the field and laboratory test results, the silty clay is considered generally firm to stiff and highly compressible. This deposit is frequently interlayered with sand and silt lenses which present a variable picture across the site both in terms of thickness and compactness condition. In the majority of the boreholes a 0.2 to 2.7 m thick layer of silty sand

.../...

with some gravel was contacted immediately above the bedrock. The depth to the surface of the limestone bedrock ranges from 10.2 to 13.1 m at the borehole locations.

The groundwater level at the time of the investigation was generally 1.8 to 2.0 m below the ground surface in the upper sand and 3 to 4 m below the ground surface in the underlying silty clay deposit.

5.1 Foundations

The existing fill is underlain at several borehole locations by 0.2 to 0.7 m thick topsoil and/or somewhat organic deposits which in turn are underlain by compact to very dense sand. The thickness of the sand is variable across the site, ranging from 1.4 to 4.4 m. Below this, the site is underlain by a weak and compressible silty clay deposit with some random competent zones. Due to the presence of variable conditions and the high compressibility of the silty clay, normal spread footing foundations can be expected to undergo high differential settlements. Furthermore, the stresses due to the weight of the 6 m high embankment fill can be expected to induce unacceptable settlements.

For these reasons it is our opinion that the proposed structure should be founded on deep foundations on the surface of the bedrock.

The structure could be supported on steel piles driven to refusal on the surface of the bedrock. In this case low displacement steel H-piles would be the most suitable. Driving procedures which will provide good contact between the pile and the bedrock must be adopted. Furthermore, a suitable fill should be used (i.e. free of cobbles and oversized material) for the approach embankment within the zone through which piles will be driven. It is also recommended that the piles have reinforced flanges for improved driving resistance and to reduce damage to pile tips.

The estimated capacities for some common sizes of steel H-piles driven to practical refusal on the surface of the bedrock are tabulated below:

ESTIMATED PILE CAPACITY (kN)

Pile Type	Size	Factored Capacity at Ultimate Limit States (Qf)	Capacity at Serviceability Limit States Type II (Qs)
Steel H	HP 310x110	1520	1070
	HP 310x79	1100	760

.../...

In some cases difficult driving or even refusal may occur in the very dense intermittent sand layers at various depths above the rock level as evidenced by dynamic cone refusal (e.g. Cone Test A). In such instances the dense layer must be penetrated to the surface of the bedrock.

Stresses due to the weight of the approach embankment will consolidate the clay stratum which will then transfer loads by negative skin friction to the piles (i.e. will cause down-drag). Our calculations indicate that a negative skin friction value of 30 kPa should be applied around the perimeter of the pile below the pile cap. Down-drag forces need not be considered for the central pier.

The settlement of the fill will also cause a vertical loading on the piles due to friction forces between the backfill and the abutment wall as the fill settles. This load can be distributed equally among all the piles in the rear row.

.../...

These forces can be neglected if the fill can be placed at least about one year before pile driving commences. Such prior filling would also minimize settlement of the road surface after the end of construction.

Unbalanced horizontal forces should be resisted by battered piles. For frost protection the pile caps should be located at least 1.8 m below the finished grade.

5.2 Lateral Earth Pressures

For retaining walls, the lateral earth pressure can be calculated using the active earth pressure and the following equivalent fluid pressures as per OHBDC 6-6.1.2.2:

At Ultimate Limit States: 8.0 kPa/m
At Serviceability Limit States: 6.5 kPa/m

The rigid abutment walls should, however, be designed to withstand the at-rest earth pressures, provided that the backfill is not heavily compacted (in which case much higher earth pressures could occur). For the at-rest earth pressure condition, the following equivalent fluid pressures can be used:

.../...

At Ultimate Limit States: 10.0 kPa/m

At Serviceability Limit States: 8.5 kPa/m

When using these values, it is assumed that the slope of the backfill behind the retaining structure is approximately level and free-draining granular material and adequate drainage have been provided. Water accumulation in the backfill behind the retaining structures should be prevented by means of perforated pipes or weep holes.

The over-compaction of the backfill could lead to the development of large horizontal pressures behind the retaining walls and the abutments. Vibratory compaction equipment for use behind the abutment walls and the retaining walls must therefore be restricted in size as per current M.T.O. Specifications.

5.3 Approach Fills

We understand that up to 6 m high approach fills will be constructed. These will be approximately 10 m wide with 2:1 side slopes.

With this configuration, the stability of the approach fills was investigated by means of conventional limit equilibrium methods along assumed circular arc failure surfaces using the following assumed and/or measured soil parameters:

Drained Stability Analysis:

Embankment Fill:	ϕ' = 32 degrees c' = 0 γ = 22 kN/m^3
Sand:	ϕ' = 30 degrees c' = 0 γ = 20 kN/m^3
Silty Clay:	ϕ' = 26 degrees c' = 10 kPa γ = 17 kN/m^3

Undrained Stability Analysis:

Embankment Fill:	ϕ = 32 degrees c = 0 γ = 22 kN/m^3
Sand:	ϕ = 30 degrees c = 0 γ = 20 kN/m^3
Silty Clay:	ϕ = 0 c = 30 kPa γ = 17 kN/m^3

Based on these parameters, the minimum calculated factor of safety against a failure is 1.5 which is considered to be adequate. The sensitive clay may however lose some of its shear strength due to vibrations and disturbance during pile driving. For this reason the use of low displacement piles is recommended.

All organic and other unsuitable soils must be removed before placing the fill. The exposed subgrade should be inspected and the approved subgrade should be compacted using a suitably heavy compactor.

The slopes of the embankment should be adequately protected against surface erosion.

It is estimated that a settlement of the order of 200 mm will occur due to the weight of the new embankment. Somewhat greater settlements could occur if the grade is raised beyond the width of the existing road embankment which is presently about 1.0 to 1.5 m higher than the surrounding ground. We have however assumed for both settlement and stability calculations that the new embankment will be placed over the existing road and that maximum grade above existing elevation will be 6.0 m.

Based on the consolidation test results performed in the laboratory, 90% of the settlement can be expected to take place within about one year and 50% within about three months. The effects of the embankment fill on the piles supporting the abutments, as discussed before, and the magnitude of the settlements after the completion of the road can therefore be reduced if the fill is placed several months before the pile driving and pavement construction.

The paving of the road should be delayed until the settlement at the interface of the embankment and the road fill has considerably decreased.

5.4 Construction

Where the excavations extend below the groundwater table, the sandy soils will be unstable and will require dewatering to stabilize the sides and to preserve the load carrying capability of the soil at the base of the excavation. The groundwater table at the time of the investigation was generally 1.8 to 2.0 m below the ground surface. It would however likely be higher during rainy periods or spring run-off.

Depending on the depth of excavation below the water table, dewatering could be effected by pumping from filtered sumps. This will however be effective only about 0.3 to 0.4 m below the water table and for excavations extending more than about 0.4 m below the water table more sophisticated methods would be required to dewater the site and to stabilize the soil. It is unlikely however that excavations extending more than 2 m below the existing grades will be required for this project.

Above the groundwater table, temporary excavations more than 1.2 m deep must be sloped at 1:1 side slopes or flatter or adequately supported in accordance with the Safety Regulations of the Province. Below the groundwater table much flatter side slopes would however be necessary in the cohesionless sand deposits.

6.0 CLOSURE

The Limitations of Report as quoted in the Appendix 'B', are an integral part of this Report.

DOMINION SOIL INVESTIGATION INC.

Z.S. Ozden, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

R. Miranda, P. Eng.

APPENDICES

APPENDIX 'A'

PROCEDURES

The field work for this project was carried out during the period of December 4 and 12, 1989. During this period a total of 10 boreholes and three dynamic cone penetration tests were put down at the locations shown on Drawing No. 1.

The boreholes were extended to depths ranging between 10.2 m (B.H.5) and 16.2 m (B.H.2). Six of the boreholes were terminated after encountering practical refusal on the inferred bedrock surface. At four borehole locations, the bedrock was proven by core drilling.

The boreholes were advanced by a power auger drilling rig. In boreholes where rock was to be cored, hollow stem augers were used. Sampling was effected at frequent intervals of depth by Standard Penetration test method. In the cohesive strata thin walled open drive (shelby) tube samples were obtained. In addition the undrained shear strength of the soil was measured in-situ by field vane tests. At four borehole locations, after refusal, the rock was cored, by diamond drilling, using 'NXL' size core barrel.

Dynamic cone penetration tests (A, B & C) were performed at three selected locations. In some cases, however, the presence of very dense layers necessitated augering through these strata to advance to the inferred bedrock surface.

The borehole and cone test locations were established in the field with the assistance of the M.T.O surveyors. Ground surface elevations at the borehole locations were determined and provided to us by Mr. D.J. Caldwell of the Ministry.

The drilling equipment was owned and operated by D.S.I.L. Drilling Inc. and the field work was carried out under the supervision of a Professional Engineer from Dominion Soil Investigation Inc. Upon completion of the field work the soil and rock samples were shipped to our laboratory, for further examination and classification. A laboratory testing programme consisting of natural moisture content, Atterberg Limits, consolidation, quick triaxial compression tests and grain-size analyses was performed on selected soil samples.

APPENDIX 'B'

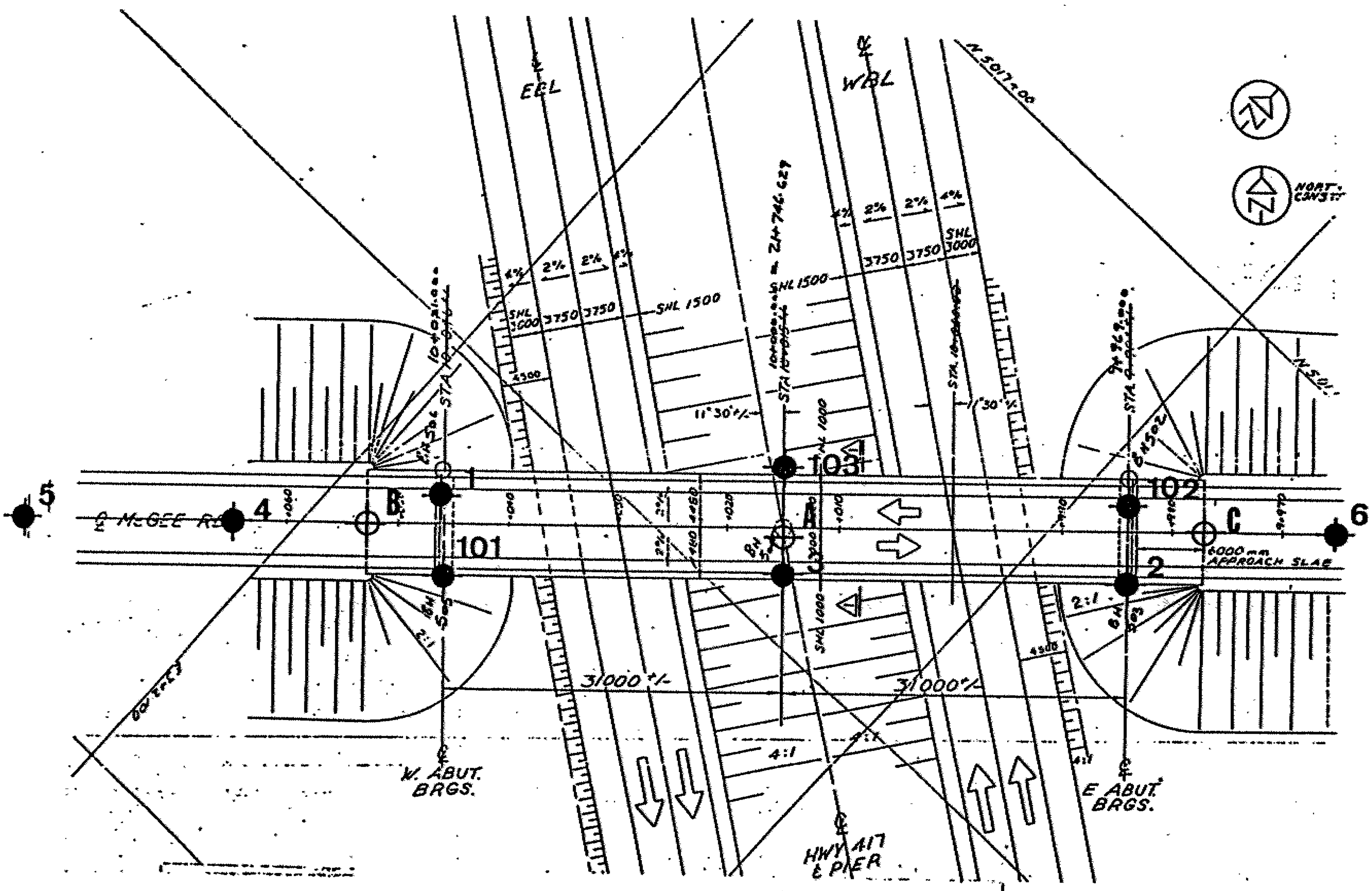
LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work.

ENCLOSURES



SITE PLAN

Ref. No. 89-II-14

Drawing No. 1
WP 34-81-03

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 1 — METRIC

W P 34-81-03 LOCATION Sta. 10 + 031, O/S 1.8m RT ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, NXL Rock Core COMPILED BY AAK

DATUM Geodetic DATE 89 12 05 CHECKED BY ZSO

ELEV / DEPTH	DESCRIPTION	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
119.2	GROUND LEVEL									
0.0	0.1 m Crushed Stone									
	FILL brown sand, some gravel	1	AS	-						
117.8		2	SS	15*		118				*Possible frost
1.4	TOPSOIL				▽					
1.7	SAND somewhat organic	3	SS	7						
2.1	Loose, grey									
	SILTY SAND dense, grey, moist to wet	4	SS	32						
115.6		5	SS	35		116				
3.6	silty sand interbeds	6	SS	5						
		7	TW	PH		114	s=4.8		18.3	Consolidation Test
	SILTY CLAY soft to stiff grey	8	SS	8			s=11.3; s=8.1			
		9	SS	9		112	s=3.5			
		10	TW	PH			s=3.0; s=9.0		18.0	Consolidation Test
		11	SS	4		110	s=5.0; s=6.0			
108.1 / 11.1	some gravel	12	TW	PH		108				
	grey Limestone BEDROCK	13	NXL RC	REC 97%		106				RQD = 40%
105.1 / 14.1	End of Borehole									

TIME	W.L. (m)
Dec.5/89 (completion)	1.5
Dec.6/89	1.5

+3, ×5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 2 — METRIC

W P 34-81-03 LOCATION Sta. 9 + 969, 0/S 4.8 LT ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, NXL Rock Core COMPILED BY AAK

DATUM Geodetic DATE 89 12 06 CHECKED BY ZSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	SHEAR STRENGTH kPa	WATER CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.2 / 0.0	GROUND LEVEL; 0.05 m ASPHALT; 0.08 m Granular Base; FILL Sand, some gravel brown	1	AS	-				
		2	SS	52				
117.4 / 1.8	SILTY SAND brown / grey with silt seams compact to v. dense wet	3	SS	32				
		4	SS	56				
		5	SS	81				
115.6 / 3.6	SILTY CLAY firm, grey	6	SS	7	+ s=8.0			
		7	TW	PH	+s=5.0; + s=6.3		19.8	Consolidation Test
113.2 / 6.0	SILTY SAND some clay grey Loose / V.dense	8	SS	7				53 33 14
		9	SS	78				62 26 12
110.4 / 8.8	SILTY CLAY soft to stiff, grey	10	SS	38*	+s=1.5			* rods rebounding, spongy
108.3 / 10.9	SILTY SAND some gravel compact, grey	11	SS	22				25 51 20 4
106.1 / 13.1	grey Limestone BEDROCK	12	NXL RC	REC 93%				RQD = 14%
103.0 / 16.2	End of Borehole							

Elevation scale: 118, 116, 114, 112, 110, 108, 106, 104

Dynamic cone penetration resistance plot: 20 40 60 80 100

Shear strength kPa: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125

Plastic limit W_p, Natural moisture content W, Liquid limit W_L; Water content (%) 20 40 60

TIME	W.L. (m)
Dec.6/89 (completion)	1.4
Immediately after rock coring	

OFFICE REPORT ON SOIL EXPLORATION

+3, ×5 : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 3

METRIC

W P 34-81-03 LOCATION Sta. 10 + 000 O/S 4.8 LT ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, NXL Rock Core COMPILED BY AAK

DATUM Geodetic DATE 89 12 07 CHECKED BY ZSO

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	SAMPLES: NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.6	GROUND LEVEL									
0.0	0.05m Asphalt; 0.08m Granular Base	1	AS	-						
	FILL sand, some gravel brown	2	SS	50/15cm						
117.5 / 2.1		3	SS	29	▽	118				
	SILTY SAND compact to very dense grey, wet	4	SS	36						
		5	SS	54		116				
114.9		6	SS	26						
114.5 / 5.1	SILTY CLAY firm grey	7	SS	7			+ s=3.6; + s=2.7			
	SANDY SILT with clay seams compact grey	8	SS	14		114				0 47 41 12
112.5 / 7.1	SILTY SAND very dense, grey	9	SS	50/15cm		112				
111.1 / 8.5	SILTY CLAY firm to stiff, grey	10	SS	wt. of rods		110	+ s=2.7; + s=3.5			
108.0 / 11.6		11	TW	PH			●; + s=10.6	o	17.4	
	grey Limestone BEDROCK	12	NXL RC	REC 89%		108				RQD = 18%
106.8 / 12.8	End of Borehole									

TIME	W.L.
Dec.7/89 (completion)	1.8

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

$15 \overset{20}{\underset{10}{\diamond}} 5$ (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 4 METRIC

W P 34-81-03 LOCATION Sta. 10 + 051 @ C/L ORIGINATED BY AAK
DIST 9 HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY AAK
DATUM Geodetic DATE 89 12 08 CHECKED BY ZSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.2	GROUND LEVEL							
0.0	0.05m Asphalt; 0.1 m Granular base; FILL sand, some gravel brown	1	SS	40	118			
117.8 1.4	TOPSOIL							
1.6	SAND somewhat organic compact, grey	2	SS	37				
1.9	SILTY SAND compact to dense, grey	3	SS	25				
		4	SS	43	116			0 49 41 10
114.8		5	SS	30				
4.4	SILTY CLAY soft to stiff grey	6	SS	9*	114			* Spongy rods rebounding
						+ s=5.3; + s=8.0		
		7	SS	7	112	+s=10.0; + s= 4.5		
		8	SS	5		+s=6.4; +s=2.9		
		9	SS	5	110	+ s=4.0; +s=1.3		
108.7	some gravel							
10.5	End of Borehole Refusal to Augering Probable Bedrock							

GROUND WATER CONDITIONS | STRAT PLOT | UNIT WEIGHT γ

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 25 50 75 100 125

PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L — 20 40 60

TIME	W.L. (m)
Dec.8/89 (completion)	2.3

OFFICE REPORT ON SOIL EXPLORATION

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

$15 \overset{20}{\diamond} 5$ (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 34-81-03 LOCATION Sta. 10 + 071 @ C/L ORIGINATED BY AAK
DIST 9 HWY 17 BOREHOLE TYPE Solid Stem Auger & Cone Test COMPILED BY AAK
DATUM Geodetic DATE 89 12 08 CHECKED BY ZSO

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125)	WATER CONTENT (%) (PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.3	GROUND LEVEL									
0.0	0.05m Asphalt									
	0.1 m Granular fill									
	FILL, sand some gravel									
118.2	brown	1	SS	50		118				
1.1	SAND somewhat organic brown									
1.5	SILTY SAND brown gr thin silty clay seams	2	SS	47	▽					
116.4	very dense to compact	3	SS	17						
2.9	SILTY CLAY sand seam very stiff to firm grey trace sand	4	SS	15		116				
		5	SS	13			+s=4.3			
		6	SS	10		114	179+s=3.1; 56+s=1.3			
112.7		7	SS	6			wt. of rods			
6.6	End of Borehole Dynamic Cone Penetration Test performed from 6.6 to 10.2 m SILTY CLAY, soft (INFERRED)					112				
						110				
109.1										
10.2	End of Dynamic Cone Penetration Test Refusal to Dynamic Cone Penetration @ 10.2 m Probable Bedrock									

TIME	W.L. (m)
Dec.8/89 (completion)	1.8

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 6

METRIC

W P 34-81-03 LOCATION Sta. 9 + 949 @ C/L ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY AAK

DATUM Geodetic DATE 89 12 09 CHECKED BY ZSO

ELEV DEPTH	DESCRIPTION	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
119.0	GROUND LEVEL									
0.0	0.08m Asphalt / 0.12m Granular fill / FILL sand, some gravel brown	1	SS	21		118				
117.6 / 1.4	SILTY SAND with silt seams compact, moist to wet; brown / grey	2	SS	26						
		3	SS	17	No.2	116				- 65 21 14
		4	SS	14						
115.4 / 3.6	SILTY CLAY firm to stiff, grey sand interbeds	5	SS	11			+s=6.7			
		6	TW	PH		114	+s=4.3		19.4	
113.4 / 5.6	SILTY SAND some clay compact to very dense, grey	7	SS	22	No.1	112				- 53 32 15
		8	SS	71						
110.4 / 8.6	SILTY CLAY firm to stiff, grey	9	SS	19*		110	+s=2.5 / +s=5.0			* rods rebounding spongy
108.4 / 10.6	SILTY SAND some gravel, compact, grey	10	SS	11		108				
		11	SS	85/20cm						
106.5 / 12.5	End of Borehole (Probable Bedrock)									

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 25 50 75 100 125

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

TIME	W.L. (m) (STANDPIPE) No.1	No.2
Dec.9,89	Dry	2.7
Dec.10/89	Dry	1.8
Dec.11/89 a.m.	4.5	2.3
p.m.	3.9	2.2
Dec.12/89	3.8	2.2

OFFICE REPORT ON SOIL EXPLORATION

+³, ×⁵ : Numbers refer to Sensitivity   15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 7 — METRIC

W P 34-81-03 | LOCATION Sta. 9 + 929 @ C/L | ORIGINATED BY AAK

DIST 9 HWY 17 | BOREHOLE TYPE Solid Stem Auger & Cone Test | COMPILED BY AAK

DATUM Geodetic | DATE 89 12 10 | CHECKED BY ZSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS
118.8	GROUND LEVEL							
0.0	0.05m Asphalt							
	FILL sand some gravel					118		
117.6	brown	1	AS	-				
1.2	SILTY SAND brown	2	SS	65				
	grey	3	SS	47	▼			
	compact to v. dense							
115.8	moist to wet	4	SS	20		116		
3.0	SILTY CLAY firm to stiff, grey	5	SS	14				
		6	SS	11			+ s=6.3	
	silty sand seams	7	SS	5		114		
		8	SS	5				
						112	+ s=7.0	
111.4							+ s=5.3	
7.4	End of Borehole Dynamic Cone Penetration Test performed from 7.4 to 11.0m							
109.8	SILTY CLAY (INFERRED)					110		
9.0	SILTY SAND some gravel (INFERRED)							
107.8						108		
11.0	End of Dynamic Cone Penetration Test Refusal @ 11.0m Probable Bedrock							

Column headings also printed: STRAT PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125; 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

TIME	W.L. (m)
Dec.10/89 (completion)	1.8

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 101

METRIC

W P 34-81-03 | LOCATION Sta. 10 + 031, O/S 4.8 LT | ORIGINATED BY AAK

DIST 9 HWY 17 | BOREHOLE TYPE Solid Stem Auger | COMPILED BY AAK

DATUM Geodetic | DATE 89 12 09 | CHECKED BY ZSO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa / DYNAMIC CONE PENETRATION	WATER CONTENT (%)
119.4	GROUND LEVEL							
0.0	0.05m Asphalt 0.2 m Granular fill FILL sand, some gravel brown							
118.0 1.4	SAND somewhat organic grey					118		
1.6	SILTY SAND brown / grey dense, wet	1	SS	38				
		2	SS	32	No.2	116		
115.4 4.0	Silty sand interbeds						+s=5.0	
	SILTY CLAY firm to stiff, grey	3	TW	PH		114	+s=1.5 +s=2.6	o
		4	SS	8			+s=11.0	
					No.1	112	+s=16.7	
		5	SS	4			+s=2.5 +s=5.0	o
		6	SS	4		110	+s=6.7 +s=3.8	
108.7		7	SS	50/8cm				
10.7	End of Borehole Refusal to Augering and sampling Probable Bedrock							

Column headings: SOIL PROFILE (ELEV DEPTH, 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 kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25, 50, 75, 100, 125); 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

TIME	W.L. (m) (STANDPIPE) No.1	No.2
Dec.9/89	5.2	1.9
Dec.10/89	2.3	1.8
Dec.11/89 a.m.	3.4	1.8
p.m.	3.2	1.9
Dec.12/89	3.2	1.9

OFFICE REPORT ON SOIL EXPLORATION

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

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 102

METRIC

W P 34-81-03 LOCATION Sta. 9 + 969, 0/S 2m RT ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY AAK

DATUM Geodetic DATE 89 12 10 CHECKED BY ZSO

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.1	GROUND LEVEL								
0.0	0.04m Asphalt								
	FILL sand some gravel brown					118			
117.9 / 1.2	SILTY SAND with silt seams v. dense to loose wet; brown / grey	1	SS	46	▽				
		2	SS	64		116			
		3	TW	PH		114			
113.5 / 5.6	SILTY CLAY firm, grey sand interbeds	4	TW	PH			H○	20.9	Consolidation Test
112.4 / 6.7	SILTY SAND some clay v.dense, grey	5	SS	55/15cm		112			
110.9 / 8.2	SILTY CLAY soft to stiff, grey	6	SS	12*		110			* rods rebounding spongy
109.0 / 10.1	SILTY SAND some gravel compact, grey	7	SS	17		108			
106.3 / 12.8	End of Borehole Refusal to Augering Probably Bedrock								

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

TIME	W.L. (m)
Dec 10/89	1.5 (completion)

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 103

METRIC

W P 34-81-03 LOCATION Sta. 10 + 000, O/S 5.6m RT ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger, NXL Rock Core COMPILED BY AAK

DATUM Geodetic DATE 89 12 12 CHECKED BY ZSO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS
119.5	GROUND LEVEL							
0.0	0.05m Asphalt 0.15m Granular fill FILL sand some gravel brown				*			
118.0						118		
1.5 / 1.8	SAND somewhat organic brown	1	SS	43				
	SILTY SAND dense to v. dense (brown / grey)	2	SS	67		116		
115.5 / 4.0								
	SILTY CLAY firm to stiff, grey	3	SS	13		114		
		4	SS	5		112	+ s=7.9 + s=6.4	
		5	SS	3			+ s=5.8 + s=5.0	
		6	SS	4		110	+ s=4.4 + s=5.0	
		7	SS	4				
108.1 / 11.4	SILTY SAND, some gravel					108		
11.7	grey Limestone BEDROCK	8	NXL RC	REC 96%		106		RQD=13%
105.9 / 13.6	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125
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

Note:
Corebarrel jammed and left in borehole from 13.6 to 9.8m depth

*water level not established

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No. A

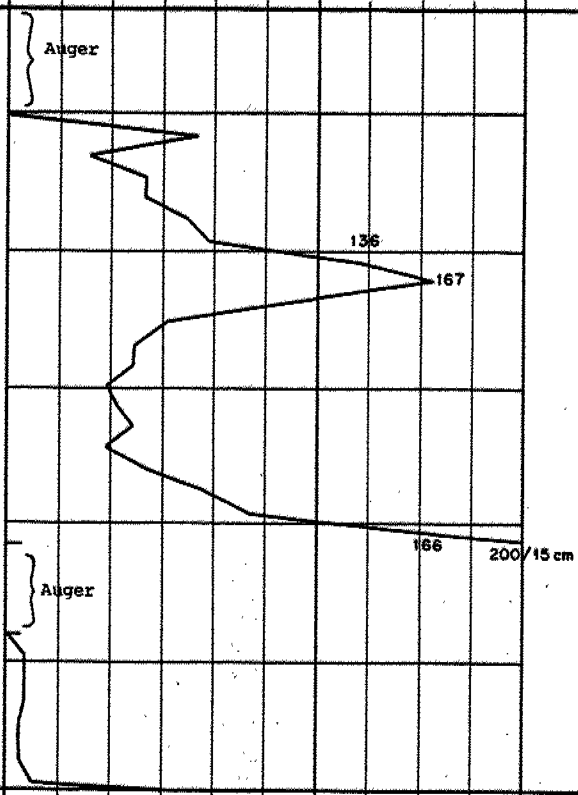
METRIC

W P 34-81-03 LOCATION Sta. 10 + 000, O/S 1.0 LT ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Dynamic Cone Penetration/Auger COMPILED BY AAK

DATUM Geodetic DATE 89 12 08 CHECKED BY ZSO

SOIL PROFILE: ELEV / DEPTH	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 kPa: 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
119.5	GROUND LEVEL										
0.0							118, 116, 114, 112, 110	Auger; 136; 167; 166; 200/15 cm; Auger			
108.1 11.4	End of Dynamic Cone Penetration Test Possible Bedrock										



OFFICE REPORT ON SOIL EXPLORATION

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

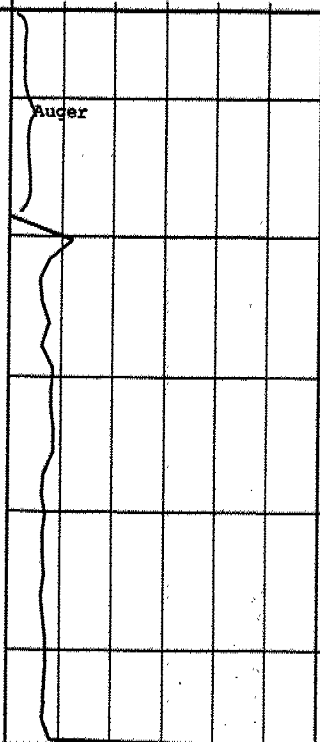
20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No B — METRIC

W.P. 34-81-03 LOCATION Sta. 10 + 037, @ C/L ORIGINATED BY AAK
DIST 9 HWY 17 BOREHOLE TYPE Dynamic Cone Penetration/Auger COMPILED BY AAK
DATUM Geodetic DATE 89 12 09 CHECKED BY ZSO

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 kPa (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
119.3 / 0.0	GROUND LEVEL						118	Auger			
							116				
							114				
							112				
							110				
108.6 / 10.7	End of Dynamic Cone Penetration Test Possible Bedrock										



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

15 -◇- 5 (%) STRAIN AT FAILURE (20 above)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation Ontario

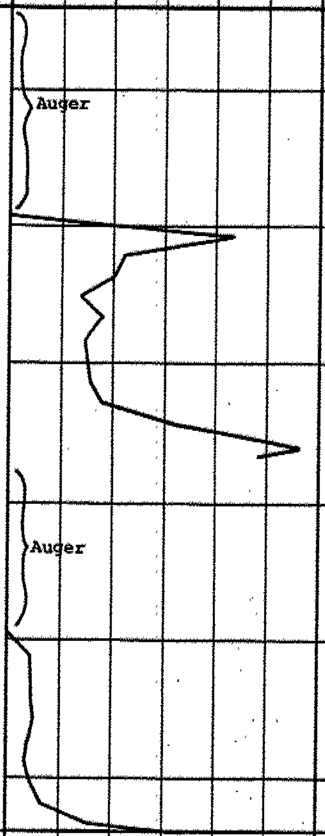
RECORD OF BOREHOLE No C

METRIC

W P 34-81-03 LOCATION Sta. 9 + 963, @ C/L ORIGINATED BY AAK

DIST 9 HWY 17 BOREHOLE TYPE Dynamic Cone Penetration/Auger COMPILED BY AAK

DATUM Geodetic DATE 89 12 10 CHECKED BY ZSO

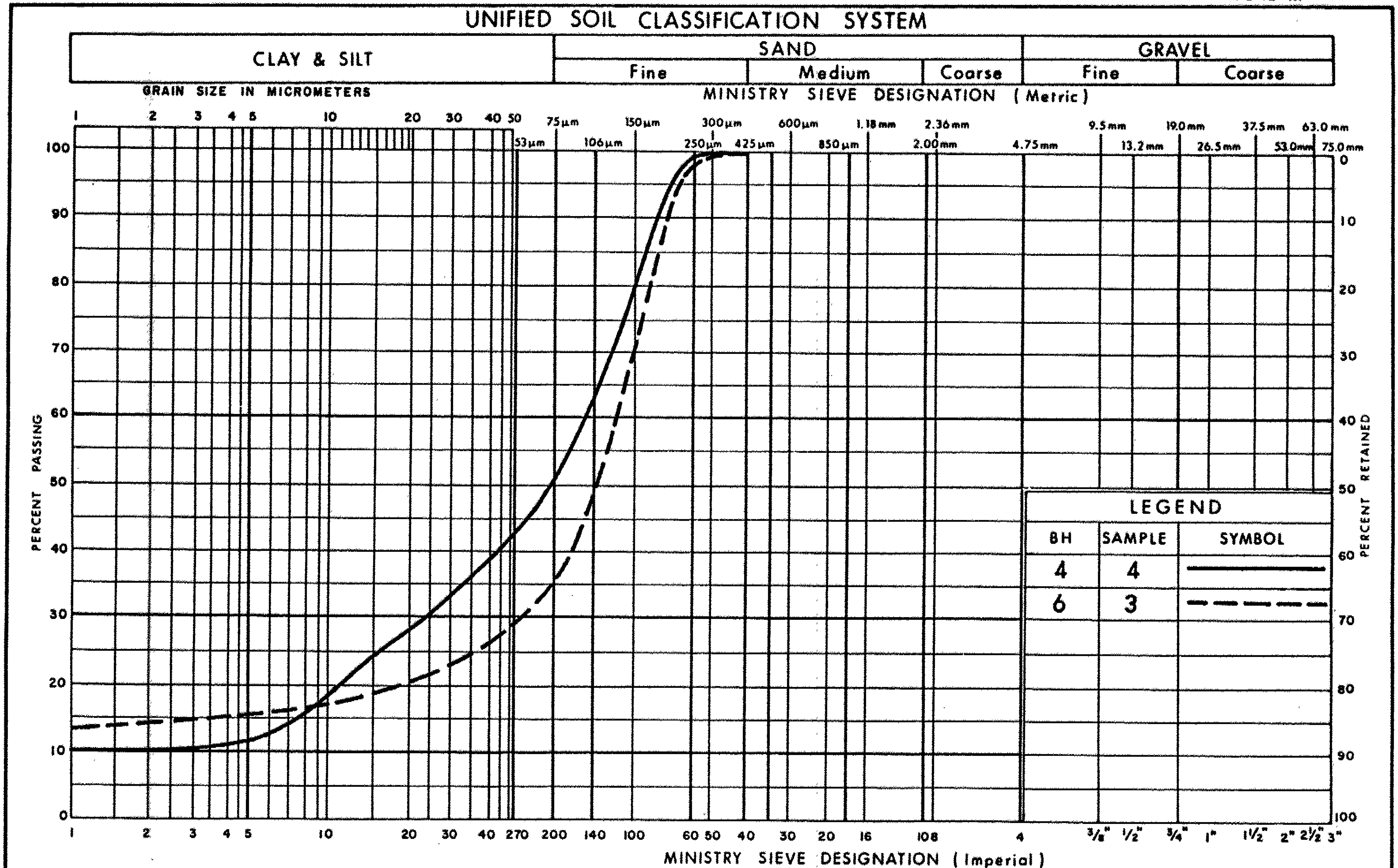
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	PLASTIC LIMIT W_p NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
119.2	GROUND LEVEL										
0.0							118, 116, 114, 112, 110, 108				
107.2 / 12.0	End of Dynamic Cone Penetration Test										

+3, x5 : Numbers refer to Sensitivity

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

OFFICE REPORT ON SOIL EXPLORATION

78 12 M



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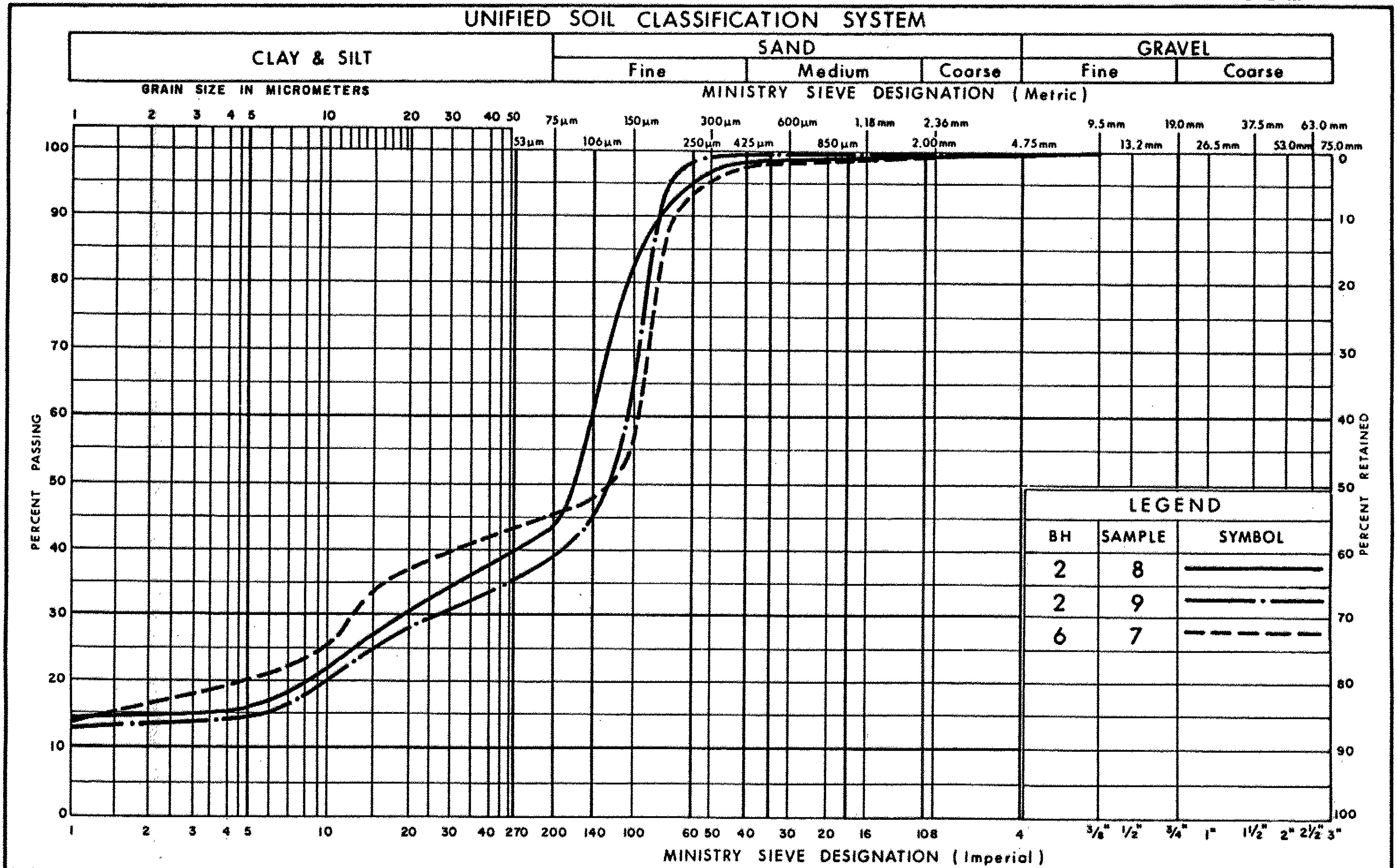
GRAIN SIZE DISTRIBUTION

SILTY SAND, TRACE OF CLAY

FIG No	1
W P	34-81-03

78 12 M



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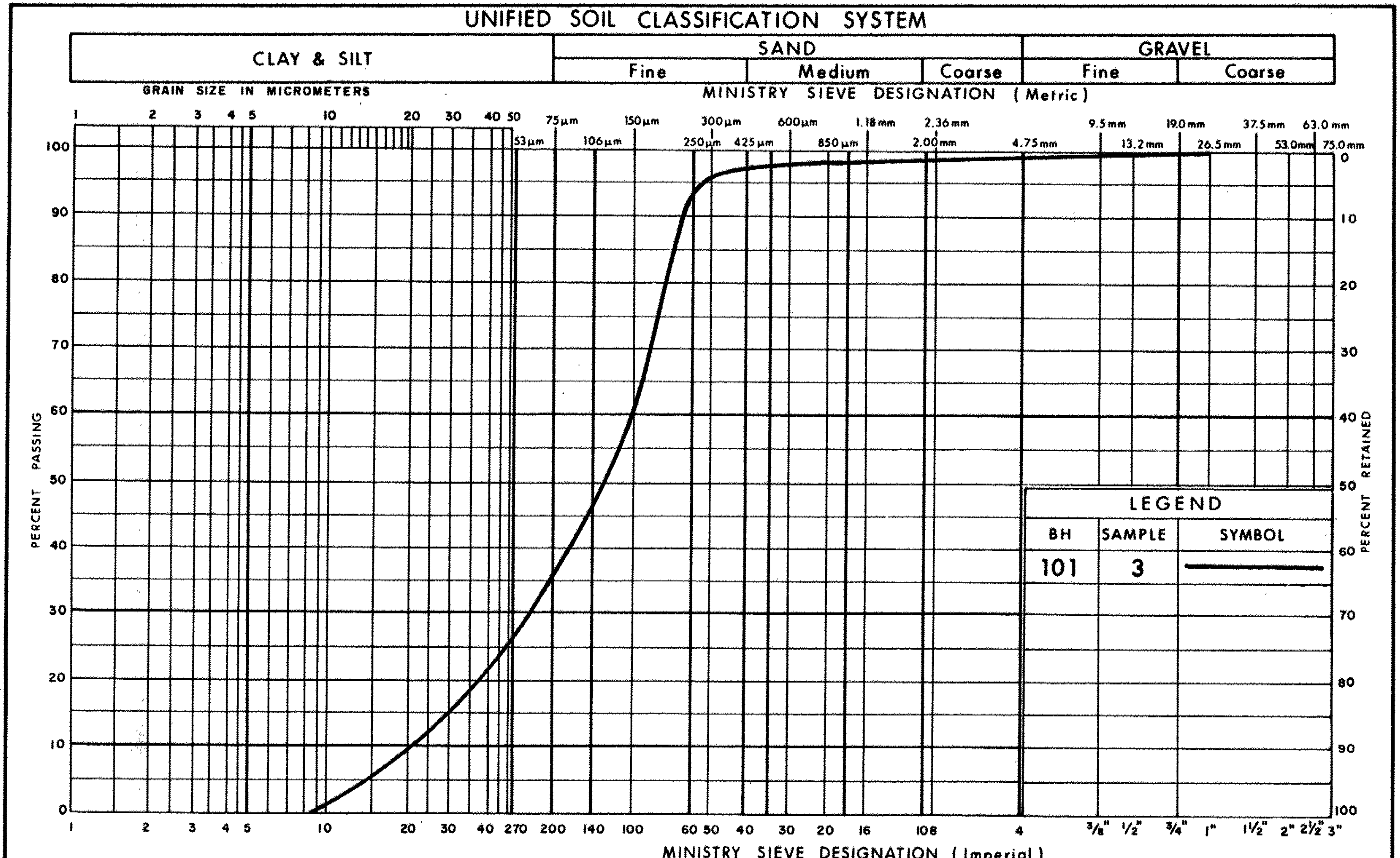
GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME CLAY

FIG No 2

WP 34-81-03

78 12 M



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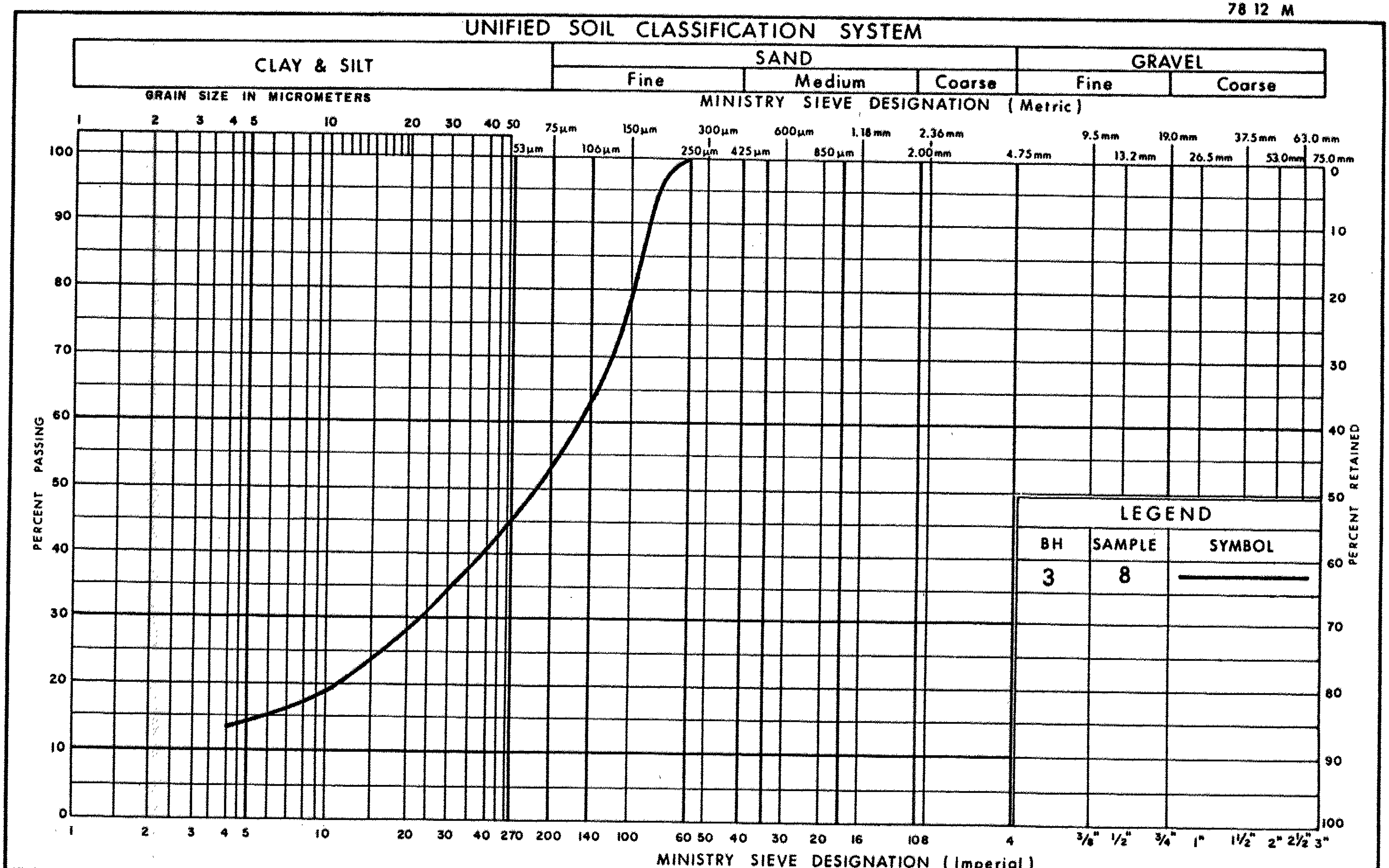
GRAIN SIZE DISTRIBUTION
SILTY SAND (INTERBED)

FIG No 3

WP 34-81-03



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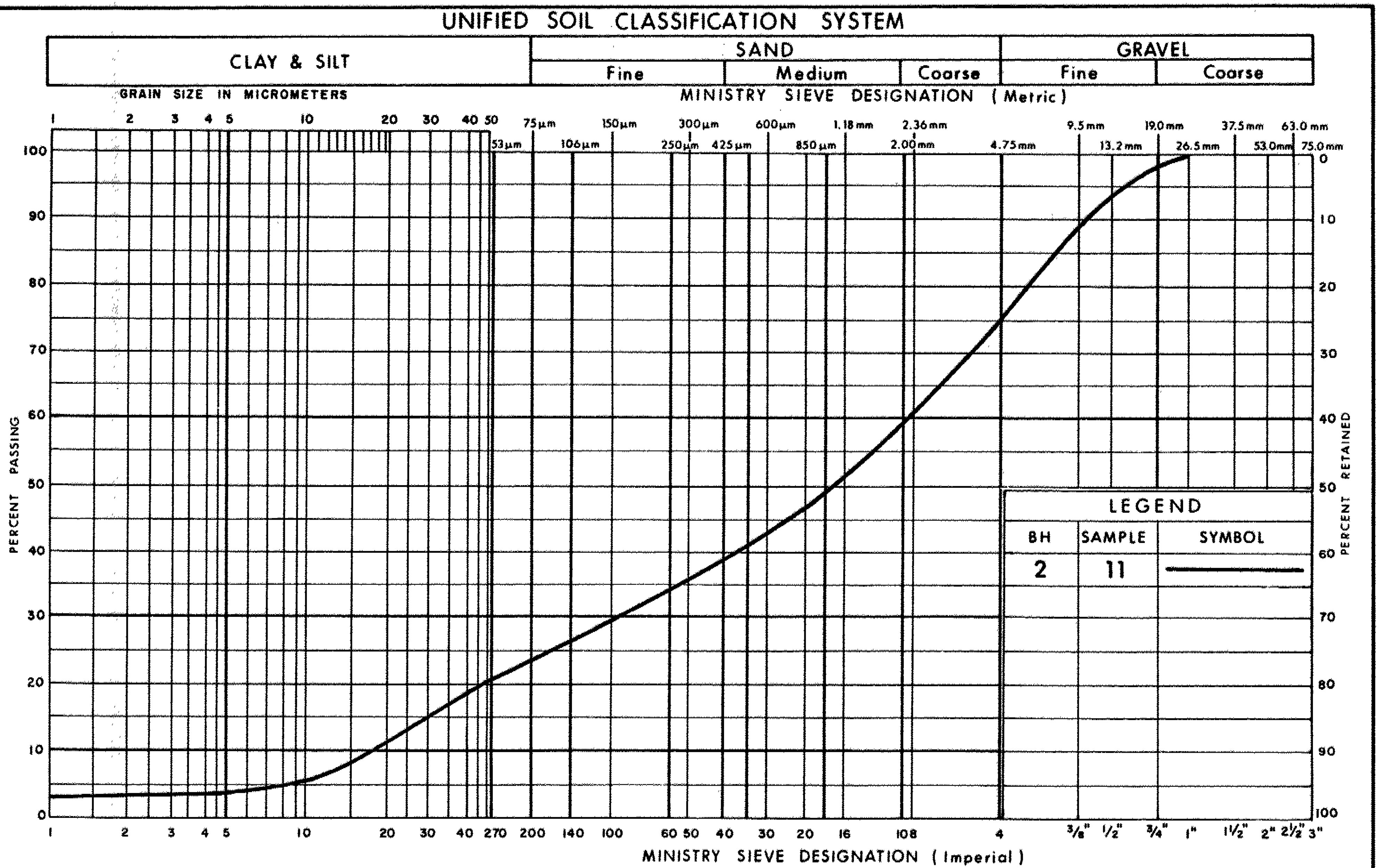
GRAIN SIZE DISTRIBUTION

SANDY SILT, TRACE OF CLAY

FIG No 4

W P 34-81-03

78 12 M



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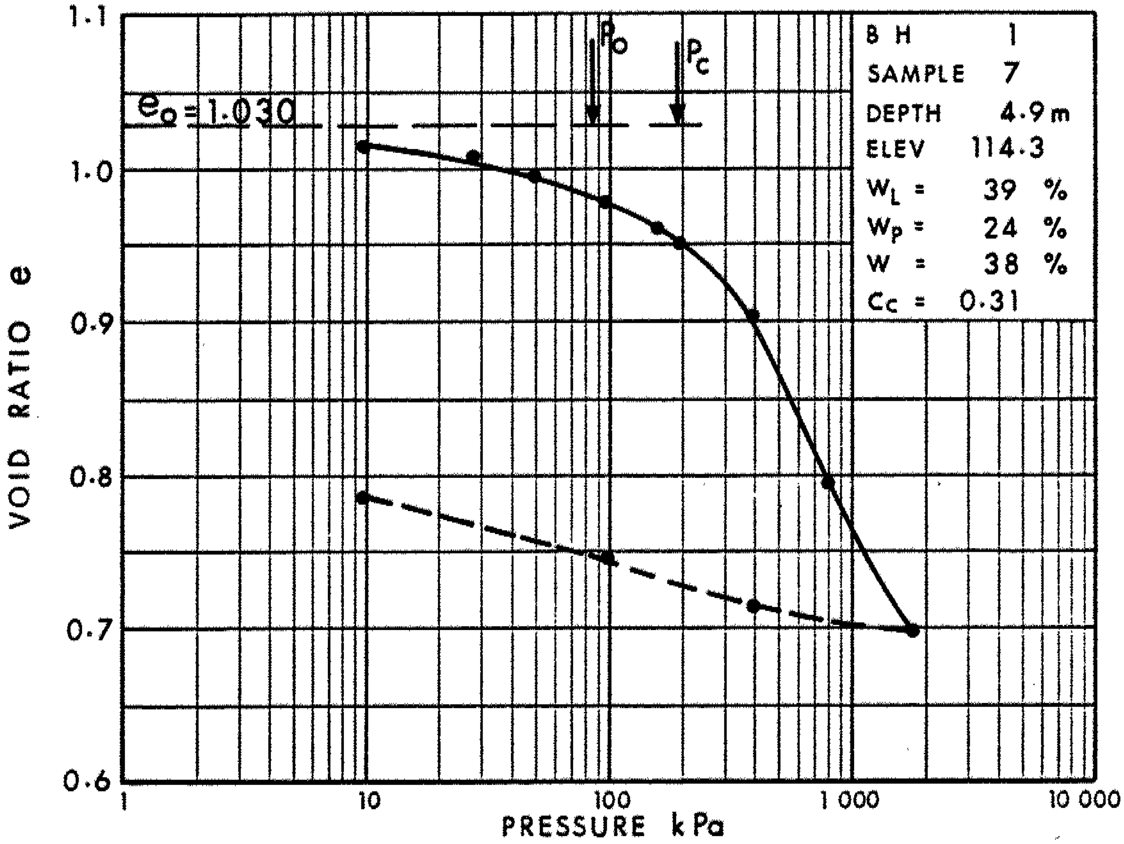
GRAIN SIZE DISTRIBUTION

SILTY SAND, SOME GRAVEL

FIG No 5

W P 34-81-03

VOID RATIO - PRESSURE CURVES



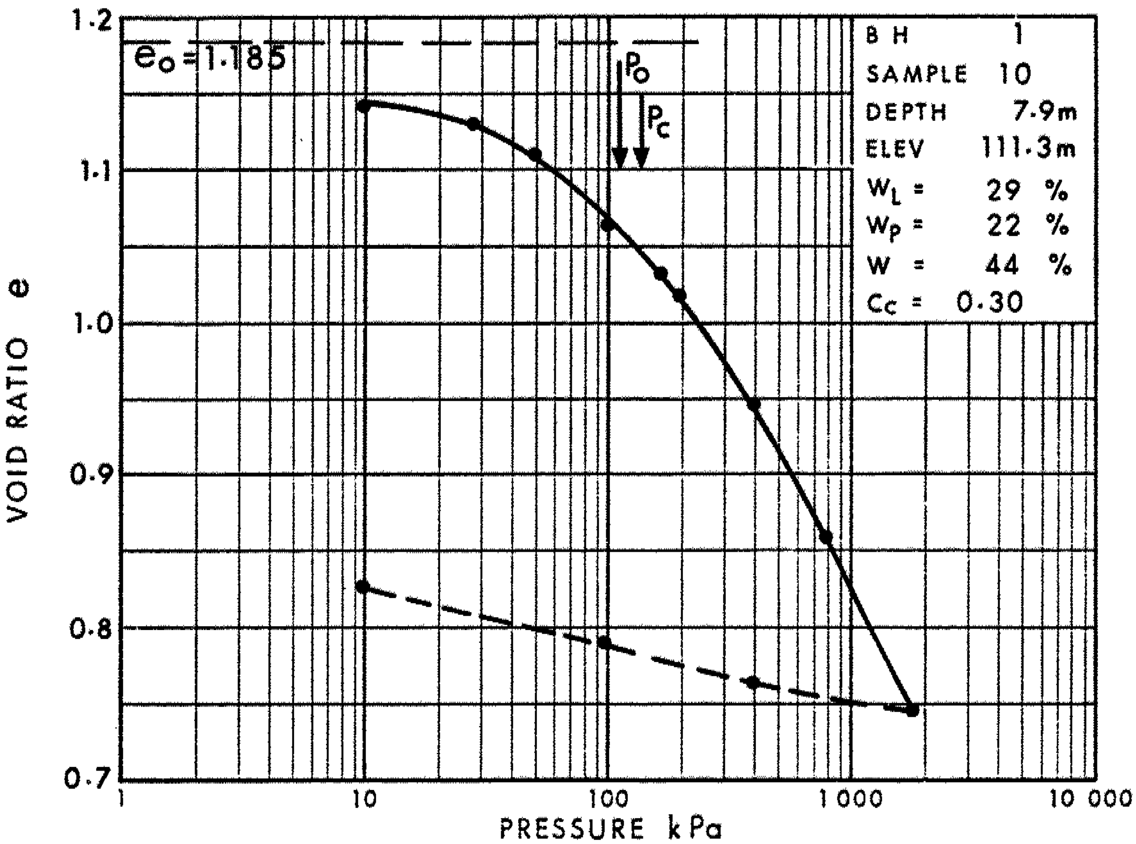
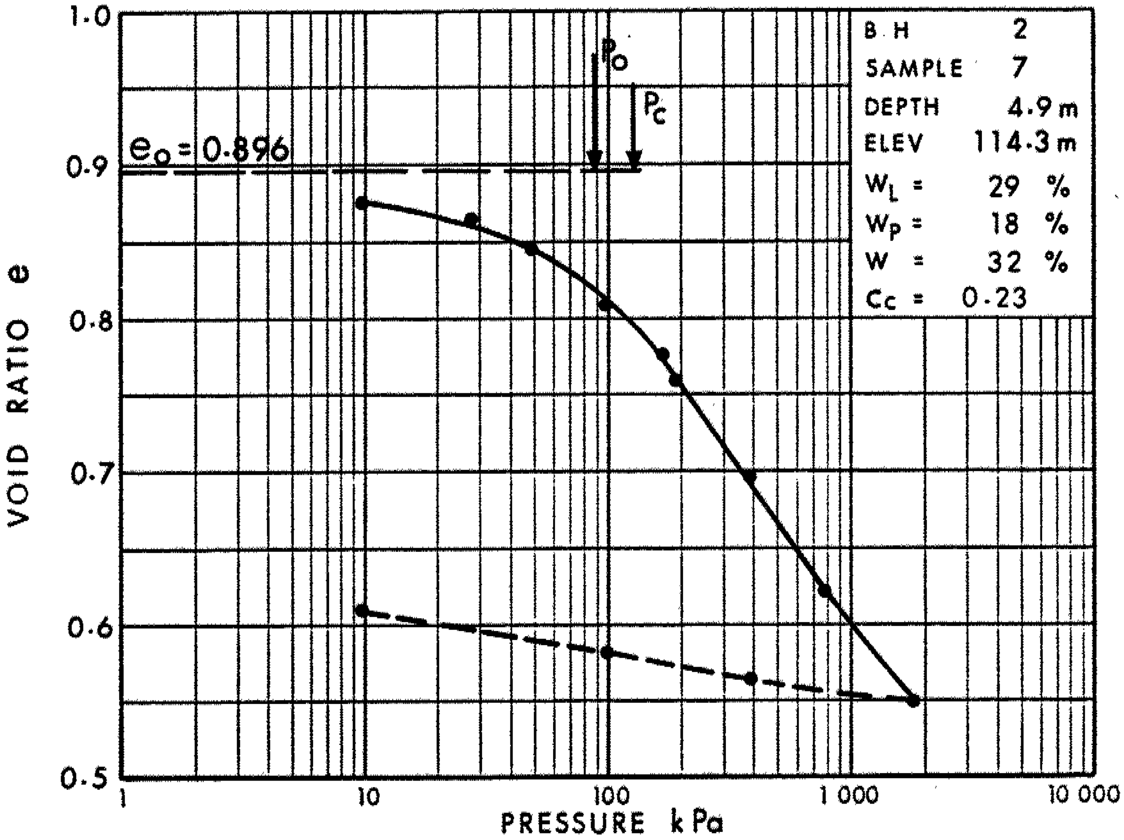


Fig 6

VOID RATIO - PRESSURE CURVES



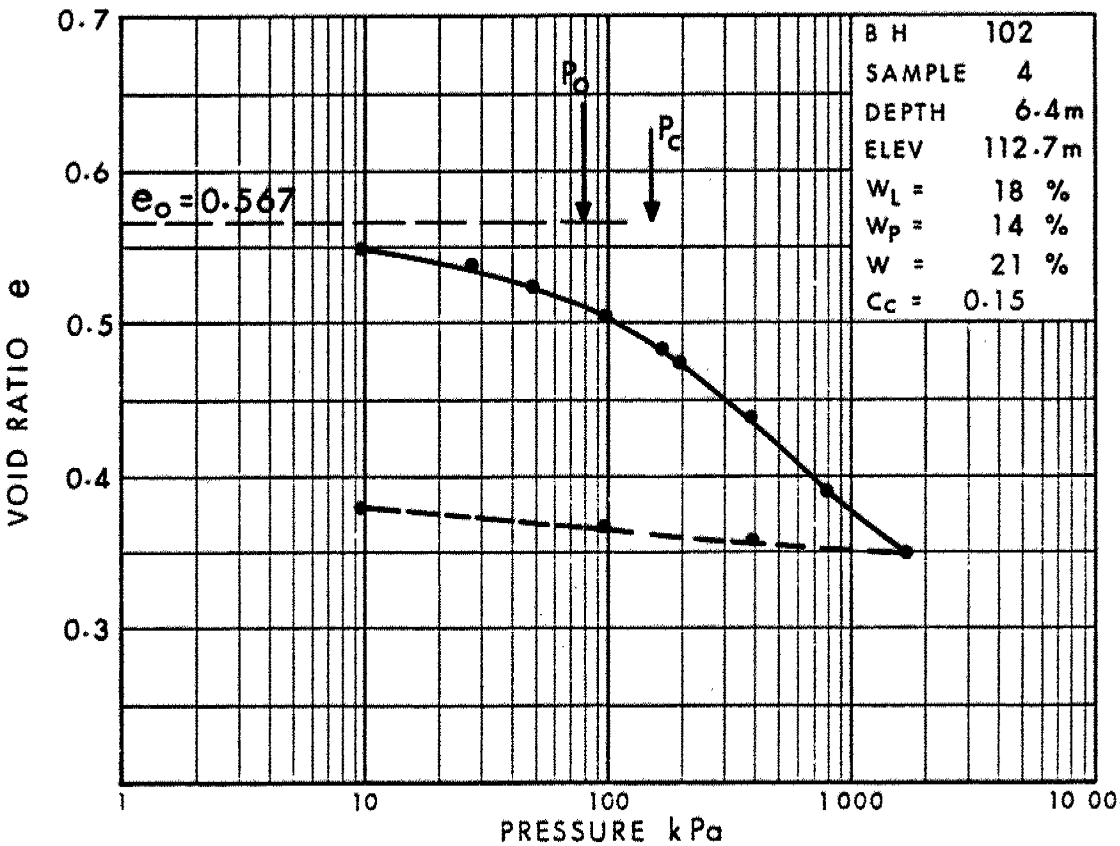
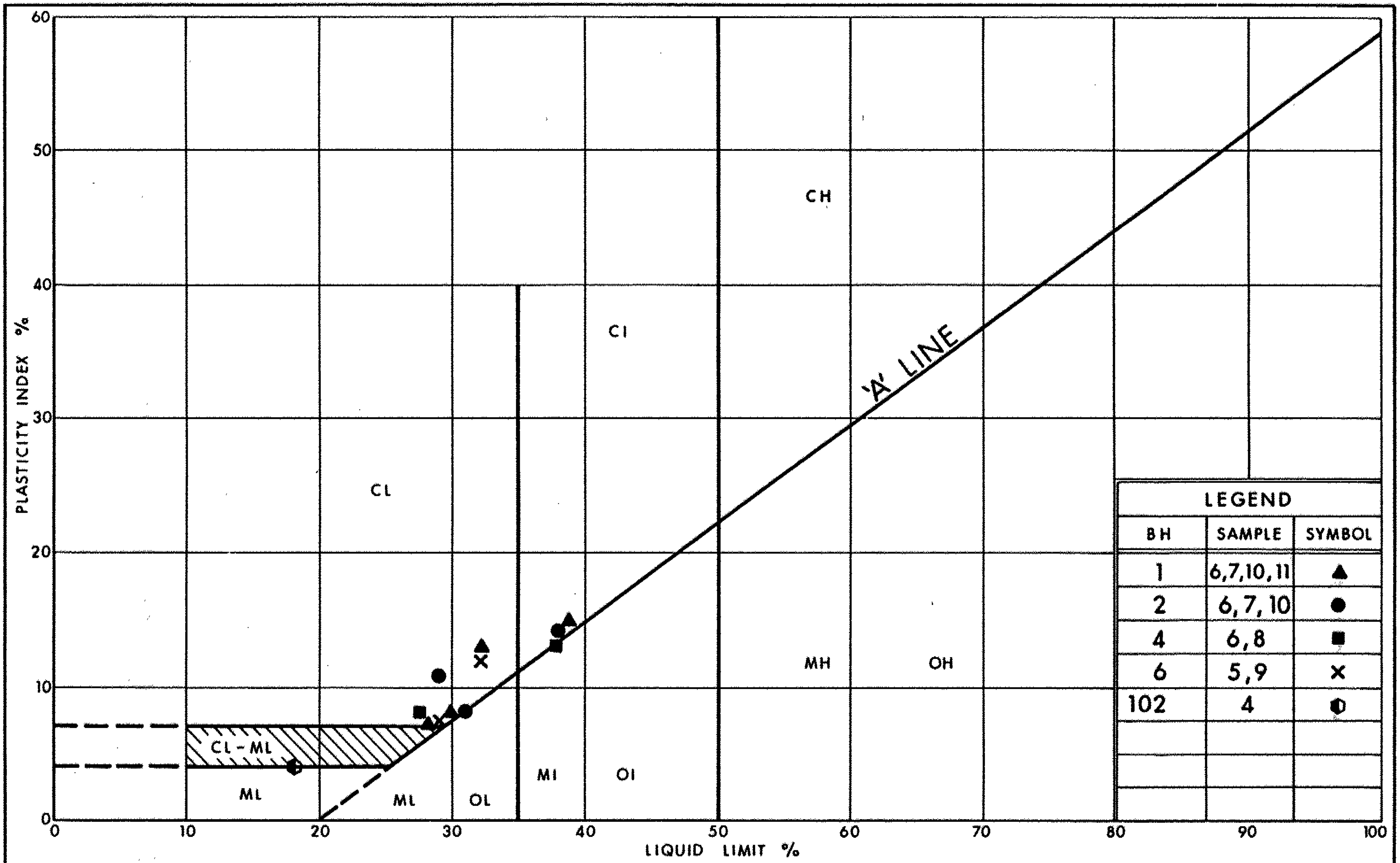


Fig 7

Oct 75, FF-S-21



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PLASTICITY CHART
SILTY CLAY

FIG No 8

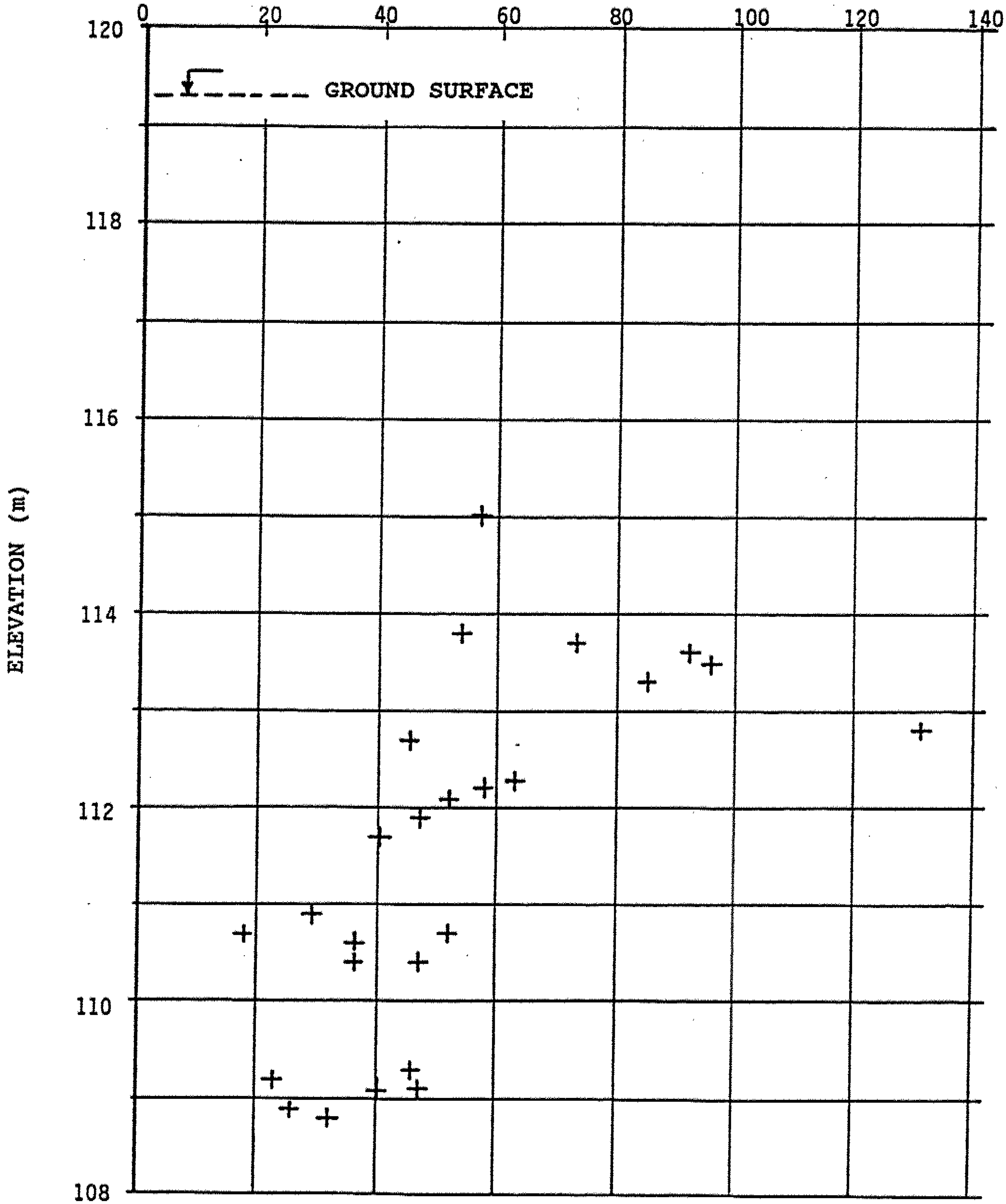
W P 34-81-03

OUR REF. NO. 89-11-14

FIGURE NO. 9
WP 34-81-03

UNDRAINED IN-SITU SHEAR STRENGTH AS MEASURED BY FIELD VANE TESTS (kPa)

BOREHOLES 1, 101 & 4



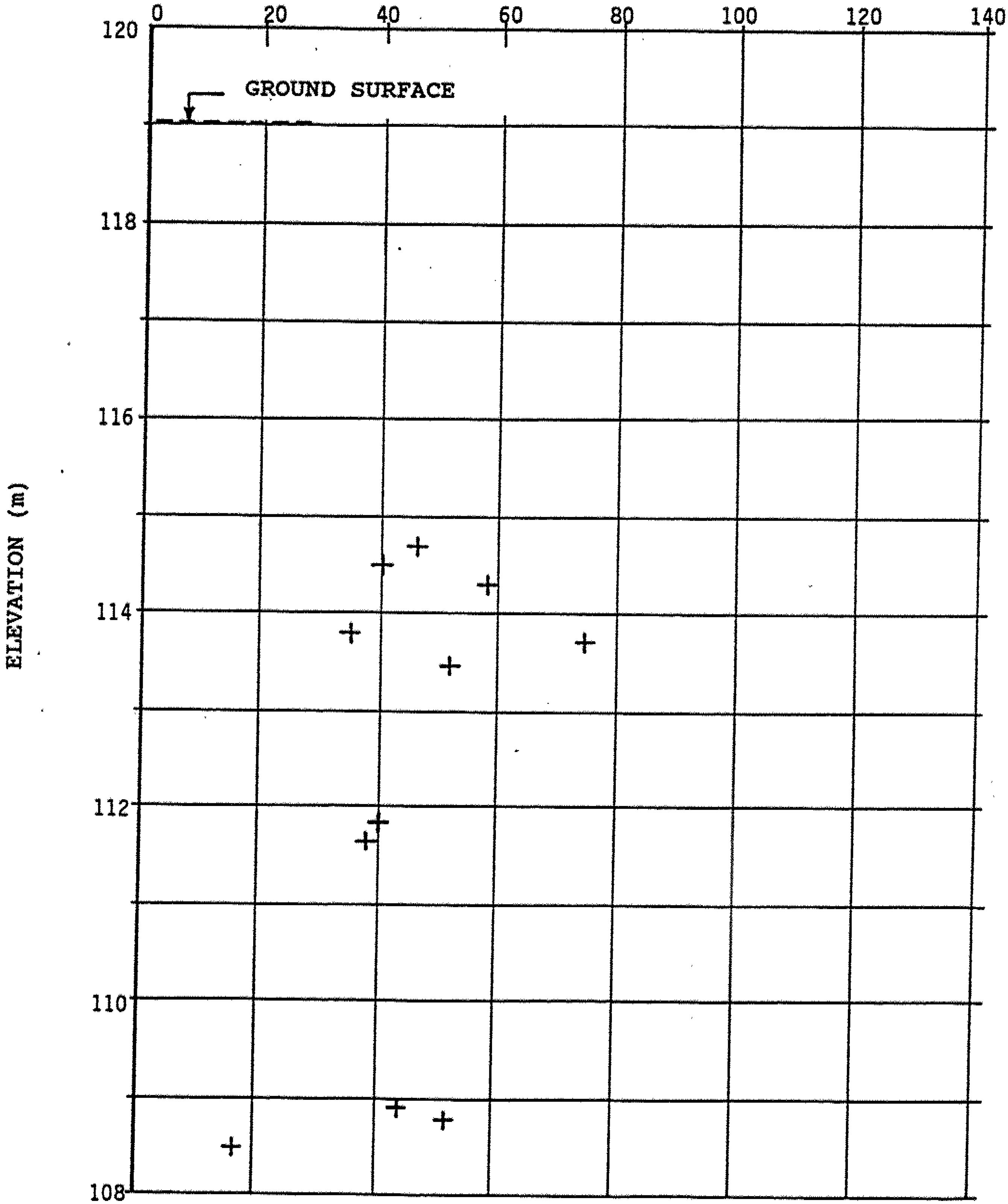
DOMINION SOIL INVESTIGATION INC.

OUR REF. NO. 89-11-14

FIGURE NO. 10
WP 34-81-03

UNDRAINED IN-SITU SHEAR STRENGTH AS MEASURED BY FIELD VANE TESTS (kPa)

BOREHOLES 2, 6 & 7



DOMINION SOIL INVESTIGATION INC.

METRIC

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

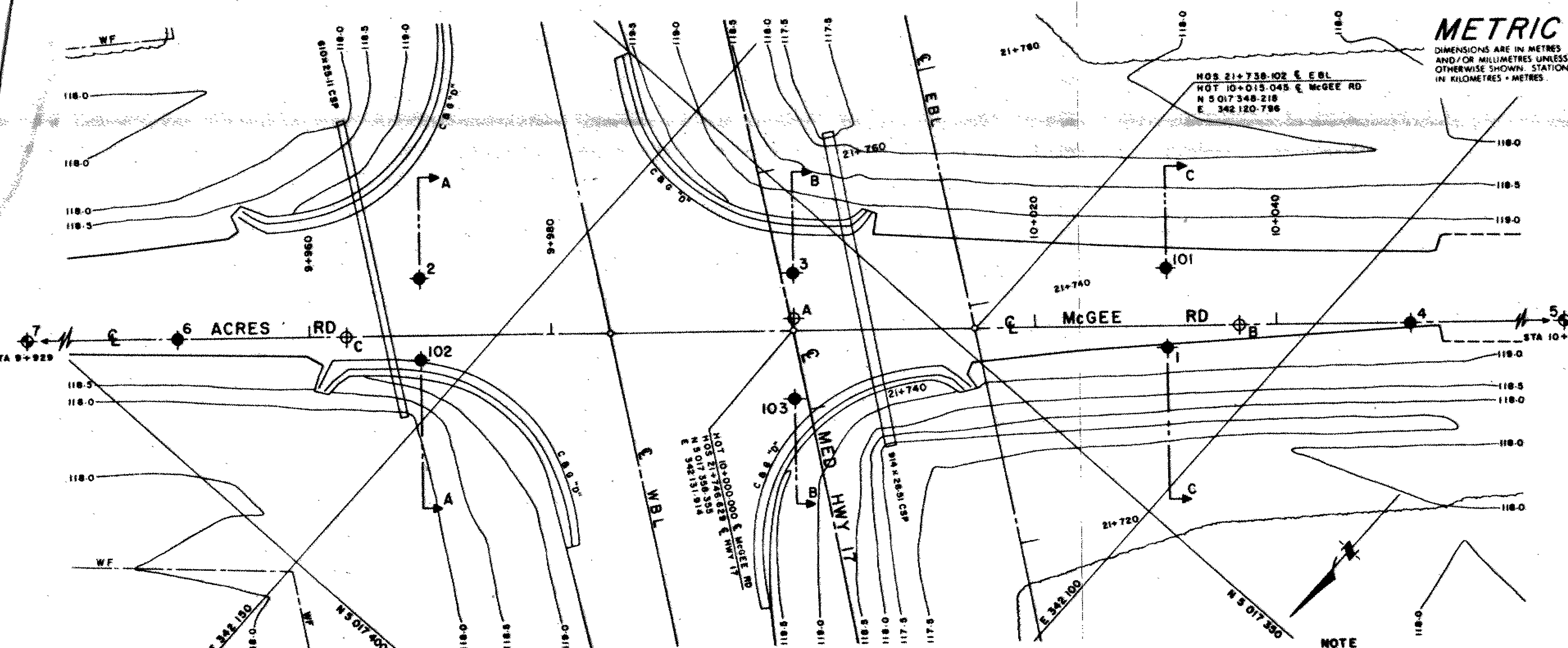
CONT No
WP No 34-81-03

ACRES RD & McGEE RD

BORE HOLE LOCATIONS & SOIL STRATA

SHEET

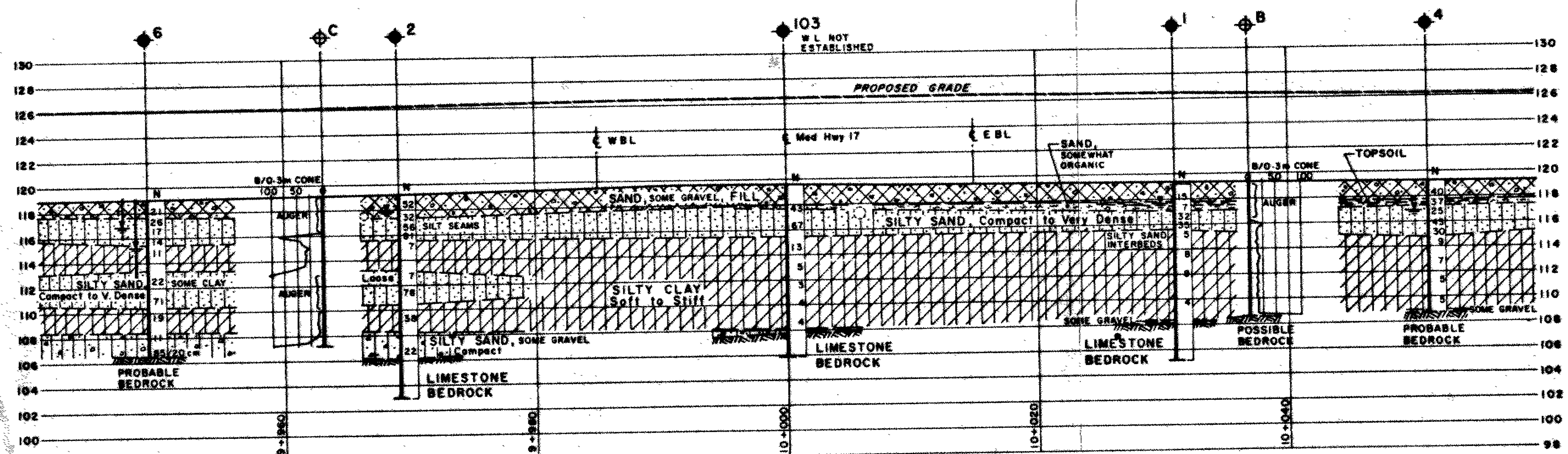
DOMINION SOIL INVESTIGATION INC.



PLAN

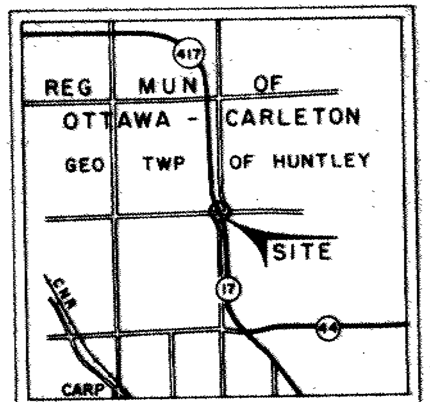
SCALE 5m 0 5m

NOTE
For Sections A-A, B-B, C-C and Boreholes 5 and 7 subsoil information refer to Dwg 348103-B



℄ PROFILE OF ACRES RD & McGEE RD

SCALE 5m 0 5m



KEY PLAN

SCALE 2km 1 0 1 2 3 4km

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 89 12
- Standpipe

No	ELEVATION	STATION	OFFSET
1	119.2	10+031	1.8m Rt
2	119.2	9+969	4.8m Lt
3	119.6	10+000	4.6m Lt
4	119.2	10+051	℄
5	119.3	10+071	℄
6	119.0	9+949	℄
7	118.8	9+929	℄
101	119.4	10+031	4.8m Lt
102	119.1	9+969	2.0m Rt
103	119.5	10+000	5.6m Rt
A	119.5	10+000	1.0m Lt
B	119.3	10+037	℄
C	119.2	9+963	℄

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

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

REV	DATE	BY	DESCRIPTION

Geocres No 31F-109

HWY No 17			DIST 9
SUBM'D ZSO	CHECKED	DATE Aug 9, 1990	SITE 3-569
DRAWN RM	CHECKED	APPROVED	DWG 348103-A

REF No E-65-17-2, 1990 02

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

METRIC

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

CONT No

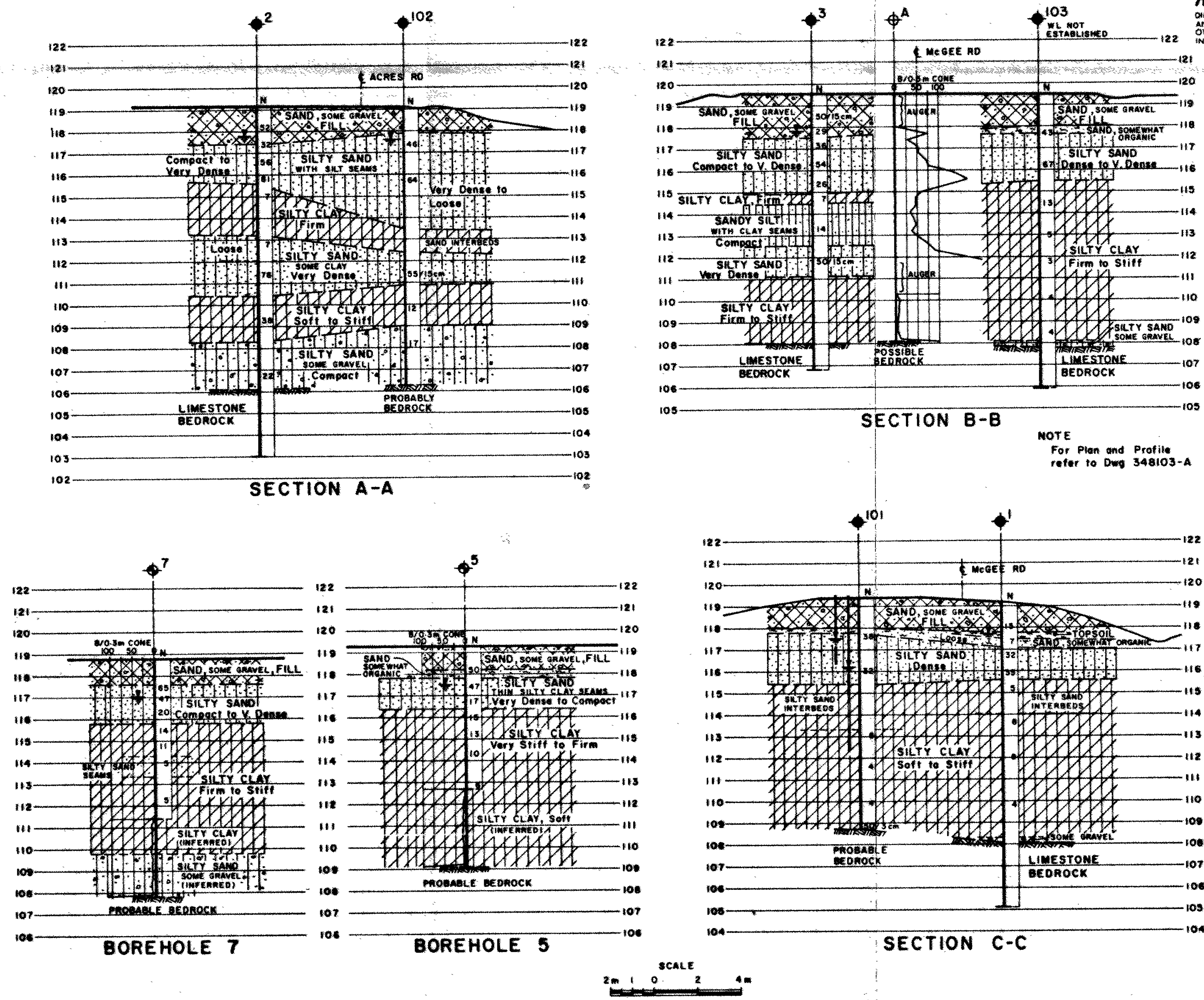
WP No 34-81-03

SHEET

ACRES RD & McGEE RD

BORE HOLE LOCATIONS & SOIL STRATA

DOMINION SOIL INVESTIGATION INC.



SEE DWG 348103-A

KEY PLAN

SCALE

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 89 12
- Standpipe

No	ELEVATION	STATION	OFFSET
1	119·2	10+031	1·8m Rt
2	119·2	9+969	4·8m Lt
3	119·6	10+000	4·8m Lt
5	119·3	10+071	℄
7	118·8	9+929	℄
101	119·4	10+031	4·8m Lt
102	119·1	9+969	2·0m Rt
103	119·5	10+000	5·6m Rt
A	119·5	10+000	1·0m Lt

≡NOTE≡

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

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

REV	DATE	BY	DESCRIPTION

Geocres No 31F-109

HWY No 17			DIST 9
SUBM'D ZSO	CHECKED	DATE Aug 10, 1990	SITE 3-569
DRAWN RM	CHECKED	APPROVED	DWG 348103-B

34-81-03

M E M O R A N D U M

Ontario

To: T.C. Tam, P. Eng.
Construction Services Engineer
Approvals Section
7th Floor, Atrium Tower

Date: July 19, 1993

From: Foundation Design Section
Room 315, Central Bldg.
Downsview

Tel: 235-3731
Fax: 235-5240

Re: Falsework Foundation Report
McGee Road U'Pass, Hwy. 17
Contract 93-31, Site No. 3-569
District 9, North Bay

We have reviewed the Falsework Foundation Report for the above site and find the contents and recommendations in the above report to be satisfactory.

We consider that the Contractor's initial proposal to build a 2 m thick granular pad for shoring and falsework foundation is probably technically superior than the one proposed in the above report.

We have no other comments.

B. Iyer, P. Eng.
Senior Foundation Engineer

BI/jb

c.c. - E.C. Lane

JOHN D. PATERSON & ASSOCIATES LTD.
Consulting Geotechnical & Environmental Engineers
28 Concourse Gate, Unit 1, Nepean, Ontario K2E 7T7 Tel: (613) 226-7381 Fax: (613) 226-6344

June 28, 1993

Deschenes Structures (Eastern) Inc.
P.O.Box 4338
Station E
Ottawa, Ontario
K1S 5B3

Attention: **Mr. Elwin Pierce**

Subject: **Founding Conditions for Formwork Shoring/Falsework Support**
McGee Road Underpass, Highway 17 (417)
Township of West Carleton (Huntley), Ontario
MTO Contract No. 93-31

Dear Sir:

At the request of Deschenes Structures (Eastern) Inc., this firm has been commissioned to evaluate the founding conditions for formwork shoring/falsework support for the bridge deck for the above-noted project. This submission will form part of the falsework foundation certificate, required as part of the contract. During the preparation of the bedding layer for the support of the formwork shoring/falsework, prior to the erecting the formwork, inspections should be conducted by this firm to confirm the recommendations provided in this report have been followed and, as such, that our recommendations are applicable. A report will be issued at that time to confirm that the formwork shoring/falsework support structures have been bedded on suitably prepared bearing media.

The client has provided this firm with the Foundation Investigation Report for Ministry of Transport Ontario (MTO) Contract No. 93-31. More specifically this includes the Foundation Investigation Report for WP 31-81-03, regarding the proposed McGee Road Underpass, Site No. 3-569, Highway 17 (417). This report was prepared by Dominion Soil Investigation Inc., under the technical supervision of the MTO Foundation Design Section.

The details of the proposed bridge deck shoring are shown on Manhire Associates Limited Drawing S1, Project No. 93-128. The structural aspects of the shoring, as they relate to the geotechnical aspects of the shoring, were reviewed by the undersigned with Mr. R.I. Manhire, P.Eng. prior to the preparation of this report.

Reference should be made to the Foundation Investigation Report for the details of the soil conditions underlying the site. In summary, the soils consist of up to 2 m of granular fill materials over compact to very dense silty sand, which is underlain at 3.6 to 4.7 m depth by a deposit of silty clay (occasionally sandy silt), with interbedded silty sand. The silty clay extends to 11 m ± depth, where it is underlain by glacial till and, in turn, bedrock.

The only stratum of concern to the founding of the formwork shoring/falsework is the silty clay. For the greater part this deposit is firm to soft in consistency, although portions were determined to be stiff. More important than its consistency, however, are its consolidation (or settlement-related) characteristics.

The silty clay is lightly overconsolidated to overconsolidated, based on the results of four consolidation tests conducted as part of the foundation investigation. Sample 10, from 7.9 m depth in Borehole 1, was determined to have an overconsolidation of only 26 kPa between its preconsolidation pressure (i.e. its maximum past effective stress level) and its existing effective overburden pressure. The client's initial proposal to build up a 2 m high fill under the shoring/falsework sills was abandoned at this firm's recommendation because the weight of the fill would exceed the preconsolidation pressure of the clay, with associated settlements expected to be intolerable to the formwork.

It is recommended that the sills for the formwork shoring/falsework be founded on a minimum of 200 mm of graded crushed stone over the existing granular fill materials. The existing fill consists of the existing road base and subbase for Highway 17, McGee Road and Acres Road. Where the formwork extends beyond the limits of the existing fill (i.e. the west shoulders of McGee and Acres Roads), the ground surface should be stripped of vegetation and topsoil to expose the silty sand, prior to backfilling to the sill level with select granular fill material.

The 200 mm graded crushed stone bedding layer can consist of OPSS Granular A or Granular B Type II crushed stone material, compacted to a minimum of 95% of its Standard Proctor maximum dry density value. Compacted crusher screenings can be used over the Granular A or Granular B Type II material in order to provide a workable bedding material for the sills. Where additional material is required to build up from the subgrade level to the base of the crushed stone bedding layer, this material should consist of material of at least Granular B Type I quality, compacted to a minimum of 95% of its Standard Proctor maximum dry density value.

The crushed stone bedding layer should extend a minimum of 0.6 m beyond the ends and/or edges of the formwork sills and/or 1.0 m minimum beyond the outside frame legs. Where thicker granular fill deposits are required, the side slopes of the fill should be sloped no steeper than 2H:1V to provide adequate stability. The top surface of the fill should extend a minimum of 0.6 m beyond the ends and/or edges of the formwork sills and/or 1.0 m minimum beyond the outside frame legs.

The excavations around the abutments and/or pier for the structure should, where necessary, be backfilled with materials compacted to at least 95% of their Standard Proctor maximum dry density values. The use of material of Granular B Type I quality, or better, is preferred, however, the use of site excavated fill will suffice if it is placed and compacted in relatively thin lifts to the recommended density.

Sills founded on 200 mm of OPSS Granular A or Granular B Type II crushed stone, placed and compacted over one or a combination of the above-noted subgrade media can be designed to an allowable bearing pressure of 150 kPa.

The routine heavy-duty shoring/falsework frames will have a maximum leg load of 71 kN. The routine sills will consist of double (i.e. side by side) 150 mm thick by 250 mm wide wood members, on 1.8 m centres, designed by the structural engineer to act together as a 500 mm wide sill. The frame leg spacing will be approximately 1.5 m along the axis of the sills.

The extra heavy-duty frames supporting the formwork structure over the opening for Highway 17 will be founded on a built up sill structure that will distribute the load over an approximately 2.4 m width of the bearing medium. The loading on these sill structures (i.e. one on each side of the opening) will be approximately 235 kN/m.

The bearing capacity of the supporting soils is, in our opinion, adequate to support the above-noted sill pressures (with an appropriate factor of safety) with respect to shear failure. Settlement of the sills will occur, however, during and after the application of the construction loads to the shoring (i.e. during the placing of the concrete deck).

Detailed settlement analyses have been conducted by this firm using the available consolidation test information for the cohesive soils from Borehole 1. The results of these analyses are provided below. The analyses were conducted using two different loading conditions. The primary values provided are for the total loading conditions due to the dead load of the formwork shoring/falsework system as well as the weight of the concrete deck. The secondary settlement values, provided in parentheses, are for the loading condition of dead load of the formwork shoring/falsework system only without the concrete weight. The difference between the settlements for these two loading conditions provides an indication of the portion of settlement attributable to the concrete placement loads.

The estimated maximum long-term consolidation settlement of the bearing medium would be of the order of 37 mm (16 mm) total under the centre of the sills or built up footing under the extra heavy-duty frames associated with the traffic opening on both sides of Highway 17. The ends of these loaded areas have an estimated settlement of 25 mm (11 mm).

The estimated settlement of the routine sills varies between 22 mm and 32 mm (10 mm and 14 mm), at the midpoint of the sills, and between 15 mm and 21 mm (7 mm and 9 mm), at the midpoint of the sills, depending on their location within the overall loaded area (i.e. interior sills settle more than exterior or outside sills).

The overall long-term settlement, therefore, is within the range of 15 mm to 37 mm (7 mm to 16 mm), with a differential settlement of 22 mm (9 mm). The actual settlement within the time-frame critical to the support of the bridge deck would be expected to be significantly less, and would reflect a reduction due to the portion of the formwork-related settlements (i.e. values in parentheses) that had occurred up to the final adjustment of the frame leg screw jacks.

No time-dependent consolidation information was provided in the reporting and it is not possible to assess the influence of sand layers in the cohesive soil strata. However, for the case analyzed (for a 7 m thick clay layer), no more than 60% of consolidation would be expected to occur after approximately 30 days of load application. The presence of the relatively thick sand layers in the silty clay tends to speed up the rate of consolidation, although the amount of compressible clay is reduced.

As such, and in our opinion, the actual settlement within the time-frame critical to the support of the bridge deck (i.e. between final adjustment of the frame leg screw jacks and 30 days after concrete placement) would be expected to be less than 15 mm total and 10 mm differential.

Please contact the undersigned if you have any questions or need further information with regard to this submission.

REGISTERED PROFESSIONAL ENGINEER
A. J. TOVELL
PROVINCE OF ONTARIO

Yours truly,

JOHN D. PATERSON & ASSOCIATES LTD.

Andrew J. Tovell, P. Eng.

AJT/
Triplicate
cc: Manhire Associates Limited

Report No. S6013-93

Ministry of Transportation
Ontario

FAXGRAM

PLEASE TYPE

DATE JAN 16, 91

PAGE 1 OF 1

TO: SAM CHENG GEOTECH. SECTION ATTN. IRIS STEBLINSKY	FROM: BALU IYER FOUND. DES. SECTION

SUBJECT: EMBANKMENT DESIGN
STATION 10+030 TO 10+200
W.P. 34-81-03

CC D. MOON, P&D.

WE UNDERSTAND THAT BETWEEN STNS. 10+030 AND 10+200, ONE SIDE OF THE EMBANKMENT MAY HAVE TO BE STEEPENED TO ACCOMMODATE A DRAINAGE DITCH. THE FOUNDATION INVESTIGATION CARRIED OUT AT THE ABOVE SITE EXTENDED ONLY TO STN. 10+070. WE WOULD THEREFORE LIKE TO RECEIVE INFORMATION COLLECTED BY YOUR SECTION FROM STN. 10+070 TO STN. 10+200, REGARDING SUBSURFACE SOIL AND GROUNDWATER CONDITIONS AND IN SITU SHEARS STRENGTH OF THE CLAY SOILS. FOLLOWING A REVIEW OF ALL AVAILABLE DATA, WE WOULD PROVIDE OUR RECOMMENDATIONS REGARDING DESIGN AND CONSTRUCTION OF THE EMBANKMENT IN THE SUBJECT AREA.

PLEASE FORWARD YOUR DATA ASAP.

ADM-S-481 87-05

METRIC

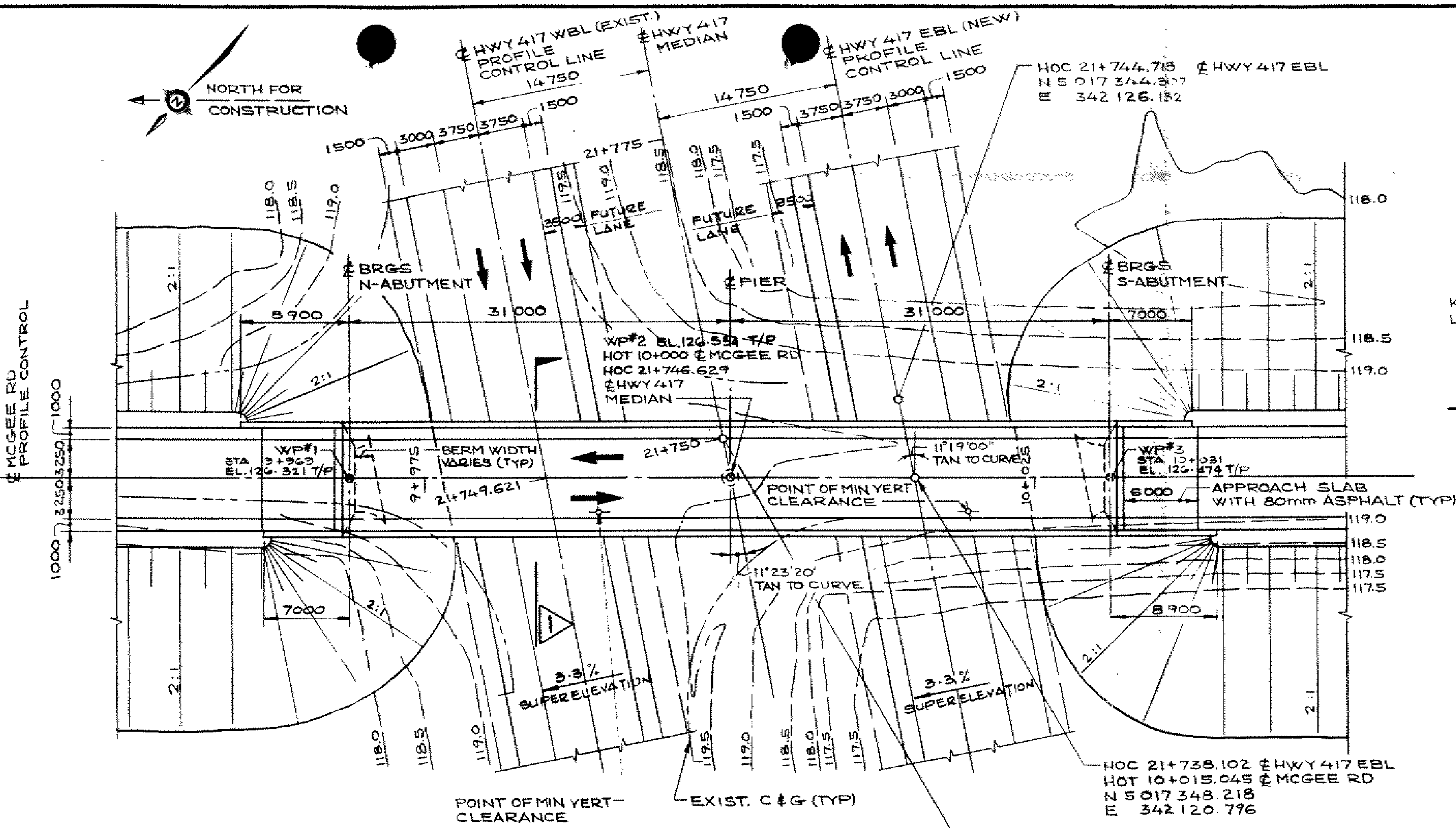
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DIST No 9
CONT No
WP No 34-81-03

MCGEE ROAD UNDERPASS GENERAL ARRANGEMENT

SHEET

Sandwell Inc.
Sandwell Swan Wooster Division

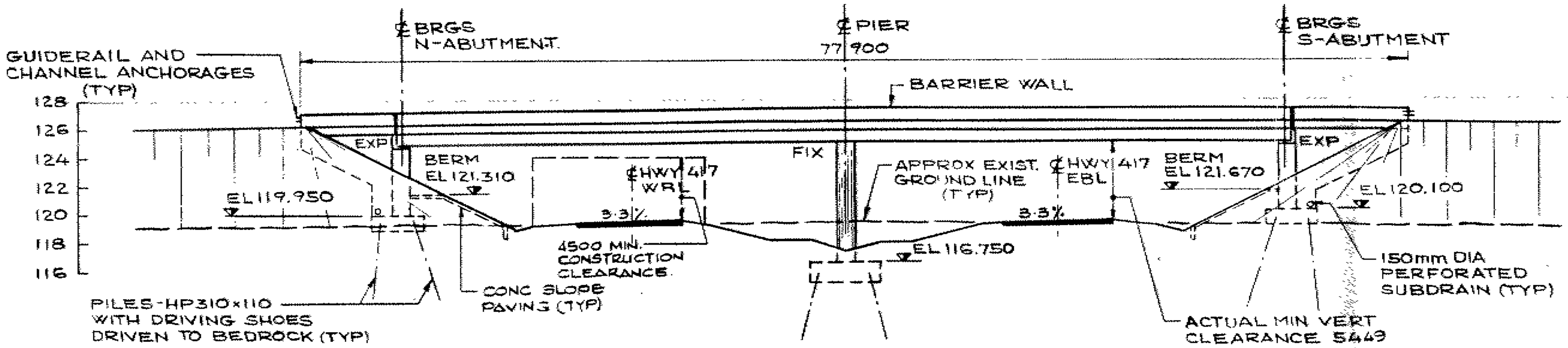


PLAN

SCALE 1:250

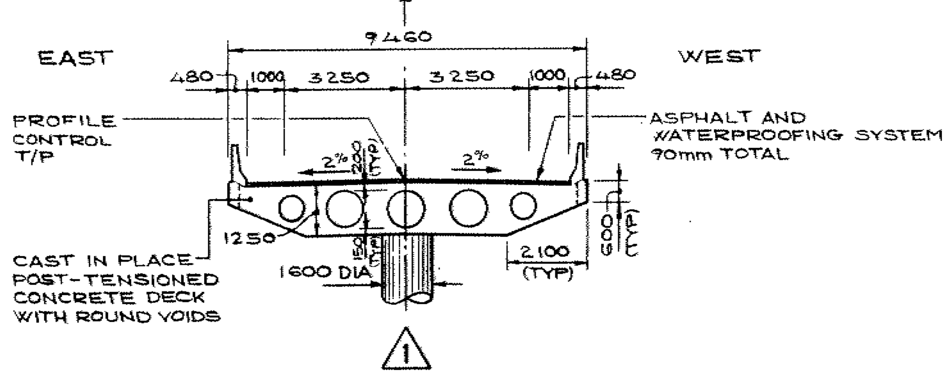
NOTES:

1. T/P DENOTES TOP OF PAVEMENT.
2. WP DENOTES WORKING POINT.
3. INFORMATION REGARDING WBL WILL BE REFERENCED TO THE MEDIAN CENTRE LINE.



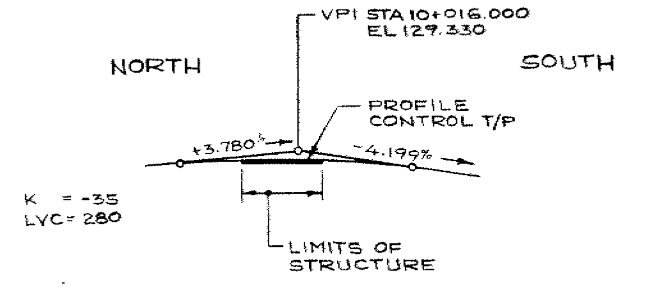
ELEVATION

SCALE 1:250



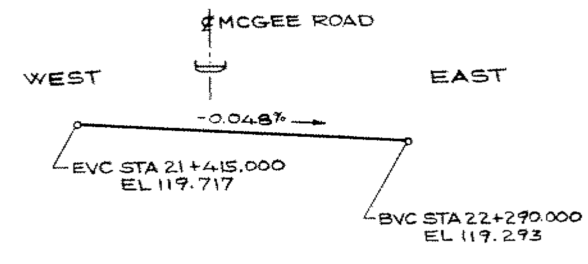
SCALE 1:100

BM EL 118.337
N & W IN ROOT OF 0.1 CEDAR 33.8 LT OF STA



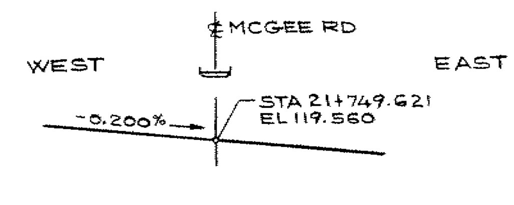
PROFILE ACRES RD AND MCGEE RD

NTS



PROFILE HWY 417 AT ℄ OF EBL

NTS



PROFILE HWY 417 AT ℄ OF WBL

NTS

GENERAL NOTES:

1. **CLASS OF CONCRETE**

DECK AND PIER COLUMNS	35 MPa
REMAINDER	30 MPa

2. **CLEAR COVER TO REINFORCING STEEL**

FOOTINGS	100 ± 25
ABUTMENTS AND WINGWALLS:	
FRONT FACE	80 ± 20
BACK FACE	70 ± 20
PIER COLUMNS	80 ± 20
DECK - TOP	70 ± 20
BOT	50 ± 10
REMAINDER	70 ± 20

UNLESS OTHERWISE NOTED

3. **REINFORCING STEEL**
REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX "C" DENOTE COATED BARS.

BEARING SEAT ELEVATIONS
IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCING STEEL TO SUIT THE ACTUAL HEIGHTS.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BORE HOLE LOCATIONS & SOIL STRATA I
3. BORE HOLE LOCATIONS & SOIL STRATA II
4. FOOTING LAYOUT
5. FOOTING REINFORCING
6. NORTH & SOUTH ABUTMENTS
7. PIER DETAILS
8. DECK DETAILS & SCREED ELEVATIONS
9. LONGITUDINAL TENDONS
10. TRANSVERSE TENDONS
11. DECK REINFORCING I
12. DECK REINFORCING II
13. BARRIER WALLS
14. 6000 mm APPROACH SLABS
15. DETAILS OF CONCRETE SLOPE PAVING
16. JOINT ANCHORAGE & ARMOURING
17. STANDARD DETAILS
18. AS CONSTRUCTED ELEV. & DIM.
19. QUANTITIES - STRUCTURE I
20. QUANTITIES - STRUCTURE II

APPLICABLE STANDARD DRAWINGS

DD-3503 MIN. GRANULAR BACKFILL REQUIREMENTS
DD-4602 FALSEWORK CLEARANCES

S. C. FOK

G. C. TEODOSIU

PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN BH	CHK GT	CODE OHBDC - 83	LOAD CLASS A	DATE SEP 1990	
DRAWN	CHK BH	SITE 3 - 569	STRUCT	SCHEME	DWG. 1

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO

METRIC

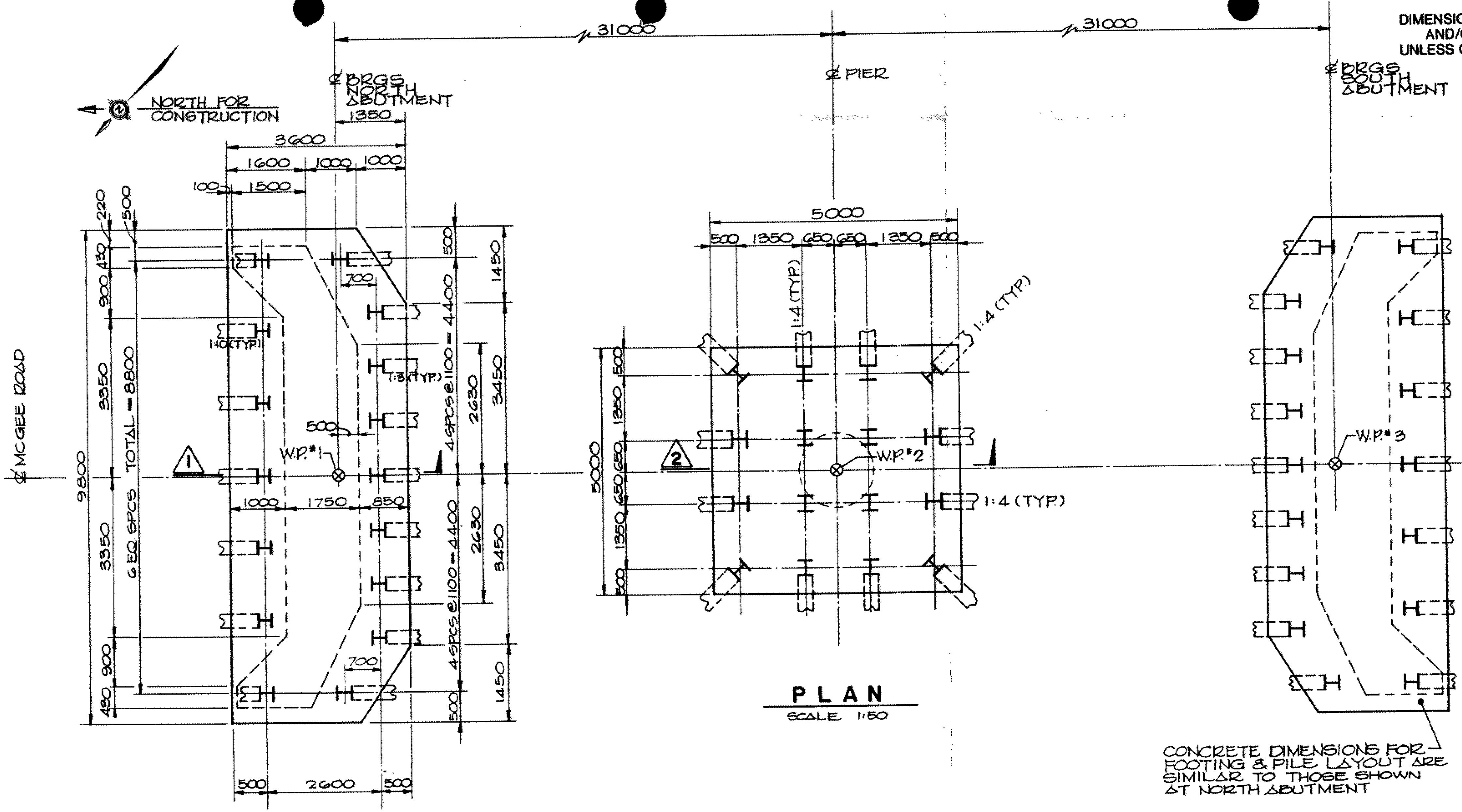
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No
WP No 34-81-03

MCGEE ROAD UNDERPASS
FOOTING LAYOUT

SHEET

Sandwell — Sandwell Inc. Sandwell Swan Wooster Division



PLAN
SCALE 1:50

WORKING POINTS DATA

W.P.	COORDINATES NORTH	COORDINATES EAST
1	5 017 379.241	342 154.822
2	5 017 358.355	342 131.913
3	5 017 337.468	342 109.006

PILE DESIGN DATA

PILE DATA (HP 310 X 110)

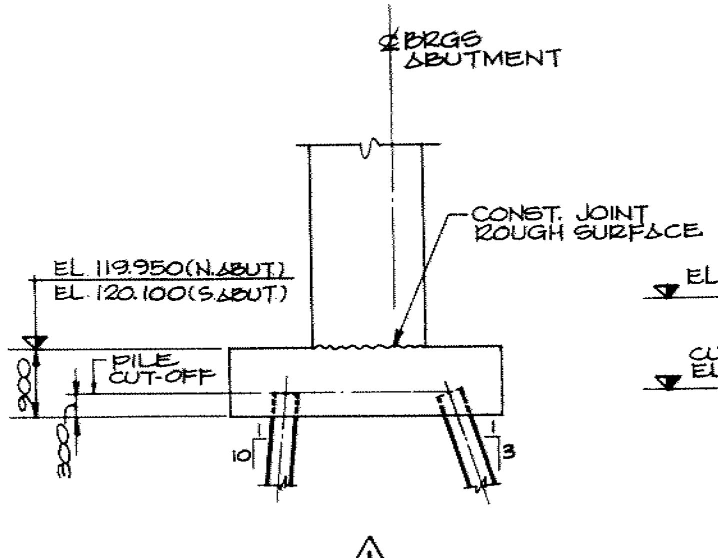
LOCATION	BATTER	NO. REQ'D	LENGTH	CUT OFF EL.
NORTH ABUTMENT	1:10	7	13.35	119.35
	1:3	9	14.00	119.35
PIER	1:4	12	8.00	115.55
	STR.	4	7.75	115.55
SOUTH ABUTMENT	1:10	7	11.45	119.50
	1:3	9	12.05	119.50

MAX. COMB. FACTORED LOAD

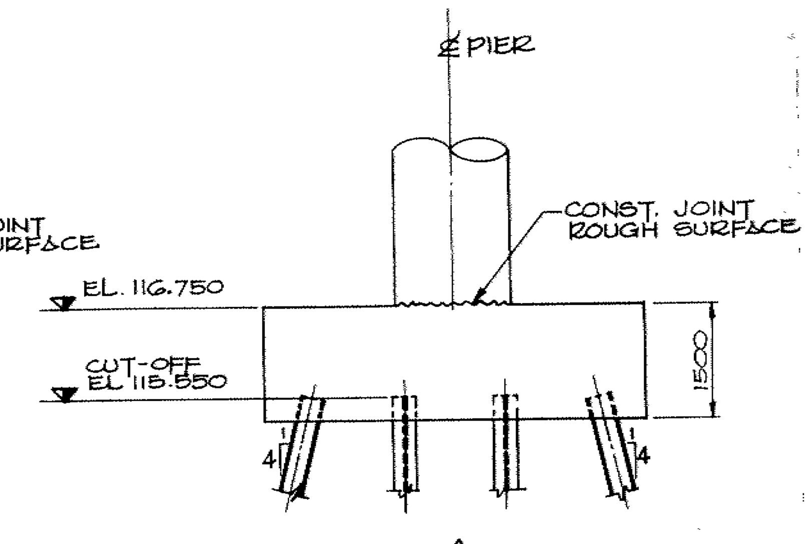
U.L.S.	AT ABUTMENTS	1250 kN
	AT PIER	1520 kN
S.L.S. II.	AT ABUTMENTS	880 kN
	AT PIER	1070 kN

NOTES:

1. ALL PILES TO BE HP 310×110.
2. PILE SPACING IS MEASURED AT UNDER-SIDE OF FOOTING.
3. PILES LENGTHS SHOWN IN TABLE ARE THEORETICAL LENGTHS BELOW CUT-OFF ELEVATION.
4. ALL PILES TO HAVE DRIVING SHOES.
5. PILES TO BE DRIVEN TO BEDROCK.



1
SCALE 1:50



2
SCALE 1:50

S. C. FOK — REGISTERED PROFESSIONAL ENGINEER — PROVINCE OF ONTARIO
REGISTERED PROFESSIONAL ENGINEER — PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS DATE	BY	DESCRIPTION

DESIGN S.C.F.	CHK GT	CODE OHBDC-83	LOAD CLASS A	DATE SEP. 1990
DRAWING T.C.	CHK GT	SITE 3-569	STRUCT. SCHEME	DWG. 4

F.

Ministry of Transportation
Ministère des Transports
Ontario

Engineering Materials Office
Foundation Design Section
Room 315, Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Tel: (416) 235-3731

1990 10 24

Dominion Soil Investigation Inc.
104 Crockford Blvd.
Scarborough, Ontario
M1R 3C6

Atten: Mr. Z. Ozden, P. Eng.

RE: Additional Analyses
Proposed Hwy. 17, McGee Road Crossing
W.P. 34-81-03, Site 3-569
District 9, Ottawa

Dear Sir,

This is in reply to your letter dated October 9, 1990 on the above subject. We understand that the height of fill is about 7 m and that the analyses in your report were done for a 6 m high fill. We would undertake to carry out the review of the impact of the increased height of fill for this structure ourselves.

We want to thank you for bringing the above matter to our attention.

Yours truly,

BIyer

Dr. Balu Iyer, P. Eng.
Sr. Foundation Engineer
for
M. Devata, P. Eng.
Chief Foundation Engineer

BI/MD/mmj

memorandum

w.P.

Ontario

To: K.G. Bassi
Head, Structural Section
7th Floor, Atrium Tower

Date: 1990 12 17

Atten: I.Hussain

From: Foundation Design Section
Room 315, Central Building

RE: McGee Road Underpass
W.P. 34-81-03, Site 3-569
District 9, Ottawa

We have reviewed the final drawings and contract documents in connection with the above project. Our comments from a foundation design and construction standpoint are as follows:

1. Excavation for the pile cap for the pier would extend below the groundwater level encountered during the foundation investigation. Advance dewatering would be required to facilitiate excavation for and construction of the above pile cap in the dry.

2. All piles are to be provided with driving shoes as per MTO Drawing No. DD-3301.

We have no other comments.

B.Iyer

Dr. Balu Iyer, P.Eng.
Sr. Foundation Engineer
for
M. Devata, P. Eng.
Chief Foundation Engineer

c.c. - E.C. Lane

7540-1478 (Rev. 10/89)

memorandum

Ontario

To: E.C. Lane
Head, Structural Section
Kingston

Date: 1990 12 27

From: Foundation Design Section
Room 315, Central Building

RE: Results of Additional Stability Analyses
Highway 17 - McGee Creek
W.P. 34-81-03
District 9, Ottawa

The foundation investigation for this project was carried out by our consultant, Dominion Soil Investigations Inc. The final report on this project was submitted to you on 1990 05 25.

Following receipt of the E-plans, we noticed that the embankment fill would be about 7 m high. Based on preliminary data received from your office, our consultant had provided recommendations for a 6 m high embankment. Additional stability analyses were carried out in our office to assess the stability of a 7 m high embankment. The factor of safety values at the end of construction would be about 1.26 to 1.29 and under long term conditions about 1.79.

Based on this data, we conclude that the 7 m high embankment shall be constructed using 2H to 1V slopes as recommended in the foundation report.

Please call this office if you need elaboration on any aspect of this memo.

B Iyer

Dr. B. Iyer, P. Eng.
Sr. Foundation Engineer

BI/jb

cc: K.G. Bassi/I. Hussain

Ministry of Transportation
Ontario

FAXGRAM

PLEASE TYPE

DATE 1991 03 21 PAGE 1 OF 2

TO: DARWYN SPROULE STRUCT. SECTION EASTERN REGION	FROM: BALU IYER FOUND. DESIGN SECTION

SUBJECT: TYPICAL CROSS SECTION TO ACCOMMODATE 1.5H TO 1V SLOPE AT McGEE RD.
W.P. 34-81-03 (01-04)

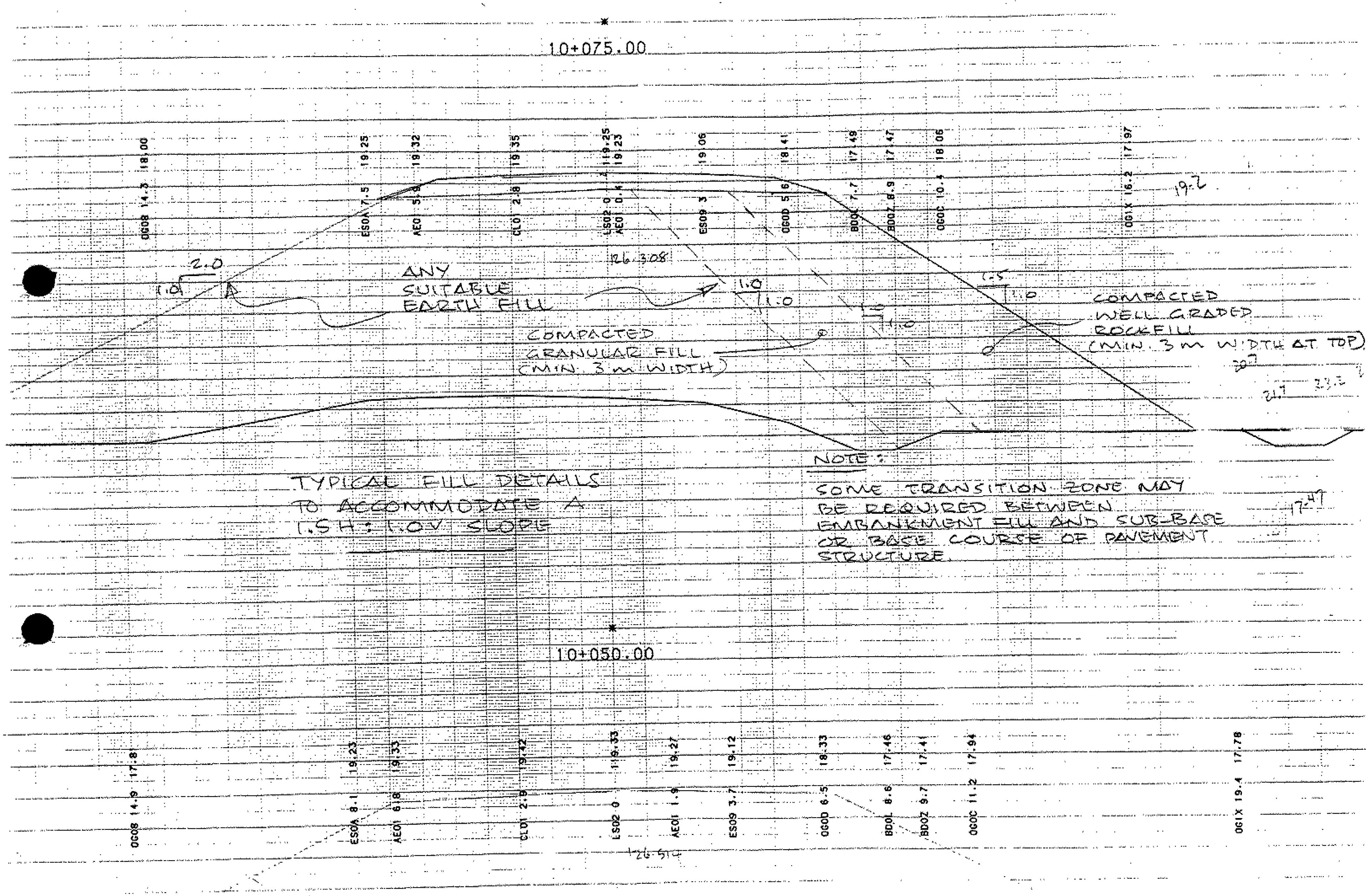
1. – USE 2H TO 1V SLOPE AS PER PREV. RECOMMENDATION

2. – ALTERNATIVELY, STEEPEN SLOPE TO 1.5H TO 1V ON ONE SIDE AS SHOWN ON ATTACHED SKETCH.

– OPTION 1 ABOVE IS OUR PREFERRED SOLUTION

CC D. MOON, P&D

ADM-S-481 87-05



10+075.00
OG08 14.3 18.00
ESOA 7.5 19.25
AE01 5.8 19.32
CL01 2.8 19.35
LS02 0 19.25
AE01 0.6 19.23
ES09 3 19.06
OGOD 5.6 18.41
BDOL 7.7 17.49
BDOZ 8.9 17.47
OGOC 10.4 18.06
OG1X 16.2 17.97
19.2
126.3.08
2.0
1.0
ANY SUITABLE EARTH FILL
1.0
1.0
1.5
1.0
1.5
1.0
COMPACTED GRANULAR FILL (MIN. 3 m WIDTH)
COMPACTED WELL GRADED ROCKFILL (MIN. 3 m WIDTH AT TOP)
NOTE:
SOME TRANSITION ZONE MAY BE REQUIRED BETWEEN EMBANKMENT FILL AND SUB-BASE OR BASE COURSE OF PAVEMENT STRUCTURE.
TYPICAL FILL DETAILS TO ACCOMMODATE A 1.5H : 1.0V SLOPE
17.47
10+050.00
OG08 14.9 17.8
ESOA 8.1 19.23
AE01 6.8 19.33
CL01 2.9 19.42
LS02 0 19.33
AE01 1.9 19.27
ES09 3.7 19.12
OGOD 6.5 18.33
BDOL 8.6 17.46
BDOZ 9.7 17.41
OGOC 11.2 17.94
OG1X 19.4 17.78
126.514

DOMINION SOIL INVESTIGATION INC.
Founded 1956
CONSULTING SOIL & FOUNDATION ENGINEERS
104 Crockford Blvd., Scarborough, Ontario M1R 3C6 Tel: (416) 751-6565 Fax: (416) 751-7592

October 9, 1990

RECEIVED
OCT 11 1990
FOUNDATION DESIGN SECTION

Ref. No. 89-11-14

Ministry of Transportation
Foundation Design Section
Central Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Att: **Mr. M.S. Devata, P. Eng.**
Chief Foundation Engineer

Re: **Foundation Investigation**
Proposed Underpass Structure
Highway 17 (417) and McGee Road
Site 3-569, W.P. 34-81-03
District 9, Twp. of West Carlton

Dear Sir,

Further to our meeting on September 22, 1990, with Dr. B. Iyer in your office, this letter is to confirm our discussions regarding the above captioned project.

As you are aware, our foundation report was based on 6 m high approach fills, in accordance with the information that was available at the time of the preparation of our report. As you also know the plans and profiles for the project were prepared several months after the submission of our report (i.e. after the E-plan became available). We recently noted that the profile grade was raised by about 1.0 m, thus increasing the height of the approach fills from 6 to 7 ± m.

As you know this will increase the stresses on the weak clays underlying the site, leading to an increase in settlements and reduction in the factor of safety against a rotational failure. It may also increase the negative skin friction (down-drag forces) on the piles and may possibly have other implications.

To file

If you wish us to review our calculations and look into this matter we will be happy to do so. We recommend that an allowance of $2,400.00 be set aside for this purpose.

If you have any questions regarding this letter please feel free to call us.

Yours very truly
DOMINION SOIL INVESTIGATION INC.

Z.S. Ozden, P. Eng.

MEMORANDUM

To: K.G. Bassi
Head, Structural Section
MTO, Central Region
7th Floor, Atrium Tower
Downsview, Ontario

Date: 1990 08 07

Attn: Dr. I. Hussain

From: Foundation Design Section, MTO
Room 315, Central Building, Downsview

Re: McGee Road Underpass
W.P. 34-81-03, Site 3-569
Dist. 9, Ottawa

We have reviewed the General Arrangement Drawing No. 3-569-P1 for the above noted structure. Our comments are as follows:

-- The excavation for the pile cap at the pier location will be carried down to elevation 115.2m within silty sand material. The groundwater elevation at this location is expected to be at elevation 117.8m. A dewatering scheme will have to be implemented in order to prevent boiling at the base of the pile cap excavation due to unbalanced hydrostatic head.

-- The drawing indicates that the piles shall be driven with driving shoes. The driving shoe details should be as per MTO Drawing No. DD-3301.

-- It is expected that pile lengths will be shown on the final drawings and grading of approach fills will also be defined. The grading should be 2H:1V or flatter.

-- As stated in the foundation report, due to settlement at approach fills, abutment piles will be subjected to downdrag forces which will result in reduced pile capacities for the abutment foundations. It is believed that this aspect has been accounted in the design.

Ken Ahmad, P. Eng.
Foundation Engineer

For

M. Devata, P. Eng.
Chief Foundation Engineer

cc: E.C. Lane

(RETAIN THIS COPY FOR FOLLOW-UP)

7540-1021

SEND TO Iqbal Husain
Structural Office

FROM T. Sanguiliano **DEPT.** Foundation Design **DATE** 90 07 26

SUBJECT WP 34-81-03/Site 3-569 ; McGee Creek & Hwy 17

Further to our telephone conversation dated 90 07 26 regarding the determination of the lateral earth pressures, it is hereby recommended that the at-rest earth pressure be applied for a rigid and unyielding structure and the active ~~[illegible]~~ earth pressure be applied for ~~the~~ a flexible and yielding structure. Compaction pressures, as suggested on pg 16 of the Foundation report, need not be considered a design parameter because of current MTO ~~[illegible]~~ specifications

REPLY that restricts heavy vibratory equipment within a distance of the retaining structure equivalent to the height of the fill being placed.

T. Sanguiliano

REPLY FROM

REPLY DATE

DOMINION SOIL INVESTIGATION INC.
Founded 1956
CONSULTING SOIL & FOUNDATION ENGINEERS
104 Crockford Blvd., Scarborough, Ontario M1R 3C6 Tel: (416) 751-6565 Fax: (416) 751-7592

December 28, 1989

Ref. No. 89-11-14

Ministry of Transportation
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Att: **Mr. M. S. Devata, P. Eng.**
Chief Foundation Engineer

Re: **W.P. 34-81-03**
Foundation Investigation
Proposed Underpass Structure
Highway 17 and McGee Road
Lot 10-11, Conc. 1V-V
Township of West Carlton

Dear Sir,

We have completed the field work for the above captioned project and, as requested, we are presenting herewith our preliminary findings.

The field work for the project was carried out during the period of December 4 to 12, 1989 and consisted of drilling ten sampled boreholes and performing three dynamic cone penetration tests. During the performance of cone tests the very dense soil strata were augered through, in some cases.

The locations of the sampled boreholes and dynamic cone penetration tests are shown on the preliminary Borehole Location Plan, enclosed with this letter. The field work was performed under the supervision of a Professional Engineer from our office.

Details of the subsurface conditions encountered at each borehole location, including the results of in situ testing, are presented on the preliminary 'Record of Borehole' sheets. The ground surface elevations at the borehole locations were provided to us by M.T.O. surveyors.

In general, below the pavement materials, the site is covered by a shallow deposit of sand fill with occasional gravel and this extends to depths ranging from 1.1 to 1.8 m below the ground surface.

At Boreholes 1, 101, 103, 4 and 5, the fill is underlain by a layer of organic topsoil and/or a somewhat organic silt to silty clay. The thickness of the organic or somewhat organic deposits ranges from 0.2 (B.H. 101) to 0.7 m (B.H. 1).

Below the surficial fill and underlying organic soils (where these occur) a stratum of sandy silt to silty sand was contacted in all the boreholes, ranging in thickness from 1.0 m in B.H. 7 to more than 4 m in B.H. 102. Standard Penetration resistances ('N'-values) measured in this deposit ranged from 14 to more than 50 blows/0.3 m, indicating a compact to very dense stratum.

Underlying the surficial deposits described in the preceding paragraphs, the predominant overburden type throughout much of the site is a silty clay (Champlain Sea) deposit. 'N'-values recorded in this material generally range from 4 to 10 blows/0.3 m and in situ undrained shear strengths as measured by field vane tests are generally 25 to 100 kPa with a sensitivity generally in the range of 4 to 10. From these results, the consistency of the deposit is described as generally 'firm to stiff'.

At some of the borehole locations, the Leda (Champlain Sea) clay is stratified with silt to silty sand layers and at several locations these layers are quite competent with 'N'-values in excess of 50 blows/0.3 m (e.g. Boreholes 2, 102, 3 and 6). These layers necessitated augering through to advance dynamic cone penetration tests. In view of the fact that no such competent zones were found in B.H. 103 which is only about 10 m away from B.H. 3, the presence of such layers are considered to be quite variable across the site.

At Boreholes 1, 2, 102, 103, 6 and 7, a sandy silt till deposit with some gravel and clay content was encountered directly overlying the bedrock. The thickness of this stratum ranges from 0.2 m in B.H. 1 to 2.7 m in Borehole 102. 'N'-values of 11, 17 and 22 blows/0.3 m were measured, indicating a competent material.

Bedrock was inferred from refusal to augering or dynamic cone penetration tests at majority of boreholes or test locations. At Boreholes 1, 2, 3 and 103, however, the bedrock was penetrated and proven by diamond drilling and rock coring (3.0, 3.1, 1.2 and 1.9 m, respectively). The rock consists of a light grey limestone with some dark grey argillaceous bands or zones. The proven and/or inferred surface of the bedrock ranges from 10.2 m (Elevation 109.1 m) at B.H. 5 to 13.1 m (Elevation 106.1 m) at B.H. 2 location. Thus an elevation difference of 3.0 m over a horizontal distance of about 100 m is indicated.

The groundwater at the time of the investigation was recorded at depths generally ranging between 1.5 and 2.0 m below the prevailing ground surface.

Our preliminary analysis of the borehole findings indicates that because of the presence of weak clay of variable thickness and consistency throughout the site, the use of end bearing driven piles will be the most suitable foundation type to support the proposed bridge which will have two equal spans of 31 m.

The piles would have to be driven to refusal on the surface of the bedrock which was generally contacted at depths ranging between 11 and 13 m below the ground surface at the proposed abutment and pier locations. Because of the presence of occasional very dense/hard layers in the weak clay (as evidenced by very high resistance during the performance of dynamic cone penetration testing) the use of steel H-piles with reinforced flanges would be better suited for this project, for improved driving resistance. In some instances pre-boring may also be required. The estimated pile capacities for some common sizes of steel H-piles driven to a final set of about 1 blow for 1 mm penetration with a pile driving hammer capable of delivering an energy of 40,000 to 70,000 Joules/blow are tabulated below:

Size	**Factored Capacity at Ultimate Limit States (Qf)**	**Capacity at Serviceability Limit States Type II (Qs)**
HP 310 x 110	1450 kN	1000 kN
HP 310 x 79	1050 kN	700 kN

Depending on the proposed grades for the structure, the existing grade at the abutment locations may be raised by about 6 m. Based on our preliminary analysis (which will be confirmed by laboratory testing and a more detailed analysis) there will be an adequate factor of safety against a shear failure under the weight of the fill embankment constructed with normal 2:1 side slopes. The weight of the approach fills will, however, induce settlements in the underlying weak clay. This settlement, in addition to affecting the performance of the road, will cause a down-drag on the end bearing pile foundations due to negative skin friction. The down-drag on the piles will have to be carefully evaluated and the load carrying capacity of the piles supporting the abutments will have to be adjusted accordingly.

At present laboratory testing is being conducted and this will be followed by an analyses of the conditions. Meanwhile, we trust that this preliminary report will be sufficient for your present purposes.

Yours very truly
DOMINION SOIL INVESTIGATION INC.

Abbas Ali Khan

Abbas A. Khan, P. Eng.

Z.S. Ozden, P. Eng.

AAK/ma

Encl. Preliminary Record of Borehole Sheets
Preliminary Borehole Location Plan