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GEOCRES No. 3165-152

DIST. 9 REGION

W.P. No. 120-87-08

CONT. No. 90-36

W. O. No.

STR. SITE No.

HWY. No. 417

LOCATION E-S Ramp over Acres Rd.
Structure #5

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:



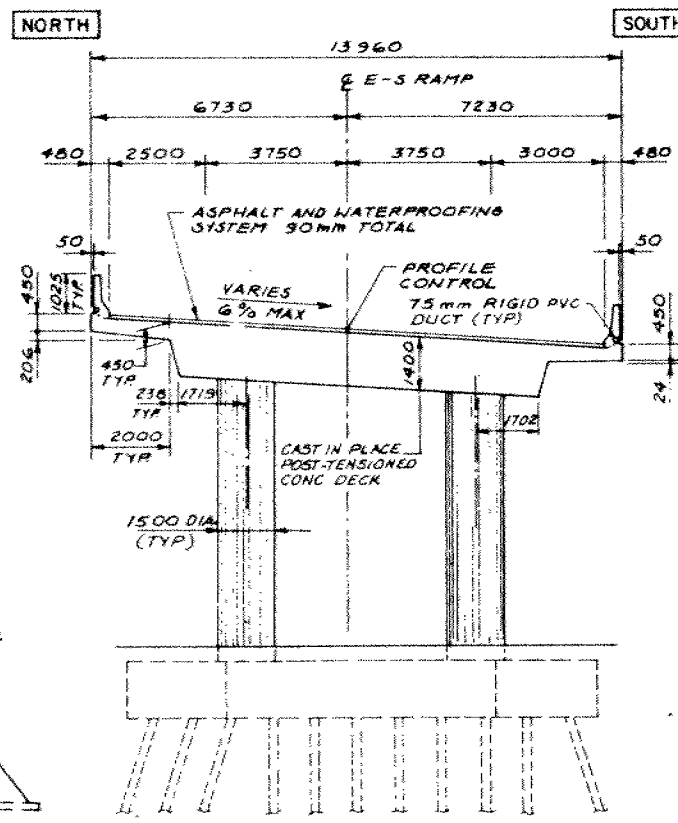
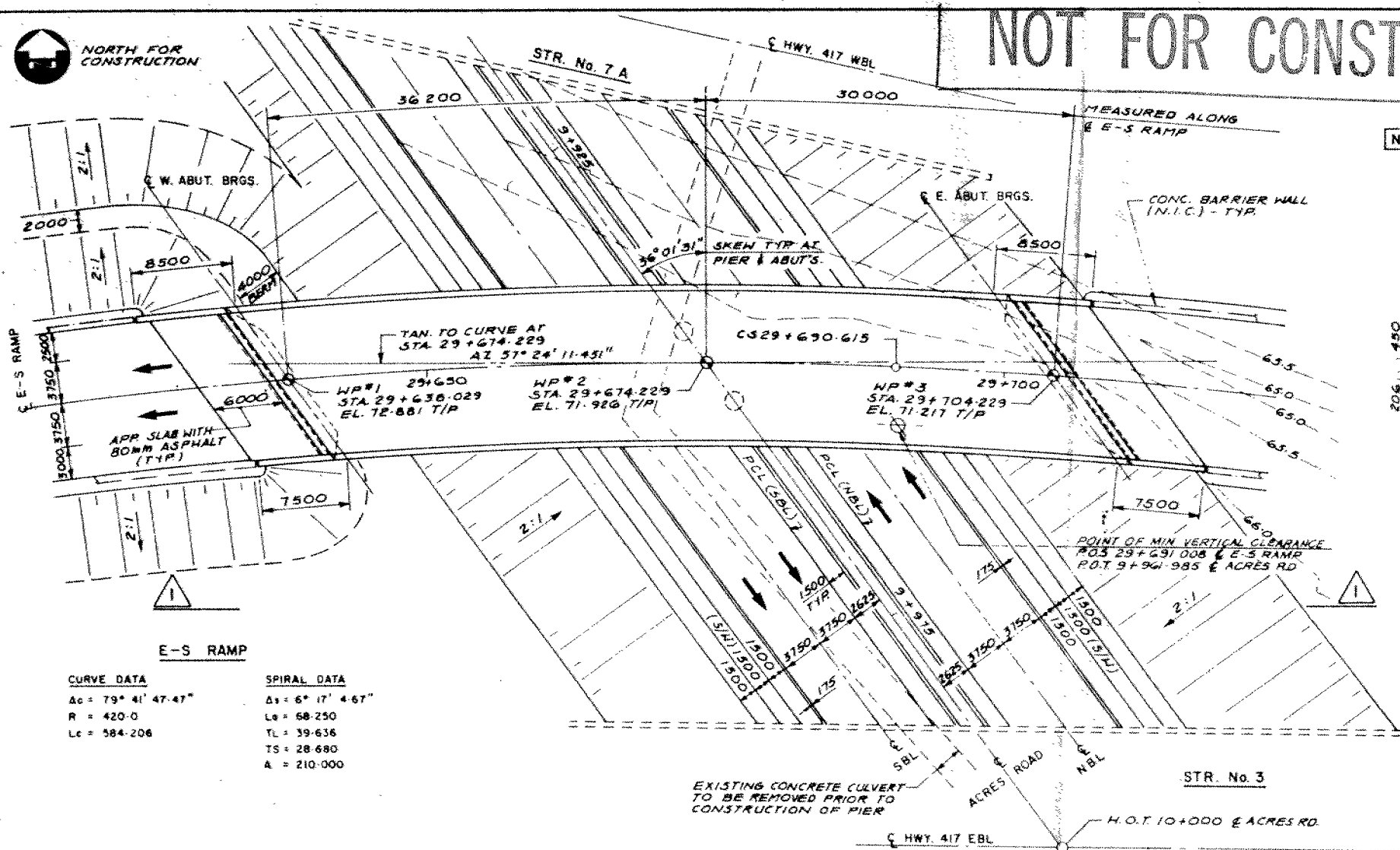
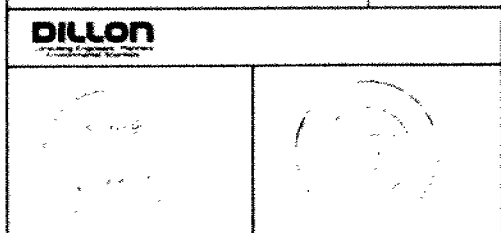
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST 9
CONT No
WP No 120-87-08
EAST TO SOUTH RAMP OVER
ACRES RD. O'PASS (STR. No.5)
GENERAL ARRANGEMENT



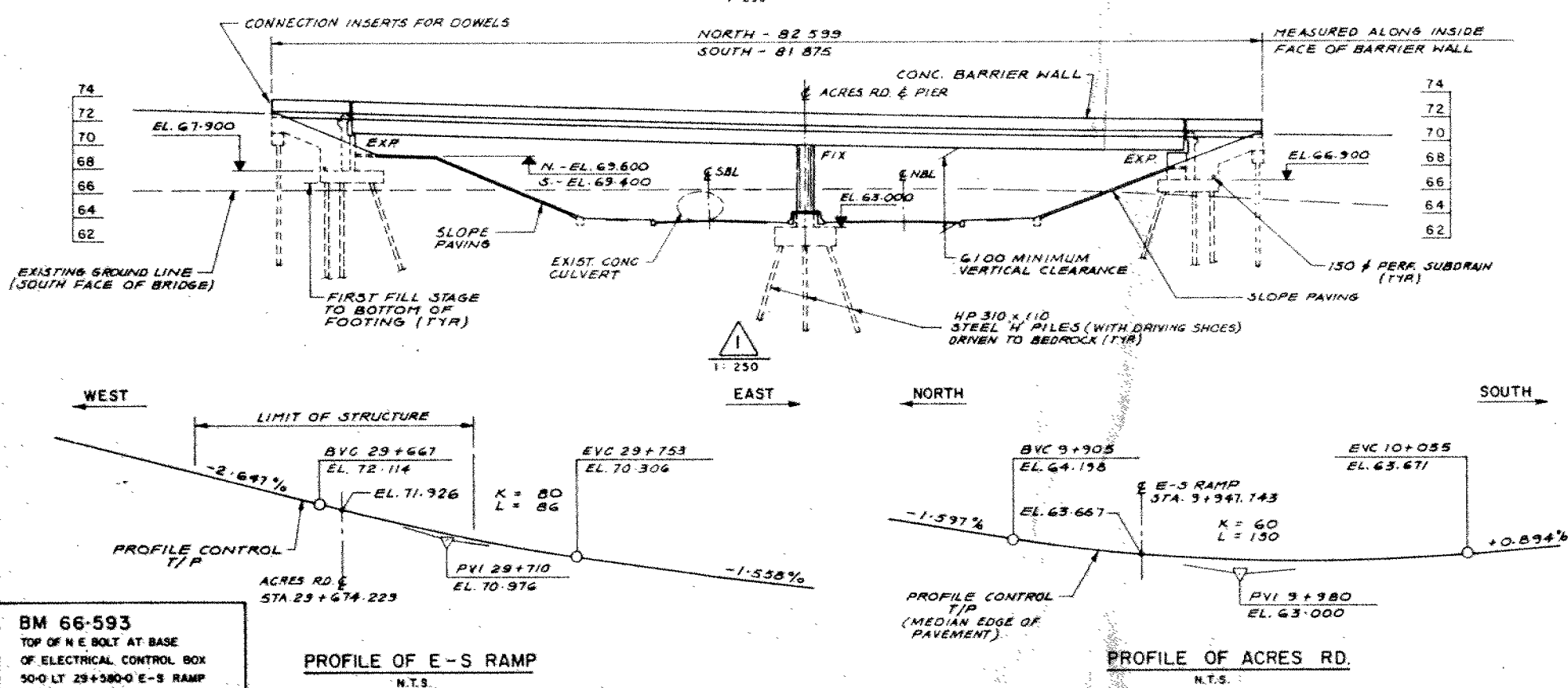
SHEET



- GENERAL NOTES:**
- CLASS OF CONCRETE
 - DECK 35 MPa
 - REMAINDER 30 MPa
 - CLEAR COVER TO REINFORCING STEEL
 - FOOTINGS 130 ± 25
 - ABUTMENTS AND WINGWALLS
 - FRONT FACE 90 ± 20
 - BACK FACE 70 ± 20
 - PIERS
 - TOP 70 ± 20
 - BOTTOM AND SIDES 50 ± 10
 - DECK 70 ± 20
 - REMAINDER UNLESS OTHERWISE NOTED
 - REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX C DENOTE COATED BARS.
- CONSTRUCTION NOTES**
- COMPACTED FILL, MAXIMUM GRAIN SIZE 75 mm SHALL BE PLACED UP TO THE BOTTOM OF FOOTING ELEVATION PRIOR TO DRIVING PILES.
 - 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. BOREHOLE LOCATION AND SOIL STRATA
 3. FOUNDATION LAYOUT
 4. PIER AND FOOTING REINFORCEMENT
 5. WEST ABUTMENT LAYOUT AND REINFORCEMENT
 6. EAST ABUTMENT LAYOUT AND REINFORCEMENT
 7. WINGWALL DETAILS AND REINFORCEMENT
 8. DECK DETAILS AND BEARINGS
 9. LONGITUDINAL PRESTRESSING DETAILS
 10. TRANSVERSE PRESTRESSING DETAILS
 11. DECK REINFORCEMENT I
 12. DECK REINFORCEMENT II
 13. JOINT ANCHORAGE AND ARMOURING
 14. NORTH BARRIER WALL
 15. SOUTH BARRIER WALL
 16. 6,000 MM APPROACH SLAB
 17. FLAGSTONE/CONCRETE SLOPE PAVING
 18. AS CONSTRUCTED ELEV. AND DIM.
 19. BRIDGE DATE AND SITE NUMBER DATA
 20. STANDARD DETAILS
 21. EMBEDDED WORKS
 22. QUANTITIES - STRUCTURE I
 23. QUANTITIES - STRUCTURE II

- LIST OF ABBREVIATIONS**
- W.P. - WORKING POINT
 - T/P - TOP OF PAVEMENT
 - T/C - TOP OF CONCRETE
 - S/W - SIDEWALK
 - PCL - PROFILE CONTROL LINE
 - NIL - NOT IN CONTRACT
- APPLICABLE STANDARD DRAWINGS**
- DD - 3503 MINIMUM GRANULAR BACKFILL REQUIREMENTS
 - OPSO - 508.02 BRIDGE DECK WATERPROOFING



PRINTED
JAN 8 1990
M. M. DILLON LTD.

BM 66-593
TOP OF N.E. BOLT AT BASE
OF ELECTRICAL CONTROL BOX
50.0-LT 29+980.0 E-S RAMP

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION

DESIGN M.V.	CHK C.L.D.	CODE 08H/C - 83	LOAD CLASS-A	DATE NOV. 89
DRAWN C.M.G.	CHK M.W.	SITE 3-535	STRUCT	SCHEME
				DWG 1

memorandum



To: K.G. Bassi
Head, Structural Office
Atrium Tower

Date: 1990 01 31

Attn: I. Hussain

From: Foundation Design Section
Room 315, Central Building

Re: E - S Ramp for Hwy. 416/417 Over Acres Rd. O'Pass
(Structure #5)
W.P. 120-87-08, Site 3-535
Hwy. 417/416, District 9, Ottawa

As requested in your memo dated January 10, 1990, the submitted final drawings and provisions have been reviewed by this section and the following comments are provided.

- 1) Unsuitable materials underneath the proposed embankment fill should be excavated before fill placement.
- 2) Considering an approximate 75 mm of settlement, the approach fills should be constructed as far in advance of the structure foundations as scheduling permits.
- 3) Side Slope

As discussed in our review on the Preliminary Drawing P-1 (June 19, 1989), it was recommended that consideration might be given to employing a flatter side slope of $2\frac{1}{2}H:IV$ if the mid-height stabilizing berm be eliminated. However, the General Arrangement Drawing 3-535-Dwg.1 still reveals 2 m berm in the transverse direction at west approach based on the original foundation report.

- 4) 2H:IV slope should be indicated towards the both abutments on Dwg. No. 1 on longitudinal Section.

We have no further comments. If you have any questions, please contact this office.

A handwritten signature in dark ink, appearing to read "B. Iyer", with a horizontal line underneath.

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

for

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

MSD/BI/jb

cc: E.C. Lane

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 120-87-08

DIST 9

HWY 416/417

STR SITE 3-52-535

East to South Ramp for Hwy. 416/417
Over Acres Road, Structure No. 5

DISTRIBUTION

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File

FOUNDATION INVESTIGATION REPORT
For
East to South Ramp For Hwy. 416/417
Over Acres Road, Structure No. 5
WP 120-87-08, Site 3-52-535
District 9, Ottawa

INTRODUCTION

This report summarizes the information obtained from a foundation investigation carried out at the above-mentioned site during the period of July 5 to July 14, 1988. A two span structure is proposed to carry East to South Ramp for Hwy. 416/417 over the realigned Acres Road.

Five boreholes (BH #5-1 to BH #5-5) were advanced and sampled as part of this project by means of hollow stem augers with washboring techniques and using a conventional diamond drill (BX Casing and BX Rock Core barrel) adopted for soil and rock sampling purposes. These boreholes extended down to depths of 15.7 and 28.6 metres below the existing ground surface.

This report contains factual information together with discussion and recommendations pertaining to the subsurface conditions, structure foundations, approach embankments and related earthworks for the Structure No. 5 as shown on Drawing No. 1208708-A.

SITE DESCRIPTION AND GEOLOGY

The proposed structure site is located immediately south of the existing Highway 417 between Richmond Road and the existing Acres Road in the City of Nepean, Ottawa-Carleton Municipality. The topography of the area is generally flat to gently undulating with the land in the immediate vicinity being used for farming purposes. Residential development exists north of the site.

Physiographically, the site lies in the area known as the Ottawa Valley clay plains founded in the Lowlands of the St. Lawrence. The subsoil consists of clay plains interrupted by ridges of rock or sand. Fault scarps are also evident within the area, an illustration of the numerous normal faults that dominate the region. The bedrock in the area is of the Rockcliffe and Gull

River formations of the middle Ordovician period. It consists of interbedded fine grained quartz sandstone, silty dolostone, and limestone. The overburden was deposited during and immediately following the Wisconsin glaciation at which time the area was depressed from the effect of the glaciation. Following the retreat of the glacier, the brackish waters of the Champlain Sea flooded the area and then gradually receded as the land rebounded with the deposition of sediments to it's present level.

FIELD INVESTIGATION AND LABORATORY ANALYSES

The fieldwork for the site investigation was carried out between July 5 and July 14, 1988 and consisted of five (5) sampled boreholes accompanied by dynamic cone penetration tests. Continuous flight hollow stem auger equipment and washboring techniques with NX or BX Casing were used to advance the boreholes in the overburden. Soil samples were retrieved at selected intervals by a split spoon sampler or Shelby tube in accordance with the Standard Penetration Test (ASTM D1586). In situ vane tests were also carried out to cohesive soil. Samples were identified in the field and then returned to the laboratory for appropriate testing. Bedrock was proven at a number of location minimum 1.8 m using conventional rock coring methods.

Water levels were obtained in the open boreholes until approximate stabilized levels were observed.

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

To identify the behaviour, gradation, properties and characteristics of the soil, various laboratory testing were performed as follows:

- 1) Atterberg Limit Tests
- 2) Grain Size Analyses
- 3) Natural Moisture Contents
- 4) Undrained Unconsolidated Tests (Quick Triaxial)
- 5) Unconfined Compression Test
- 6) Consolidation Tests

Laboratory tests results have been summarized and are included in the Appendix of this report.

SUBSURFACE CONDITIONS

The subsoil conditions are generally consistent across the site. The surficial layer consists of a generally soft to very stiff cohesive silty clay to clayey silt which extends to a maximum thickness of 2.5 metres. Underlying this layer is a deposit of clayey silt interbedded with irregular layers or seams of sandy silt. The maximum thickness of this deposit is about 7.6 metres. A deep deposit of silty sand to sand is the subsequent underlying deposit with a maximum thickness of about 19.1 metres and this in turn underlain by a heterogeneous mixture of sand, gravel and boulders (Glacial Till). Approximately 3.1 to 5.7 metres of the till deposit was found before encountering the silty dolostone bedrock.

It should be noted that in the vicinity of east abutment and approach of the Structure No. 5, upper two layers (silty clay to clayey silt layer and clayey silt with interbedded sandy silt layer) are gradually diminished. In stead, fill material was encountered adjacent to the existing Graham Creek. Fill material consists of sandy silt or clayey silt to silt as shown on Record of boreholes (BH #5-4 and #5-5).

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

Fill Material

The fill material was encountered in the vicinity of the northeast portion of the site at two borehole locations (BH #5-4, #5-5). This fill consists of a brown Sandy Silt or Clayey Silt to Silt. The thickness of this layer varies from 0.8 metres at BH #5-4 to 1.3 metres at BH #5-5 as shown on Record of boreholes. No Atterberg Limit Tests and Grain Size Distribution Analysis were carried out. However, through visual observation, it is apparent that the fill material is similar to the surficial material which was found adjacent to the site.

Silty Clay to Clayey Silt

This stratum was encountered in most of the boreholes except near the east portion of the site (Boreholes #5-1, #5-2 and #5-3). This material consists of a silty clay to clayey silt with an average thickness up to 2.5 metres. The material changes in colour from brown to grey at approximately elevation 64.9 metres at Boreholes #5-1, #5-2 and #5-3.

Atterberg Limit tests were performed on these samples and the results are plotted on Figure 1 and summarized as follows:

<u>Property</u>	<u>Range (%)</u>
Natural Moisture Content (w)	23.5-42.0
Liquid Limit (w_L)	22.0-57.0
Plastic Limit (w_p)	12.5-36.0
Plasticity Index (I_p)	16.0-29.0
Unit Weight (kN/m^3)	17.6-19.3

From the plasticity chart (see Figure 1) it is evident that the layer can be classified as an inorganic silty clay to clayey silt with intermediate to low plasticity (CI or CL).

Grain size distribution tests were carried out on these materials. Figure 2 in the Appendix shows the results in envelope form.

Undrained shear strength of the soil were determined both by in situ vane tests and by laboratory tests, namely undrained unconsolidate (quick triaxials) and unconfined compression tests. The results are plotted on the Record of boreholes in the Appendix and summarized as follows:

<u>Undrained Shear Strength (C_u)</u>	<u>kPa</u>	<u>Sensitivity</u>
Field Vane	48-126	3.0-5.7
Laboratory Results	30-72	-

As shown on the above table, it can be concluded that the laboratory testing provided lower values possibly attributable to disturbance during sampling, transportation and testing. Recommended shear strength for this deposit can be assumed to be within the range of 60 to 70 kPa. Based on this conclusion, the soil has generally a firm to very stiff consistency. The sensitivity of the soil is generally low to moderate.

The results (e-log P curves) of two consolidation tests on representative samples obtained in the silty clay to clayey silt deposit are shown on Figure 5. These tests indicated that this cohesive stratum has been preconsolidated in the past to an effective pressure ranging from 340 kPa to 450 kPa in excess of the existing effective overburden pressure. The details of the results are as follows:

<u>Parameters</u>	<u>Ranges*</u>
Preconsolidation pressure, P_c (kPa)	340-450 kPa
Initial Void Ratio (e_0)	0.863-1.250
Compression Index (C_c)	0.45-0.94

*Test data of similar soil from adjacent investigations
(WP 120-87-06) incorporated in results.

Clayey Silt with Interbedded Sandy Silt

Underlying the surficial deposit of silty clay to clayey silt, a layer of grey clayey silt with interbedded sandy silt was encountered. This stratum extends to depths ranging from 0.8 metres to 10.1 metres below the ground surface. The thickness of the stratum varies between 1.7 and 7.6 metres.

The results from the Atterberg Limit Test performed on this material are summarized as follows:

<u>Property</u>	<u>Range (%)</u>
Natural Moisture Content (w)	23.0-42.0
Liquid Limit (w_L)	16.0-34.5
Plastic Limit (w_p)	12.0-15.0
Plasticity Index (I_p)	2.0-21.0
Unit Weight (kN/m^3)	19.3-19.9

From the plasticity chart (Figure 1), it is evident that the layer can be classified as an inorganic clayey silt with interbedded Sandy Silt with low plasticity (CL or CL-ML).

Grain size distribution tests were carried out on these materials. Figure 2 in the Appendix shows the results in an envelope form.

Undrained shear strength of the soil were determined both by the in situ vane tests and by laboratory tests, namely undrained unconsolidated (quick triaxial) and unconfined compression tests. The results are plotted on the Record of boreholes in the Appendix and summarized as follows:

<u>Undrained Shear Strength</u>	<u>kPa</u>	<u>Sensitivity</u>
Field Vane	51-104	2.7-4.0
Laboratory Results	17-55	-

Due to the irregular nature of the deposit, that reveals numerous seams and layers of Sandy Silt interbedded within the Clayey Silt, the results provided in the above table are not necessarily indicative of the shear strength of the Clayey Silt portion. In view of this consideration, the consistency of the Clayey Silt portion can be described as soft to very stiff. The Sandy Silt portion was generally very loose in denseness. For design purposes, an undrained shear strength of 40 kPa can be assumed for this stratum.

The results (e-log P curves) of two consolidation tests on representative samples are shown on Figure 6. These tests indicated that the Clayey Silt has been preconsolidated in the past to an effective pressure ranging from 190 kPa to 327 kPa in excess of the existing effective overburden pressure. The details of the results are as follows:

<u>Parameters</u>	<u>Ranges*</u>
Preconsolidation pressure, P_c (kPa)	190-327
Initial Void Ratio (e_0)	0.617-1.111
Compression Index (C_c)	0. 0 78-0.87

*Test data of similar soil from adjacent investigations (WP 120-87-06) incorporated in results.

Silty Sand to Sand

Silty Sand to Sand was encountered below the Clayey Silt with interbedded Sandy Silt layer. The thickness of this layer ranges from 11.2 metres at BH #5-2 to 19.1 metres at BH #5-4. The material changes in colour from brown to grey at an approximate elevation of 57 metres on the east portion of the site (BH #5-5).

This deposit contains minor variations in gravel content throughout its thickness. Generally, the deposit contains trace of gravel, but at some locations, considerable gravel (in excess of 40%) was encountered. Grain size distribution analysis indicate that the soil varies between a silty sand to Sand. This layer is basically non-plastic. Figure 3 in the Appendix shows the results of grain size distribution tests in an envelope form.

In this stratum, the 'N' values ranged from 4 to over 70 blows/0.3 m indicating a state of compaction described as loose to very dense.

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

Underlying the silty sand to Sand deposit at a depth ranging from 21.3 to 27.3 metres, a heterogeneous mixture of Sand, Gravel and boulders of glacial origin

was encountered. The thickness of this stratum ranges from about 3.1 m at BH #5-2 to 5.7 metres at BH #5-4. Rock coring techniques were required to penetrate occasional boulders within the stratum. This stratum may be described as a heterogeneous mixture of Sand, Gravel and boulders. Figure 4 shows the result of grain size distribution tests in an envelope form for these materials.

In this stratum, the 'N' values ranged from 28 to over 100 blows/0.3 metres indicating a state of compaction described as compact to very dense.

Bedrock

The glacial till deposit is directly underlain by bedrock of the Rockcliffe and Gull River Formations and was proven at various locations by obtaining up to 1.8 metres of rock core samples. The bedrock consists mainly of a Silty Dolostone. Minor beds of Sandstone and Limestone were also found interbedded in the rock formation. Detailed description of the rock are attached in the Appendix entitled "Description of Rock Core".

Core recoveries and rock quality designation (RQD) were determined in situ and also in the laboratory to evaluate the competence and integrity of the rock. Based on these results, the rock can be classified as medium strong to strong rock and predominantly unweathered.

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes. Groundwater level in the boreholes was found to range between 60.9 metres at BH #5-2 and 64.9 metres at BH #5-1 which corresponds to depths of 5.3 metres to 1.4 metres below the existing ground surface.

DISCUSSION AND RECOMMENDATIONS

The recommendations in this report apply to the bridge structure and related approaches.

It is proposed to construct an overpass structure that will carry East to South Ramp lanes over the realigned Acres Road. This structure is a component of the proposed Hwy. 416/417 interchange which involves numerous proposed structures.

The new structure No. 5 will be located immediately south of the existing Hwy. 417 between the Acres Road and Richmond Road. The proposed structure is a two-span structure having an approximate length and width of 78 metres (39 + 39 m) and 13 metres, respectively. The proposed profile grade of the Ramp lanes is approximately at elevation 74.5 metres which is equivalent to approach embankment fill height of approximately 8.5 metres. In addition, an excavation cut of approximately 2.0 metres will be required to achieve the realigned Acres Road profile grade of 64.0 metres.

Recommendations pertaining to the following geotechnical considerations should be included:

Approach Embankments

- slope stability
- settlements

Structure Foundations

Other Considerations

- lateral earth pressure on structure
- dewatering

In view of the presence of the compressible surface layer of Silty Clay to Clayey Silt and the underlying Clayey Silt with interbedded Sandy Silt which is of low shear strength, slope stability and settlement of the approach embankments are the major problems anticipated at this site. Consequently, these will be discussed in detail.

APPROACH EMBANKMENTS

Stability Considerations

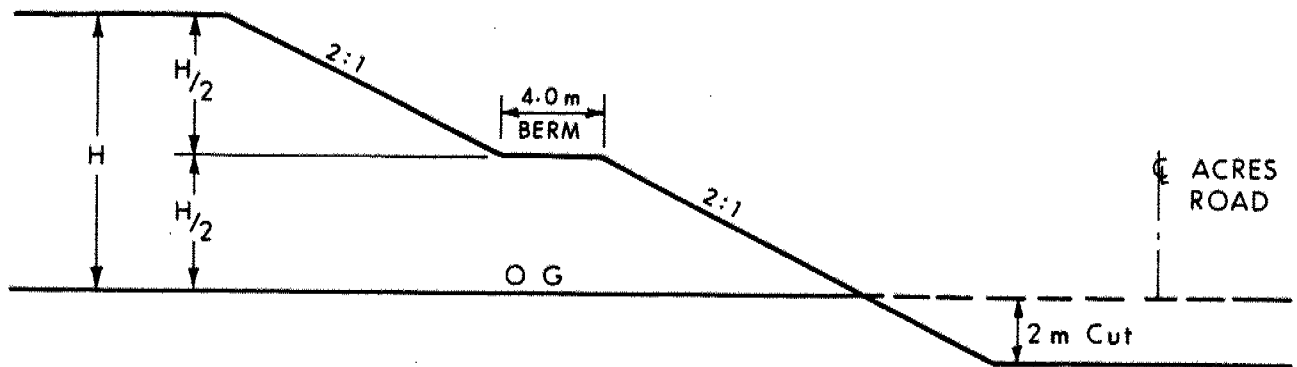
Stability analyses were carried out to evaluate the effect of the approach fills to the overall stability both in the longitudinal and transverse directions, the internal stability of the fills, and the stability of the excavation cut required for the realigned Acres Road.

A total stress analysis was applied for calculations of slope stabilities pertaining to the fills while an effective stress analysis was used to examine the stability of the cut. A minimum factor of safety of 1.3 was incorporated for all analyses. At this location maximum height of approaches will be in the order of 8.5 m as mentioned previously. Since the grades may be revised at a later date, consequently analyses were carried out for approaches higher than 7.0 m. Based on the analyses, the following conclusions have been derived:

Approach fills up to 8.0 metres in height both in the longitudinal and transverse directions at the east and west approaches will be stable provided they are constructed with standard 2H:1V slopes. For fills exceeding 8.0 metres, nominal mid-height stabilizing berms will be required. The berm length requirements for various heights of fill and the soil parameters, surface geometry and groundwater levels used in the analysis is provided on Figure 7 and 8 in the Appendix for both the longitudinal and transverse directions, respectively.

Any localized softened and/or surficial organic soil should be removed within the planned limits of the fill prior to its placement. The fills should be placed and compacted according to MTO Standards. An adequate surface erosion protection scheme should also be designed to preserve the surficial embankment slopes.

The fill heights (± 8.5 metres) and excavation requirements (± 2.0 metres) for the proposed realigned Acres Road, slope stability problems are anticipated. Based on the stability analyses, it is recommended that the approaches in the above mentioned area are to be constructed with a 2.0 m wide berm to the midheight of the slope. All slopes are to be built with 2H:1V (see detail below):



BERM DESIGN

Berms should be constructed as an integral part of the main embankment up to the berm height.

In the longitudinal direction if the toe berm at the original ground surface is eliminated, the effective height of the approach would be in the order 10.5 m. A mid height berm 4.0 m would be required in order to ensure the stability in this direction. No dewatering problems are anticipated in view of the fact that the groundwater level is below the excavation cut and the deposit is impervious in nature. The cut slopes should be protected against surface erosion. Topsoil and sodding is one recommended method of achieving this protection.

Earthquake forces were also incorporated in the stability analysis. A value of 0.2 g was used as the peak horizontal ground acceleration. Based on the results the proposed approach fills are considered to be statically and dynamically stable.

Settlement Considerations

Anticipated settlements are based on laboratory results obtained from Taylor's (1948) (Square Root Fitting Method) Consolidation testing procedures and employing Osterberg (1957) solution to determine the increase in vertical stress due to embankment loading. The total settlement anticipated as a result of elastic and consolidation settlement of the Silty Clay to Clayey Silt layer, Clayey Silt with interbedded Sandy Silt layer and settlement of the fill under its own weight is approximately 295 mm for embankment approaches 8.5 m in height. The elastic settlements were computed using Steinbrenner's (1934) method and comprise approximately 5-6 percent of the total settlement (15 mm).

These settlements will be immediate in nature and will be realized during construction of the embankments. In addition, in view of the slightly preconsolidated nature of both strata, a further 75 mm of settlement will be due to the recompression of the soil and consequently will occur during or immediately after construction. This results in net consolidation (time-dependent) settlements in the order of 120 mm.

Consolidation settlements curves for varying heights of fill are provided on Figure 9 in the Appendix. Estimates of the time rate settlement indicate that the total anticipated consolidation settlements should be realized within a period of 9 years after application of the embankment loading. It is also estimated that about 30 to 40 percent of the total anticipated settlement will be completed within a period of 6-9 months after construction of the embankments. In view of this, consideration should be given in constructing the approach fills as far in advance of the structure foundations as scheduling, feasibility and economics permit and also in delaying the final paving operations as long as possible.

Structure Foundations

Abutments and Pier Foundations for New Structure and Related Wing Walls

As discussed above, in view of the low shear strength and compressibility of the upper layers of soils, conventional spread footing shallow foundations are not applicable at this site. It is recommended that the abutments and pier foundations may be supported on end-bearing piles, equipped with reinforced tips in order to facilitate pile penetration through the basal glacial till stratum and driven to bedrock.

In consideration of the negative skin friction forces (additional downdrag forces), which will be induced as a result of the consolidation of the underlying cohesive deposits due to the imposed load of embankment, at the abutment locations the following design parameters are suggested, however, no negative skin friction forces are anticipated at pier locations:

<u>Structure</u>	<u>Pile Type</u>	Axial Capacity at S.L.S. Type II	Factored Axial Capacity at U.L.S.	Estimated Pile Tip
		(kN)	(kN)	El. (m)
W. Abutments	HP310x79	800	1050	41.8
	HP310x110	1050	1450	
Piers	HP310x79	900	1150	39.4
	HP310x110	1150	1600	
E. Abutments	HP310x79	800	1050	38.0
	HP310x110	1050	1450	

In view of the extreme denseness of the glacial till stratum located immediately above the bedrock, some piles may not penetrate this dense stratum. In such a case, the pile capacity should be controlled in the field using current MTO pile driving standards. However, attempts should be made in all cases to drive the piles to the bedrock surface.

Lateral strains induced by the settlement of the cohesive deposit shall be accounted for in the design of the abutments. To resist the potential lateral displacement and rotational forces caused by this settlement, it is recommended that the extreme ends of the wing walls should also be supported on end-bearing piles driven to bedrock using values as identified in the above table. Resistance to lateral load shall be computed in accordance with Section 6.8.3.8 of the O.H.B.D.C.

Pile caps may be perched within the embankment fill provided that particle sizes in the fill immediately beneath the pile locations does not exceed 75 mm. Alternatively, the pile caps may be founded within the surficial cohesive deposit. No dewatering problems are anticipated for this excavation.

Other Considerations

Lateral Earth Pressures on Structures

Free draining material such as Granular 'A' or Granular 'B' is recommended as

appropriate backfill to the abutments to prevent hydrostatic pressure build-up. Design parameters of the soil are given below:

	<u>Granular 'A'</u>	<u>Granular 'B'</u>
Angle of Internal Friction (ϕ)	35°	30°
Unit Weight (kN/m^3), γ	22.8	21.2
Coefficient of Active Earth Pressure (K_A)	0.27	0.33
Coefficient of Earth Pressure at Rest (K_0)	0.43	0.5

The earth pressure coefficient at rest is to be used in design if the abutment walls are rigid and unyielding. Weep holes in the abutment walls should be designed to drain any accumulation of water in the backfill.

Dewatering

No dewatering problems are anticipated in view of the fact that the groundwater level is below the excavation cut and the deposit is relatively impervious in nature. However, if surface water does accumulate in the excavations, it should be removed by means of a sump pump.

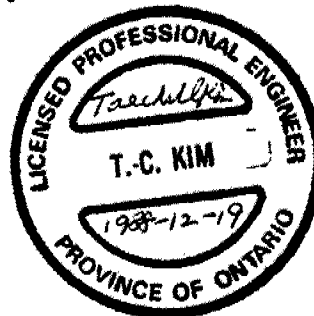
Frost Protection

The footings should be placed so as to have a minimum earth cover of 1.8 metres to allow for frost protection.

MISCELLANEOUS

The fieldwork for this investigation was carried out during the period of 88 07 05 to 88 07 14 under the supervision of Tae C. Kim, Foundation Engineer. The equipment was owned and operated by Marathon Drilling Co. Ltd., Ottawa and F.E. Johnston Drilling Co. Ltd., Ottawa.

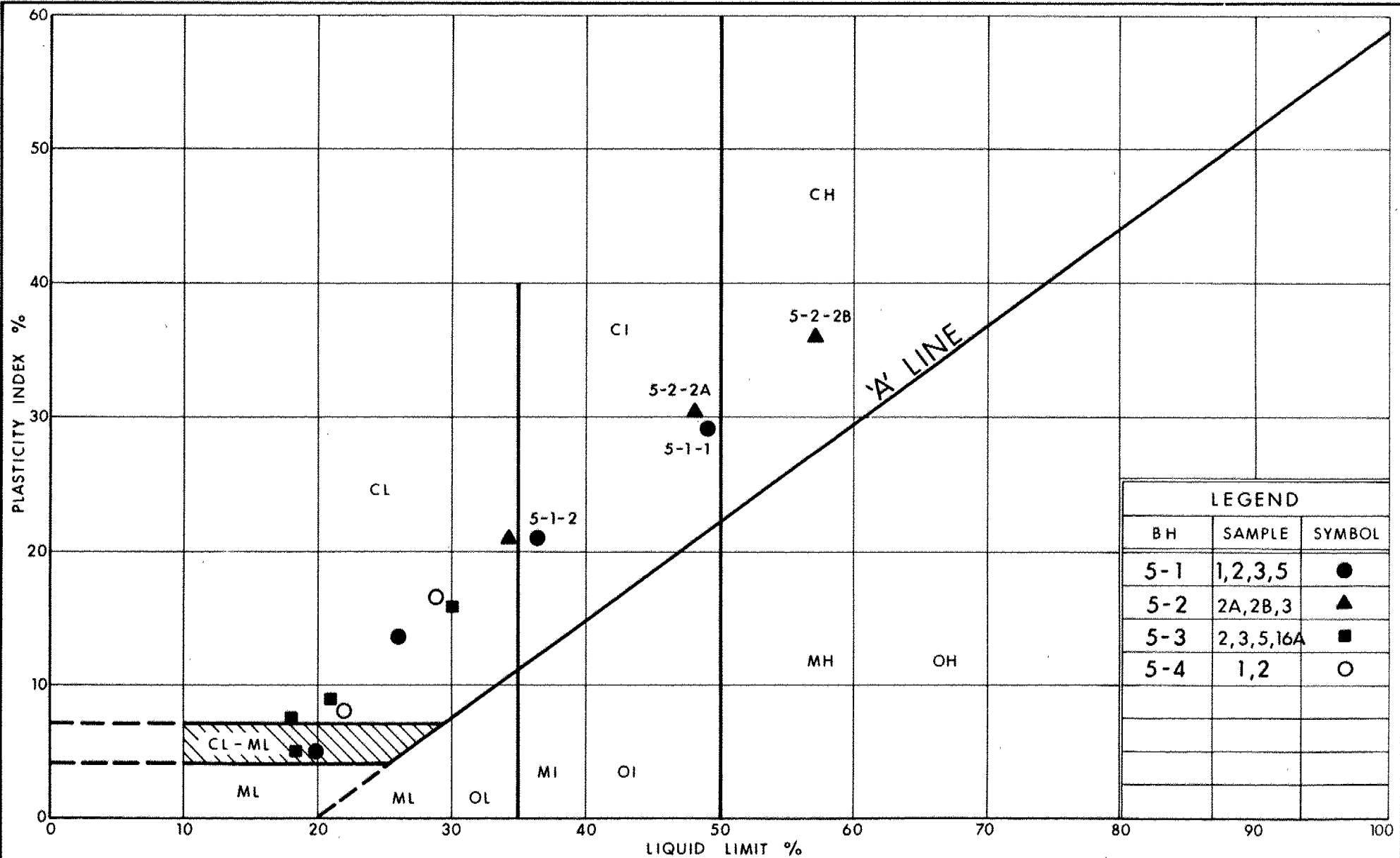
This report was written by Tae C. Kim, Foundation Engineer and reviewed by Murty Devata, Chief Foundation Engineer.



Tae C. Kim
Tae C. Kim, P.Eng.
Foundation Engineer

Murty Devata
Murty Devata, P.Eng.
Chief Foundation Engineer

APPENDIX



Ministry of
Transportation

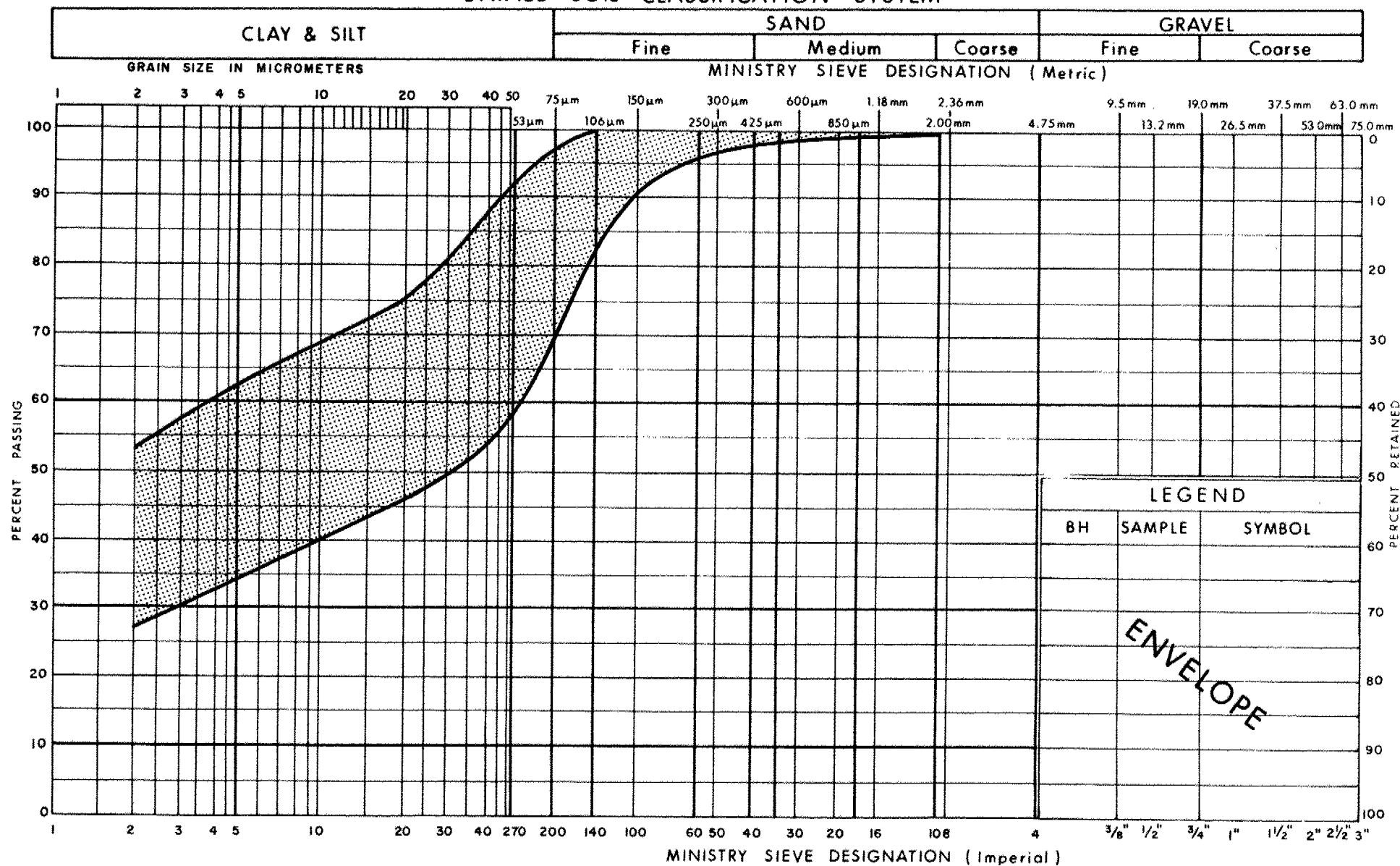
Ontario

PLASTICITY CHART SILTY CLAY TO CLAYEY SILT

FIG No 1

W P 120-87-08

UNIFIED SOIL CLASSIFICATION SYSTEM



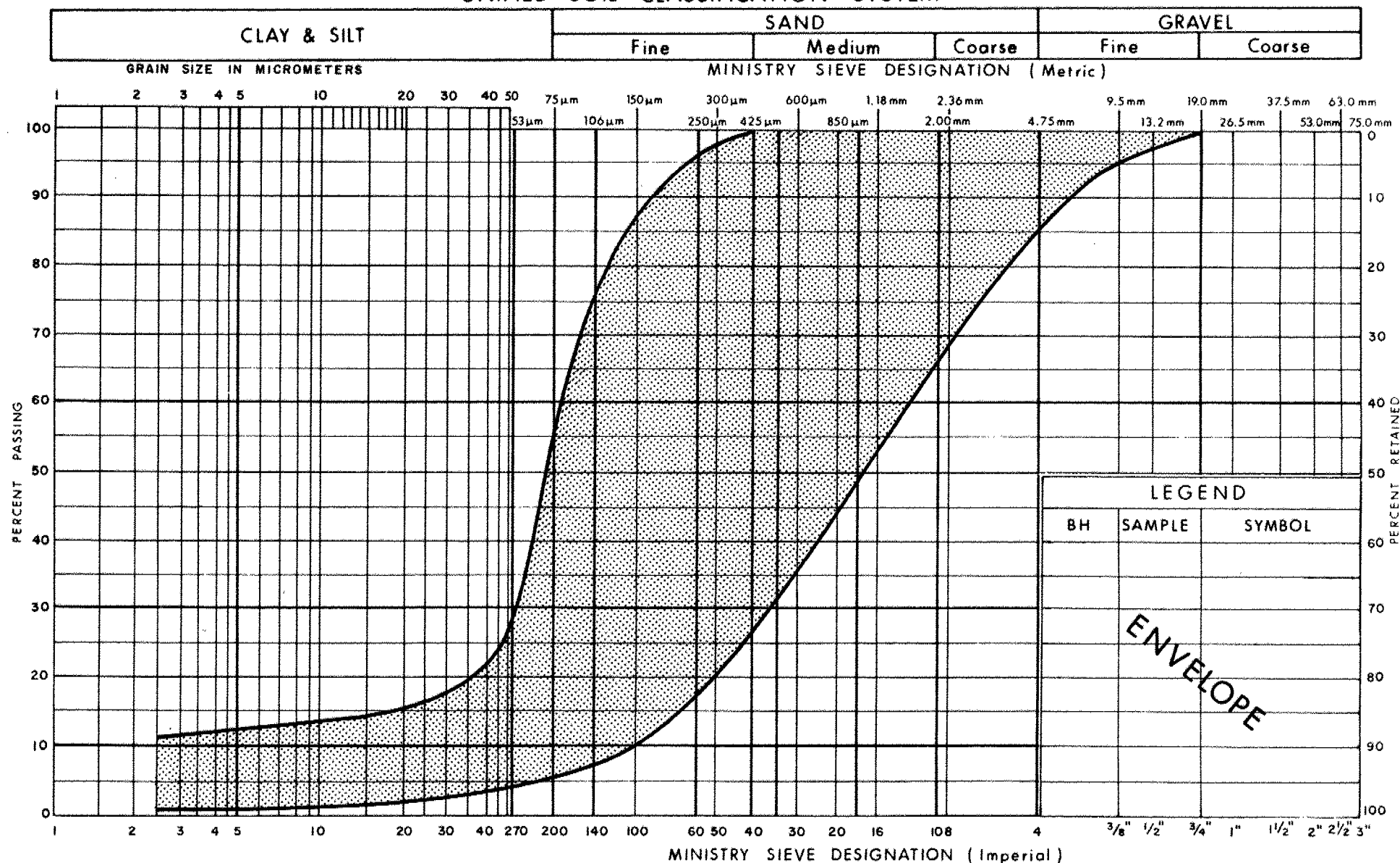
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAYEY SILT

FIG No 2

W P 120-87-08

UNIFIED SOIL CLASSIFICATION SYSTEM



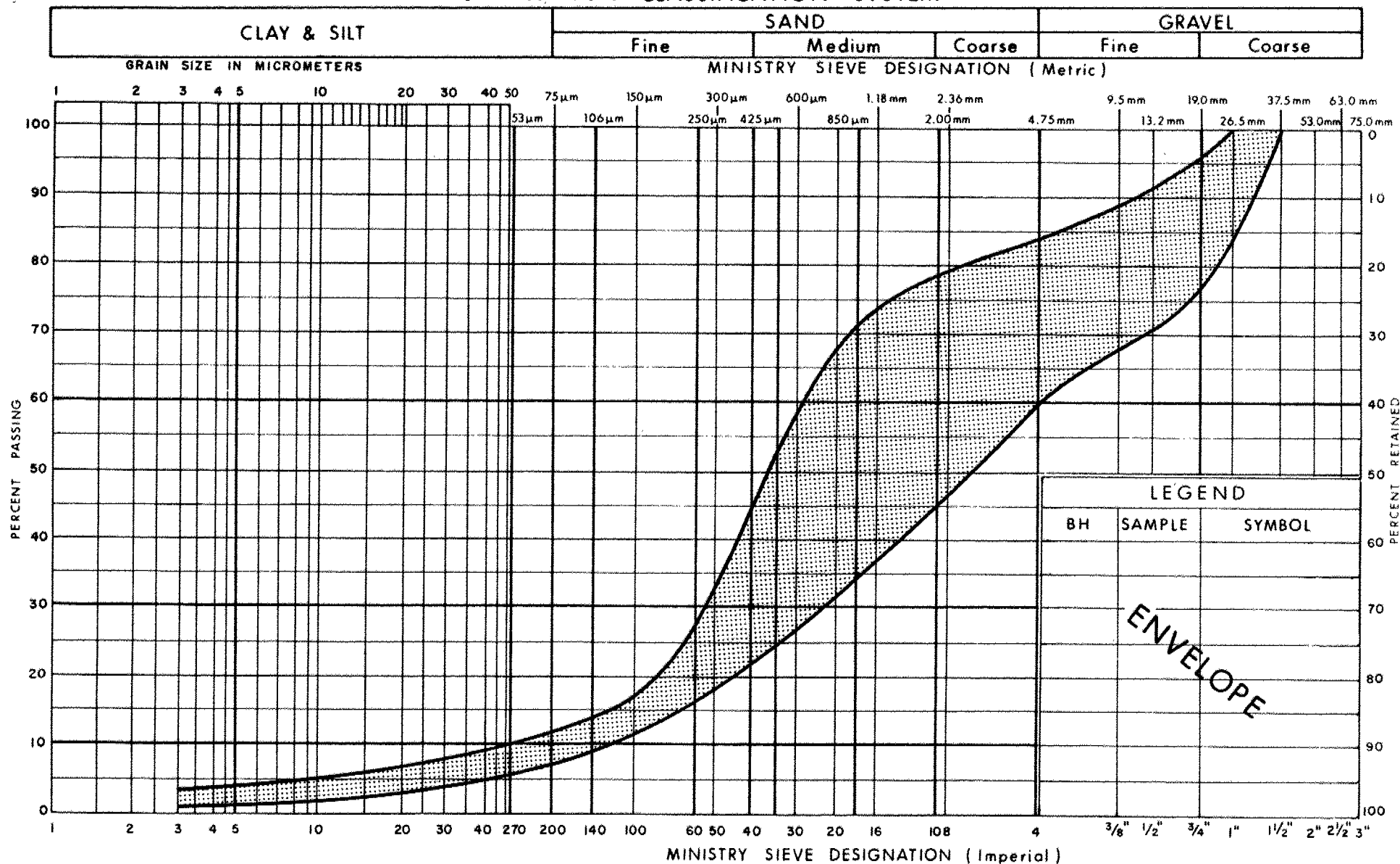
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY SAND TO SAND, TRACE GRAVEL

FIG No 3

W P 120-87-08

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
HET MIXTURE OF
SAND, GRAVEL & BOULDERS (Glacial Till)

FIG No 4

W P 120-87-08

VOID RATIO - PRESSURE CURVES

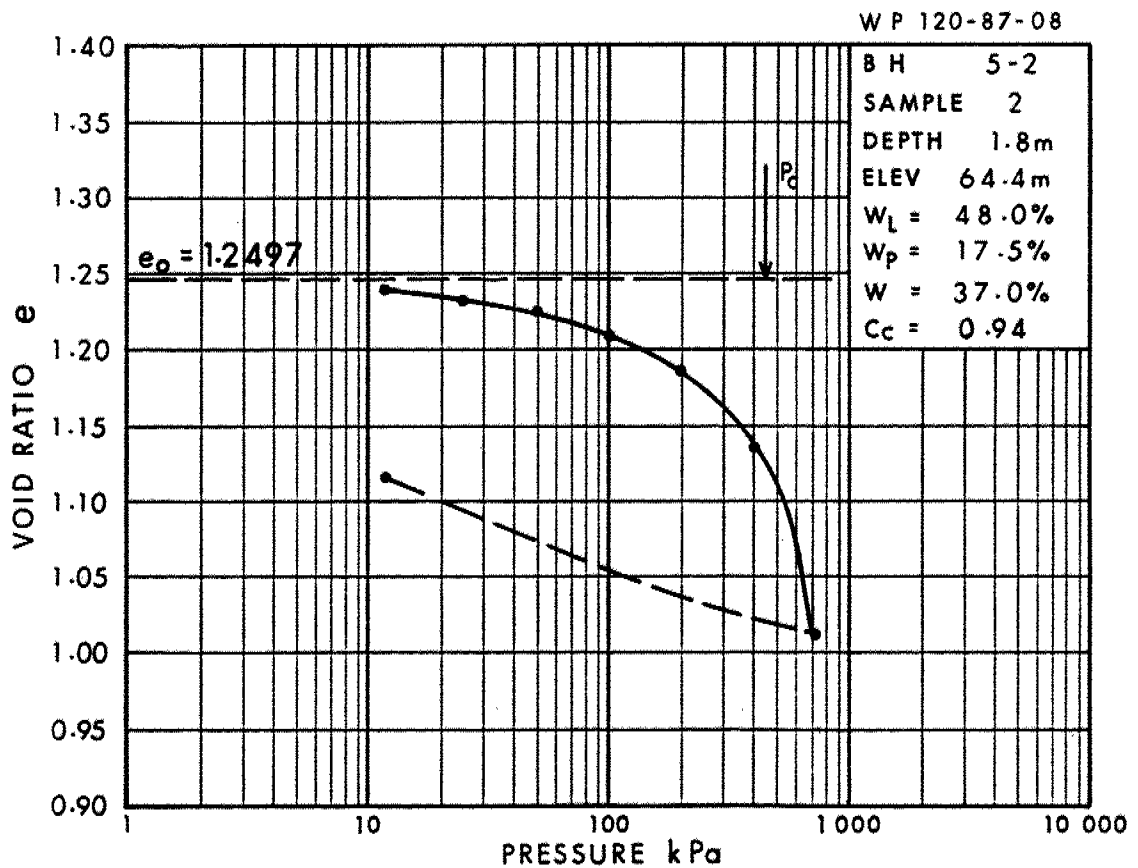
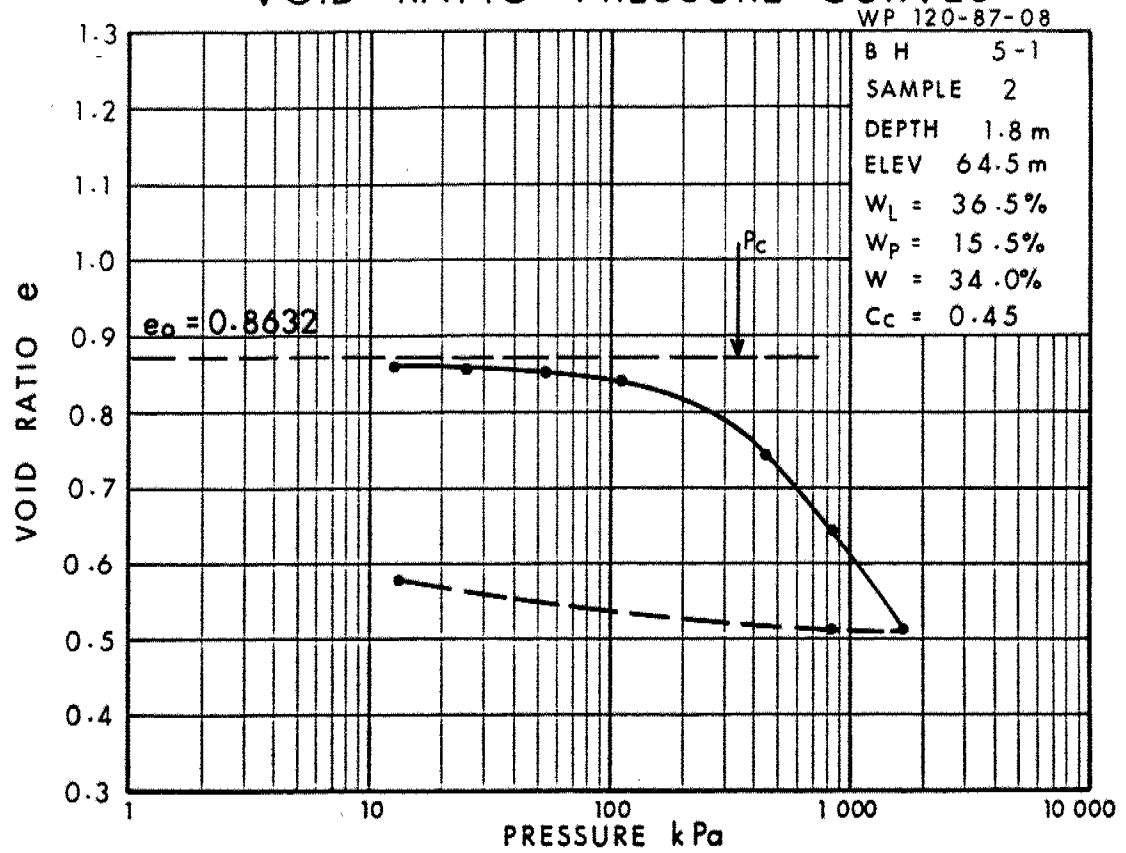


Fig 5

WP 120-87-08

VOID RATIO - PRESSURE CURVES

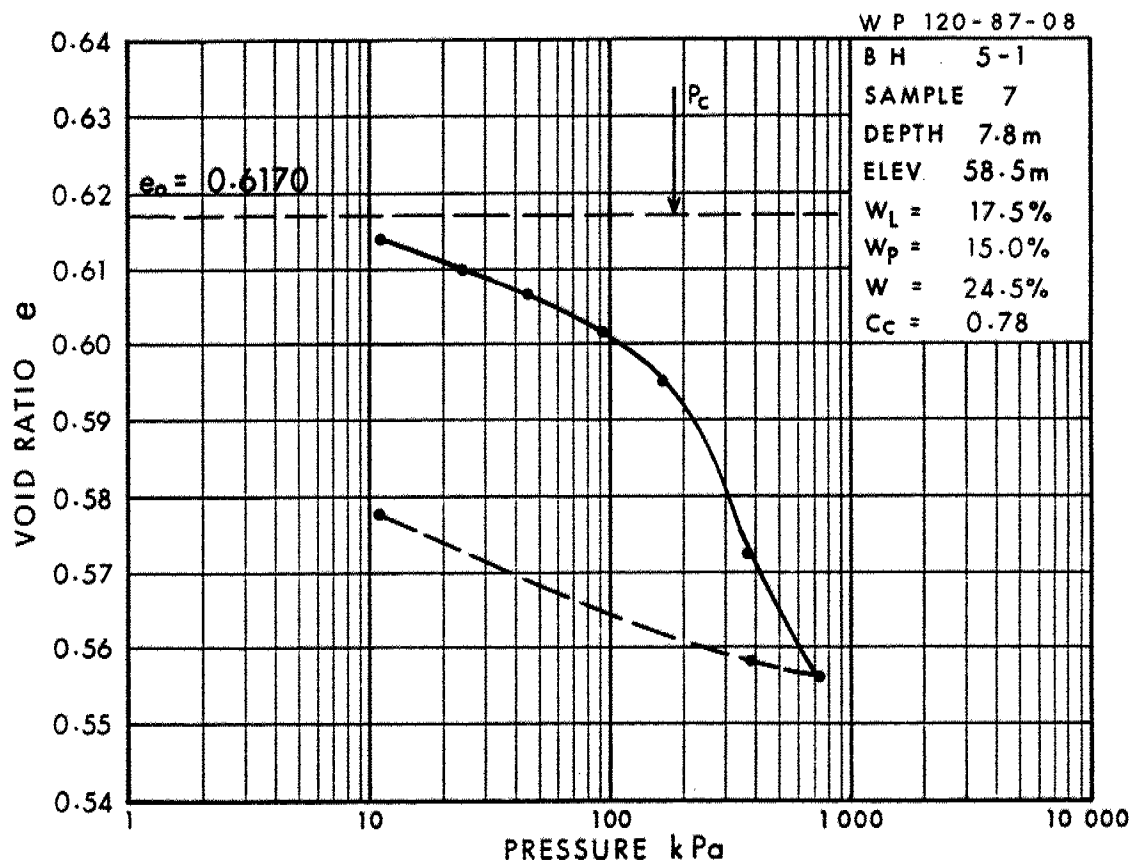
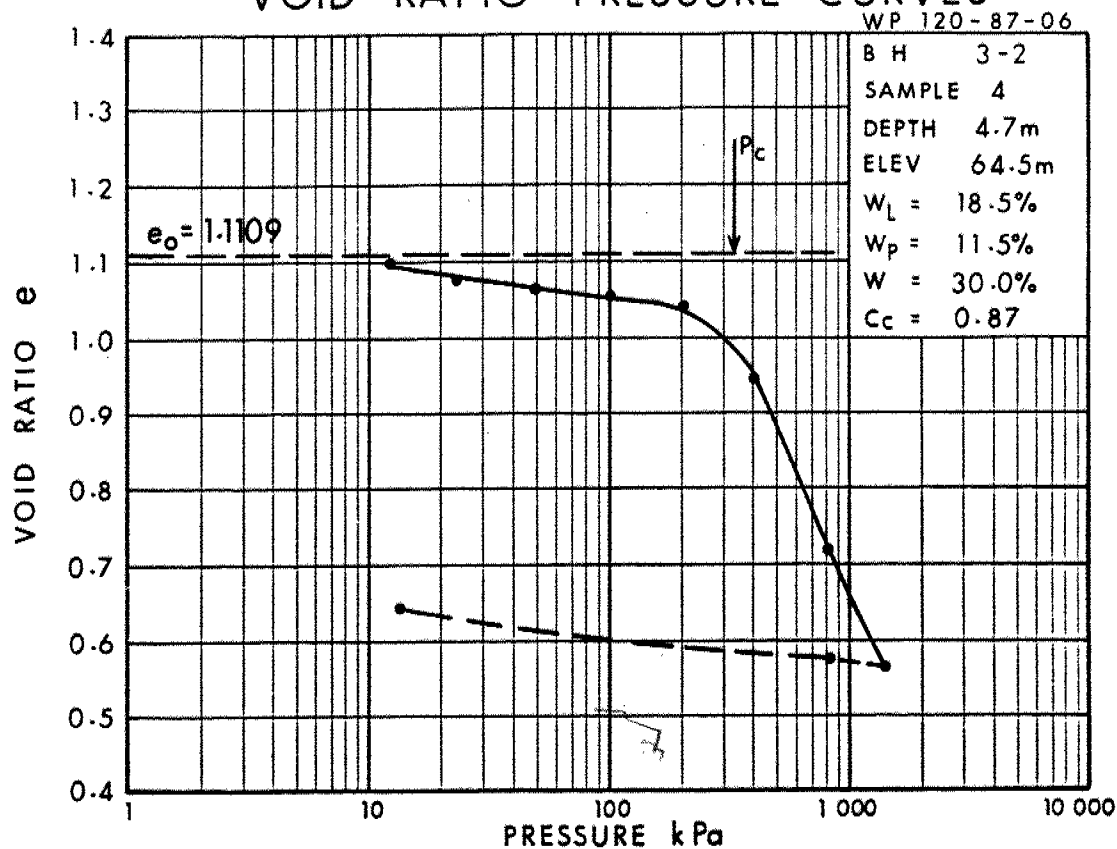
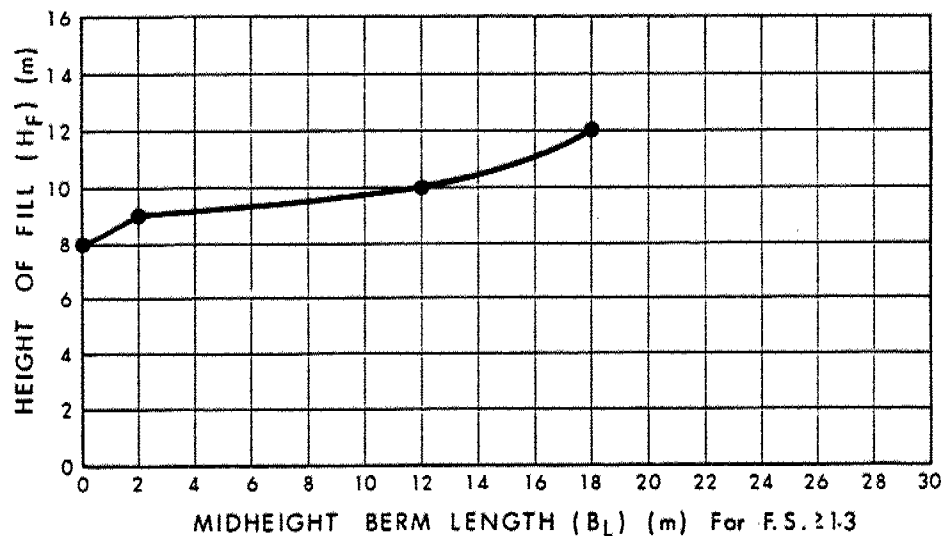
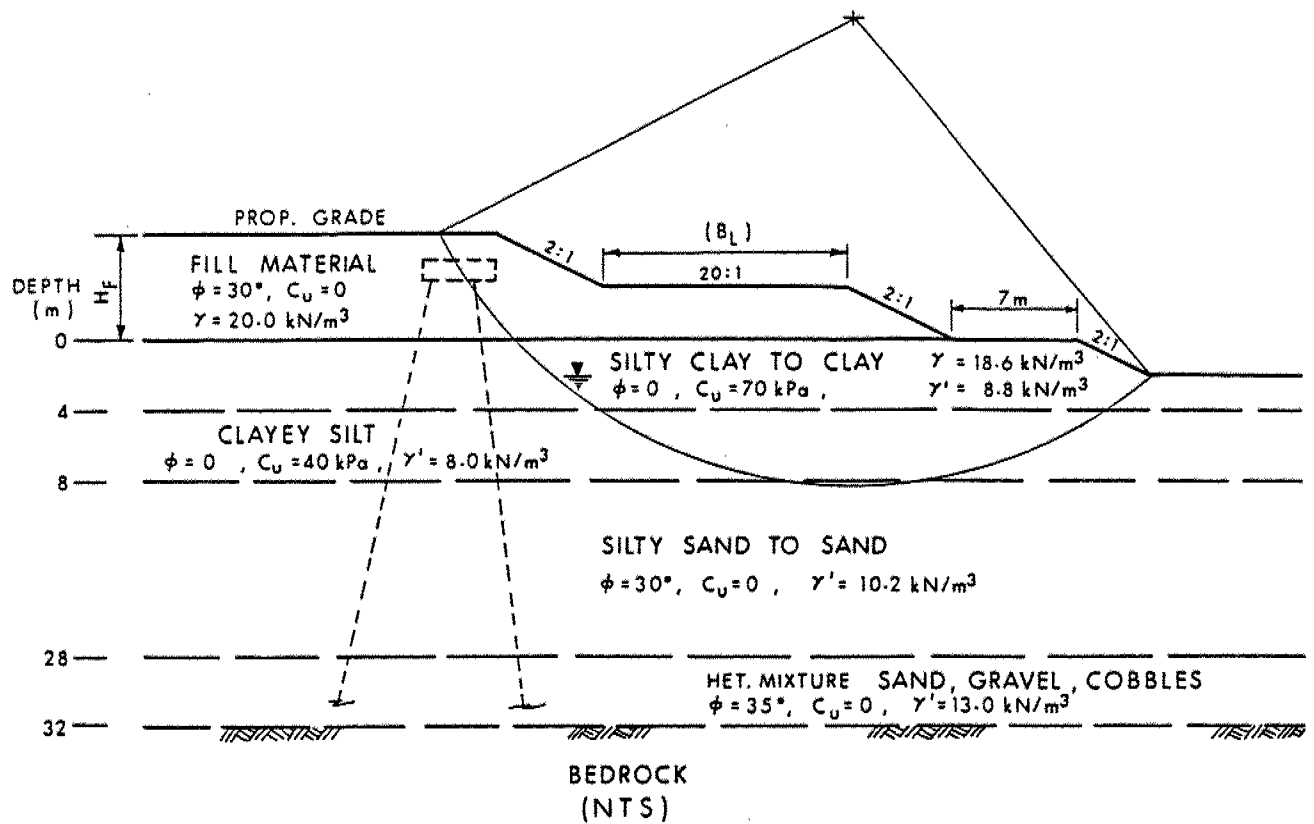


Fig 6

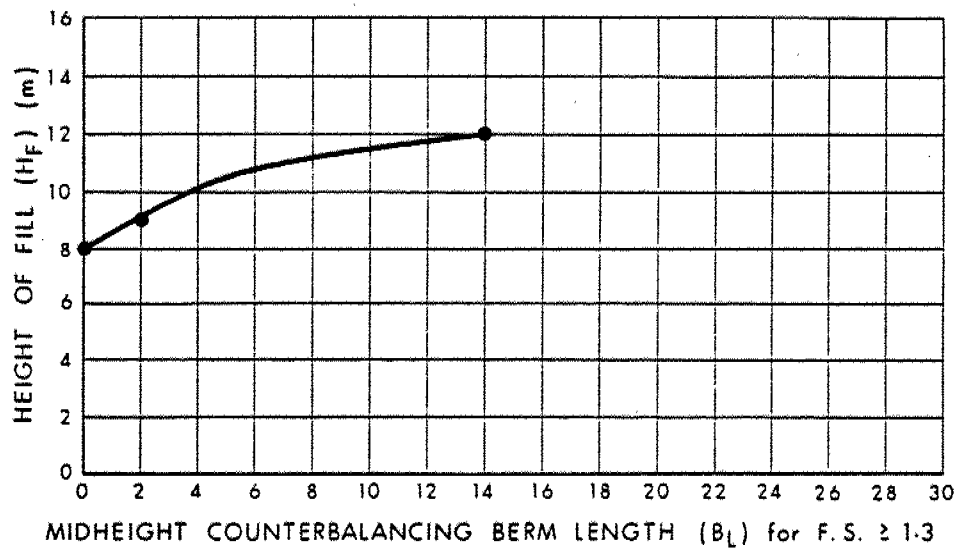
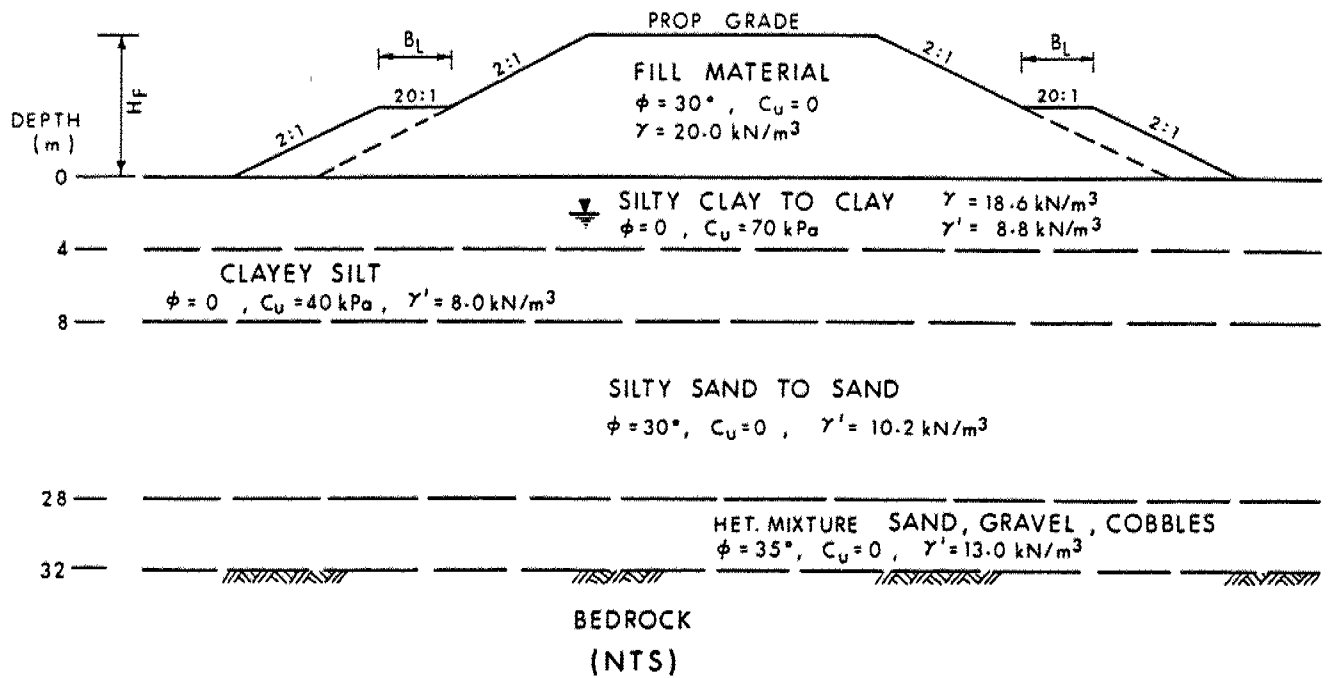
WP 120-87-08



EAST/WEST APPROACH EMBANKMENTS STABILITY ANALYSIS
LONGITUDINAL DIRECTION

WP 120-87-08

Fig 7

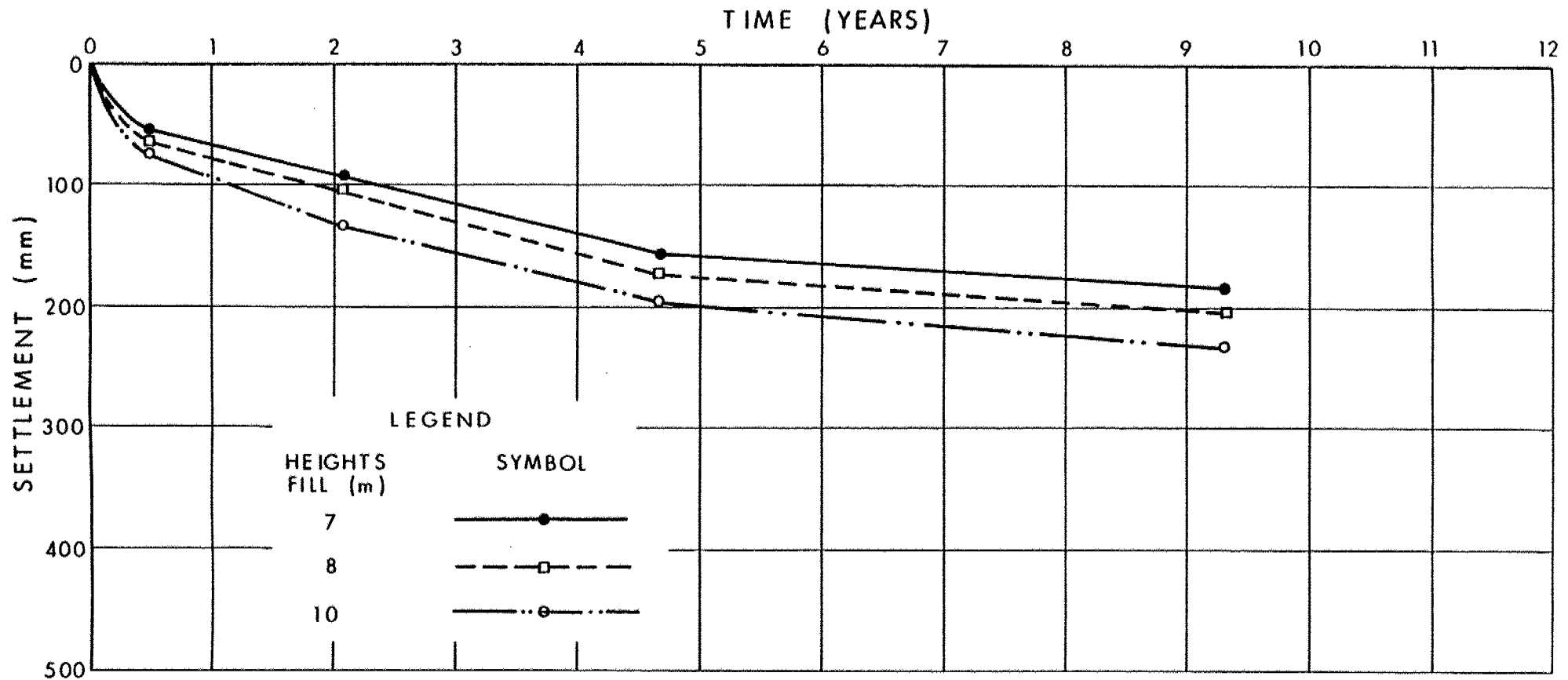


EAST / WEST APPROACH EMBANKMENTS STABILITY ANALYSIS TRANSVERSE DIRECTION

WP 120-87-08

Fig 8

Fig 9 - PREDICTED SETTLEMENT-TIME CURVES OF NATIVE SOILS UNDERLYING APPROACH EMBANKMENTS



LEGEND

HEIGHTS
FILL (m)

SYMBOL

7

—●—

8

- - □ - -

10

- · · ○ · · -

ASSUMPTIONS

- 1) Unit Weight of Fill = 20 kN/m^3
- 2) Osterberg Stress Distribution
- 3) Single Drainage
- 4) Coefficient of Consolidation (C_v)

(a) Silty Clay to Clay ($0.026 \text{ m}^2/\text{day}$)

(b) Clayey Silt with interbedded Sand ($0.040 \text{ m}^2/\text{day}$)

WP 120-87-08

Fig 9

DESCRIPTION OF ROCK CORE - WP 120-87-08

CORE RECOVERY				CORE DESCRIPTION	
HOLE #	DEPTH (m)	%CR*	%RQD*	DEPTH (m)	DESCRIPTION
5-2	24.38-24.49	63	-	24.38-26.21	SILTY DOLOSTONE , medium dark grey; fine grained, thinly bedded with minor argillaceous bands; medium strong rock; unweathered; intensely fractured zone from 25.30 - 25.75 m, and from 26.00 - 26.21 m.
	24.49-25.60	57	11		
	25.60-26.21	94	55		
5-3	25.73-26.01	50	-	25.73-26.63	OVERBURDEN , containing foreign and locally derived bedrock material up to 6 cm diameter.
	26.01-26.14	50	-	26.63-28.04	SILTY DOLOSTONE , medium dark grey; fine grained, thinly bedded with minor argillaceous bands and LIMESTONE beds; medium strong rock; unweathered; moderately close spaced fractures: flat, rough, open, slightly altered, clean.
	26.14-26.52	NOT	CORED		
	26.52-28.04	98	91		
5-4	23.47-23.77	63	-	23.47-27.25	OVERBURDEN , containing foreign and locally derived bedrock material up to 12 cm diameter.
	23.77-24.38	NOT	CORED	27.25-28.58	SILTY DOLOSTONE , medium dark grey; fine grained, thinly bedded; medium strong rock; unweathered; moderately spaced fractures: flat, rough, open, slightly altered, clean.
	24.38-25.43	10	-		
	25.43-25.73	83	-		
	25.73-26.87	33	-		
	26.87-27.20	0	0		
	27.20-28.58	96	43		

NOTE: Depths are approximated in zones of poor core recovery.

*CR = CORE RECOVERY

*RQD = ROCK QUALITY DESIGNATION

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 5-1

METRIC

W P 120-87-08 LOCATION Co-Ords N 5 022 651.6; E 358 543.0 ORIGINATED BY JF
 DIST 9 HWY 417 BOREHOLE TYPE Hollowstem Auger, Cone Test COMPILED BY JF
 DATUM Geodetic DATE 88 07 08 CHECKED BY TCK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
66.3	Ground Level													
0.0	Silty Clay to Clayey		1	SS	12		66						49	0 2 52 46
	Silt, some to trace sand	Brown Grey	2	TW	PH		64		126				18.6	0 12 58 30
63.8	Stiff													
2.5	Clayey Silt with interbedded Sandy Silt		3	SS	1		62		3.6					0 31 40 29
	Occ. Silt Layers		4	SS	2		60		3.3					0 26 42 32
	Very soft to soft		5	SS	1		58							
			6	SS	1		56							
			7	TW	PH		54						19.3	0 29 50 21
57.7							52							
8.6	Silty Sand to Sand		8	SS	16									
	Trace of Gravel													
			9	SS	24									1 84 (15)
	Compact													
50.6			10	SS	*									
15.7	END OF BOREHOLE													
	* Not representative due to Sand blow back in Augers													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 5-2

METRIC

W P 120-87-08 LOCATION Co-Ords N 5 022 676.9; E 358 562.9 ORIGINATED BY JF
 DIST 9 HWY 417 BOREHOLE TYPE H-S Auger, Washboring, Rock Coring & Cone Test COMPILED BY JF
 DATUM Geodetic DATE 88 07 05 - 14 CHECKED BY JCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
66.2	Ground Level													
0.0	Silty Clay to Clayey Silt Occ. Clay Layers Brown Grey		1	SS	9		66							
63.7	Stiff		2	TW	PH		64		3.6				18.2	0 5 46 49
2.5	Clayey Silt with interbedded Sandy silt Occ. Silt Layers Very soft to soft		3	TW	PH		62		3.1				17.6	0 2 45 53
			4	SS	1		60							0 32 39 29
			5	SS	2		58							0 31 44 25
			6	SS	1		56							
			7	SS	2		54							
			8	SS	1		52							
			9	SS	2		50							
56.1			10	SS	29		48							
10.1			11	SS	32		46							
	Silty Sand to Sand trace to some Gravel		12	SS	43		44							23 59 9 9
			13	SS	44		42							
	Compact to Dense		17	SS	28		40							
44.9			18	RC	REC		38							
21.3	Het. Mixture of Sand, Gravel and Boulders Compact to very Dense (Glacial Till)		19	RC	REC		36							30 62 (8)
41.8			20	RC	REC		34							
24.4	Bedrock					63%	32							
	Silty Dolostone						30							RQD = 11%
40.0							28							RQD = 55%
26.2	END OF BOREHOLE						26							

+3, x5: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 5-3

METRIC

W P 120-87-08 LOCATION Co-Ords N 5 022 693.9; E 358 607.4
 DIST 9 HWY 417 BOREHOLE TYPE H-S Auger, Washboring, Rock Coring & Cone Test
 DATUM Geodetic DATE 88.07.05 - 07
 ORIGINATED BY TK
 COMPILED BY TK
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
66.0	Ground Level													
0.0	Silty Clay to Clayey Silt Some Sand Trace Gravel Firm	Brown Grey	1	SS	6								19.3	8 17 46 29
63.5			2	TW	PH									
2.5	Clayey Silt with interbedded Sandy Silt		3	TW	PH									
			4	SS	1									
			5	SS	1									
			6	SS	1									
	Very Soft													
58.1			7	SS	30									
7.9			8	SS	15									
	Silty Sand to Sand		9	SS	18									
	Trace of Gravel		10	SS	4									
	Occ. Gravelly Sand Layers		12	SS	9									
	Occ. Clayey Silt Seams		13	SS	70									
	Loose to very Dense		14	SS	45									
			15	SS	59									
	Clayey Silt		16	SS	73									
			17	SS	51									
43.7			18	SS	83									
22.3	Het. Mixture of Sand, Gravel and Boulders Very Dense (Glacial Till)		19	SS	67									
			20	RC	REC 50%									
39.4			22	SS	*									
26.6	Bedrock													
	Silty Dolostone		23	RC	REC 98%									
38.0														
28.0	END OF BOREHOLE * Not Representative													

RECORD OF BOREHOLE No 5-4

METRIC

W P 120-87-08 LOCATION Co-Ords N 5 022 719.6; E 358 627.5 ORIGINATED BY TK
 DIST 9 HWY 417 BOREHOLE TYPE R-S Auger, Washboring, Rock Coring & Cone Test COMPILED BY TK
 DATUM Geodetic DATE 88 07 05 - 07 CHECKED BY TCK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	Wp	W	W _L	WATER CONTENT (%)		
65.3	Ground Level													
0.0	Sandy Silt (Fill)													
64.5														
0.8	Clayey Silt with Interbedded Sandy Silt Soft		1	SS	2									0 25 46 29
62.8			2	TW	PH									
2.5			3	SS	22									
	Silty Sand to Sand		4	SS	14									14 80 (6)
	Some Gravel Occ.Gravelly Sand Layers	Grey Brown	5	SS	14									
			6	SS	25									43 49 6 2
			7	SS	18									
	Compact to Dense	Brown Grey	8	SS	14									12 79 (9)
			9	SS	29									
			10	SS	36									
			11	SS	33									
			12	SS	40									
43.7			13	SS	38									
21.6	Het. Mixture of Sand, Gravel and Boulders Dense to Very Dense (Glacial Till)		14	RC	REC	63%								41 48 9 2
			15	SS	49									
			16	RC	REC	83%								
			17	RC	REC	33%								
38.0			18	SS	37	10cm								
27.3	Bedrock		19	RC	REC	96%								
36.7	Silty Dolostone													
28.6	END OF BOREHOLE													ROD = 43%

OFFICE REPORT ON SOIL EXPLORATION

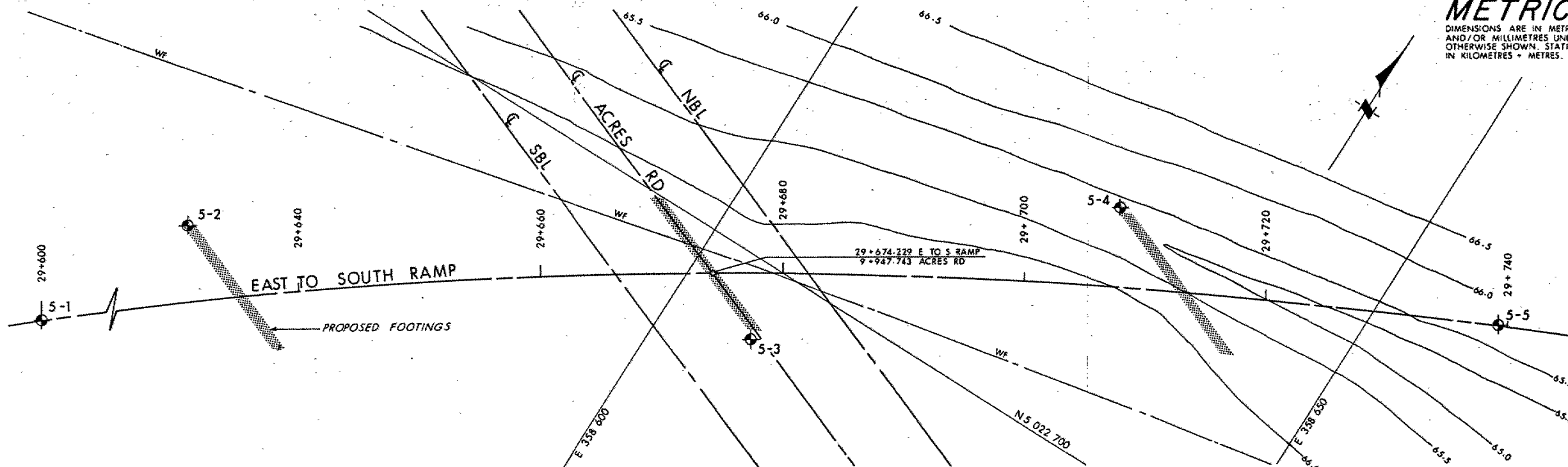
RECORD OF BOREHOLE No 5-5

METRIC

W P 120-87-08 LOCATION Co-Ords N 5 022 728.2; E 358 659.4 ORIGINATED BY TK
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger, Wash Boring, Cone Test COMPILED BY TK
 DATUM Geodetic DATE 88 07 05 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	Wp	W	WL		
65.8	Ground Level												
0.0	Clayey Silt to Silt (Fill)		1	TW	PH								
64.5			2	SS	18								
1.3			3	SS	5								
			4	SS	5								
			5	SS	8								
	Silty Sand to Sand		6	SS	11								
			7	SS	8								
	Trace to some Gravel												
			8	WS	4								
	Loose to Brown Grey		9	SS	17								
	Very Dense		10	SS	38								
			11	SS	32								
			12	SS	52								
50.1			13	SS	41								
15.7	END OF BOREHOLE												
47.5													
18.3	END OF CONE TEST												



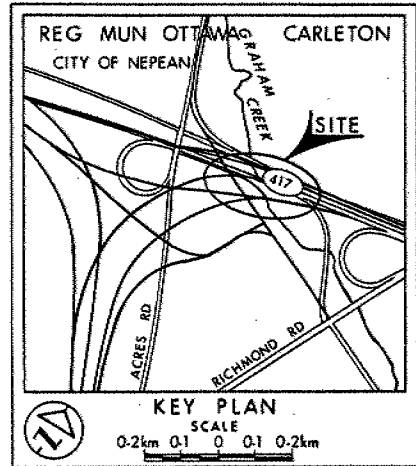
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 120-87-08

E-S RAMP OVER ACRES RD
(STRUCTURE -5)
BORE HOLE LOCATIONS & SOIL STRATA

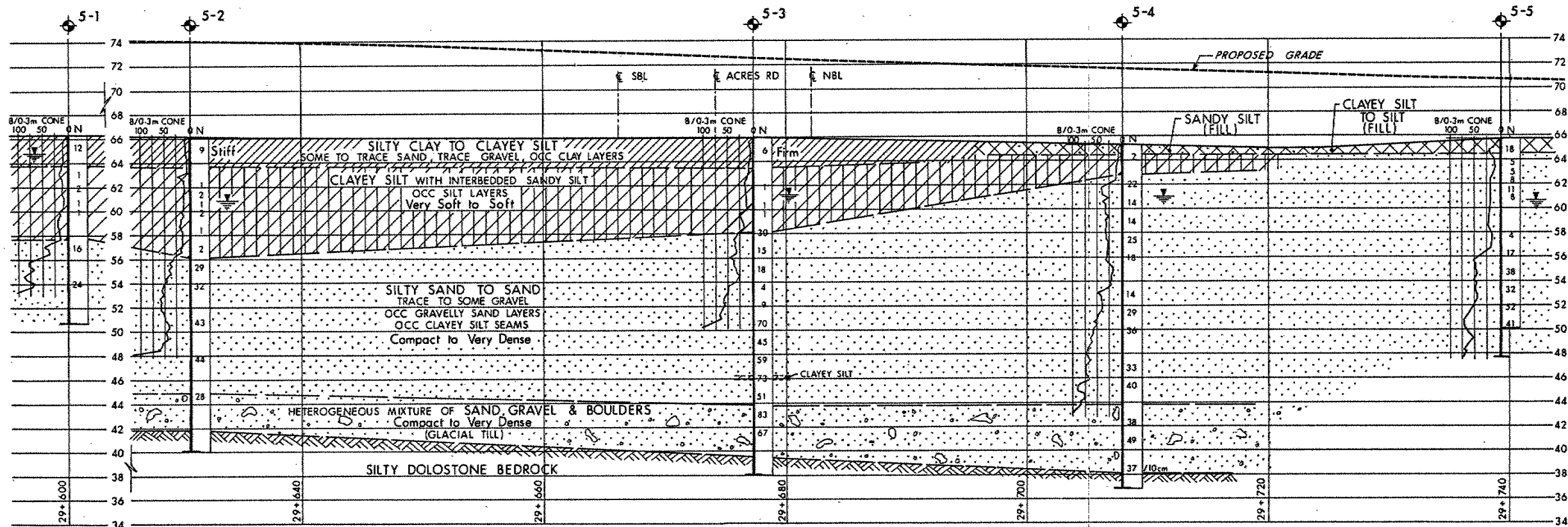


SHEET



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (5rd Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation 88 07



PROFILE EAST TO SOUTH RAMP

SCALE
4m 2 0 4m

No	ELEVATION	CO-ORDINATES NORTH	EAST
5-1	66.3	5 022 651.6	358 543.0
5-2	66.2	5 022 676.9	358 562.9
5-3	66.0	5 022 693.9	358 607.4
5-4	65.3	5 022 719.6	358 627.5
5-5	65.8	5 022 728.2	358 659.4

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.

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Geocres No 31G5-152		
HWY No 417		DIST 9
SUBMITTAL CHECKED	DATE 88 11 24	SITE 3-535
DRAWN BY	CHECKED	APPROVED
		DWG 1208708-A