DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31F-105

DIST. 9 REGION

W.P. No. 87-80-04

CONT. No. 86-15

W. O. No.

STR. SITE No. 29-33

HWY. No. Reg. Rd. 19

LOCATION Muskrat River Bridge (Mud Lake Rd.)

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

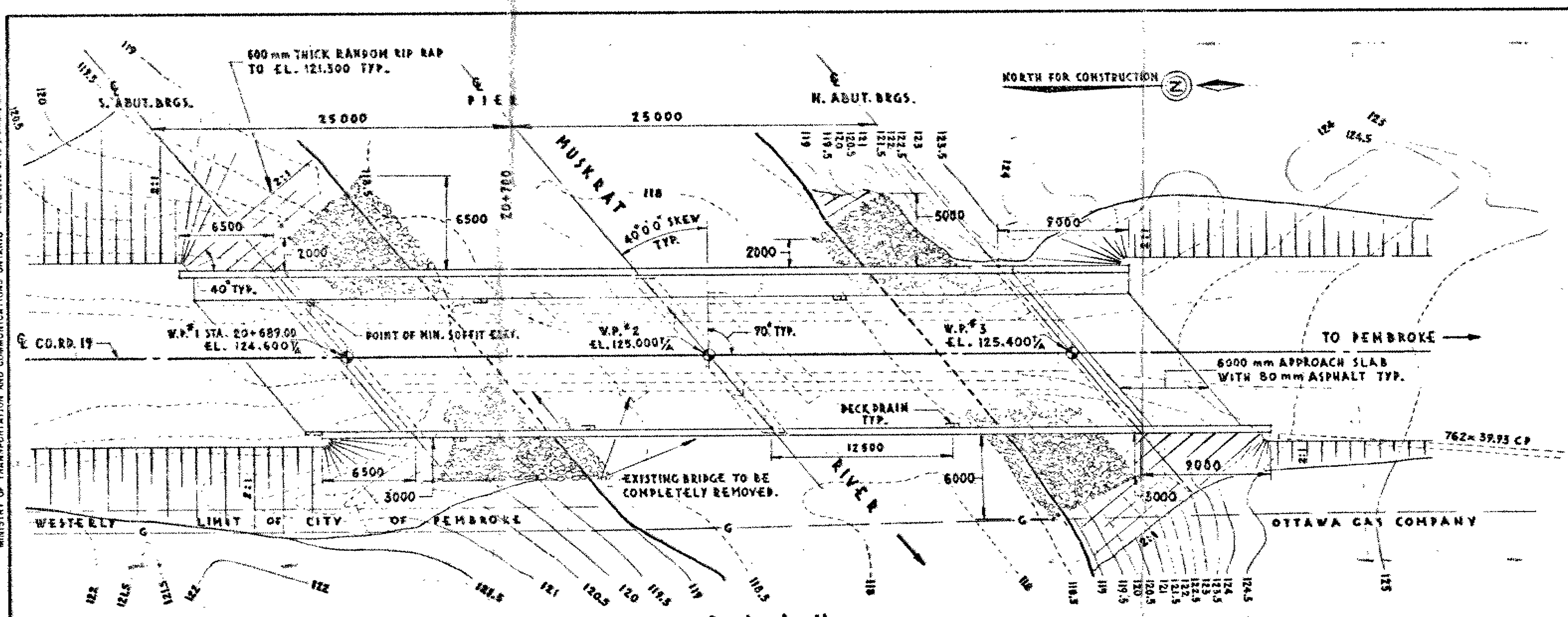
REMARKS:

G.I.-30 SEPT. 1976

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

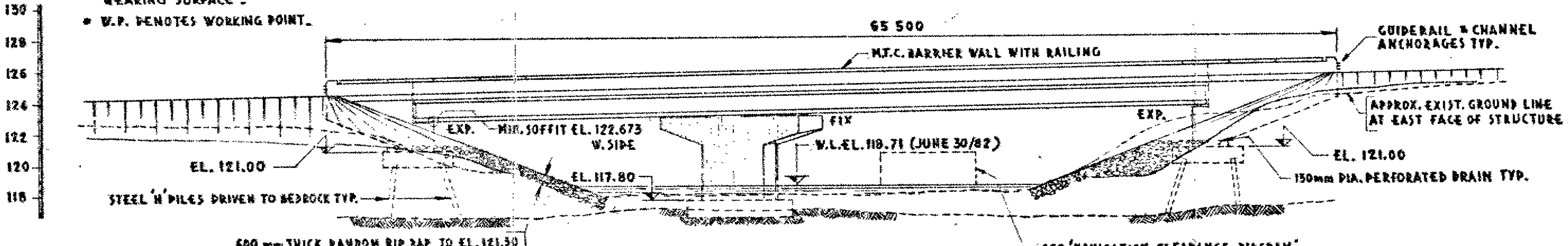
DISTR. 9	
CONT No	
WP No 87-80-04	
MUSKRAT RIVER BRIDGE CO. RD. No 19 GENERAL ARRANGEMENT	SHEET



NOTE
- ⁄A DENOTES TOP OF ASPHALT WEARING SURFACE.
- W.P. DENOTES WORKING POINT.

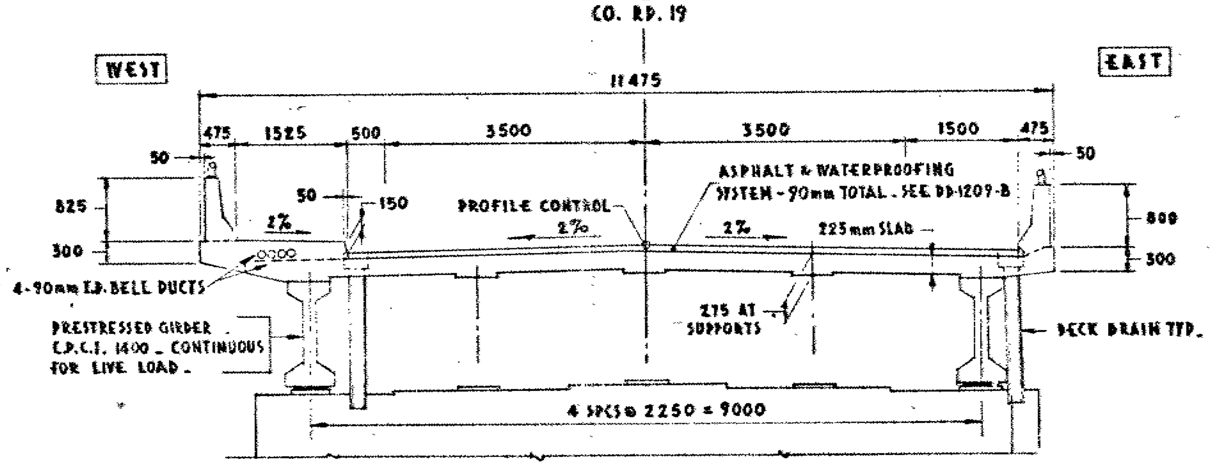
PLAN

SCALE 1:200



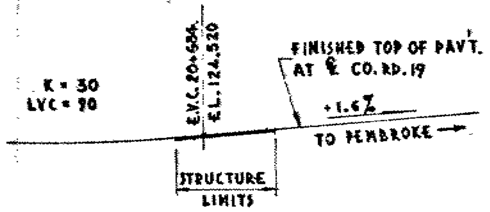
ELEVATION

SCALE 1:200



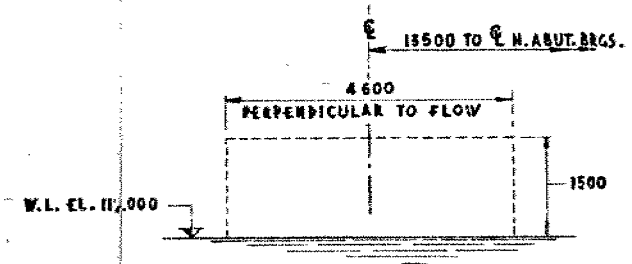
TYP. DECK SECT.

SCALE 1:50



PROFILE OF CO. RD. 19

N.T.S.



NAVIGATION CLEARANCE DIAGRAM

N.T.S.

GENERAL NOTES

CLASS OF CONCRETE

- PRESTRESSED GIRDERS .... 40 MPa
- FOOTINGS & APPROACH SLABS .... 20 MPa
- REMAINDER .... 30 MPa

REINFORCING STEEL

- GRADE 400
- BARS MARKED WITH SUFFIX "C" DENOTE COATED BARS.

CLEAR COVER TO REINFORCING STEEL (mm)

- FOOTINGS .... 100±25
- ABUTMENTS & WINGWALLS :
  - FRONT FACE .... 80±20
  - BACK FACE .... 70±20
- PIER .... 80±20
- DECK :
  - TOP .... 70±20
  - BOTTOM .... 40±10
- REMAINDER UNLESS OTHERWISE NOTED .... 70±20

CONSTRUCTION NOTE

- THE CONTRACTOR SHALL FINISH THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS TO A TOLERANCE OF ± 3 mm.

LIST OF DRAWINGS

29-33-1 GENERAL ARRANGEMENT
-2 BOREHOLE LOCATIONS & SOIL STRATA.
-3 FOOTING DETAILS
-4 SOUTH ABUTMENT
-5 NORTH ABUTMENT
-6 PIER DETAILS
-7 PRESTRESSED GIRDERS & BEARINGS
-8 DECK DETAILS I
-9 DECK DETAILS II
-10 BARRIER WALL ON SIDEWALK
-11 BARRIER WALL WITH RAILING
-12 RAILING FOR BARRIER WALL
-13 6000 mm APPROACH SLAB
-14 JOINT ANCHORAGE AND ARMOURING
-15 JOINT ANCHORAGE AND ARMOURING
-16 STANDARD DETAILS
-17 BRIDGE DATE & SITE NUMBER DATA
-18 AS CONSTRUCTED ELEV & DIM
-19 QUANTITIES - STRUCTURE I
29-33-20 QUANTITIES - STRUCTURE II

B.M. 125.752

SOUTH END OF CONC. WINDOW SILL 21.8 RT 20+767.2

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REGISTERED PROFESSIONAL ENGINEER
R. J. TAYLOR
PROVINCE OF ONTARIO

REVISIONS		
DATE	BY	DESCRIPTION
DESIGN	CHECK	LOADING OHBDC-A-79	DATE
DRAWING C. C.	CHECK	SITE No 29-33	DWG 1

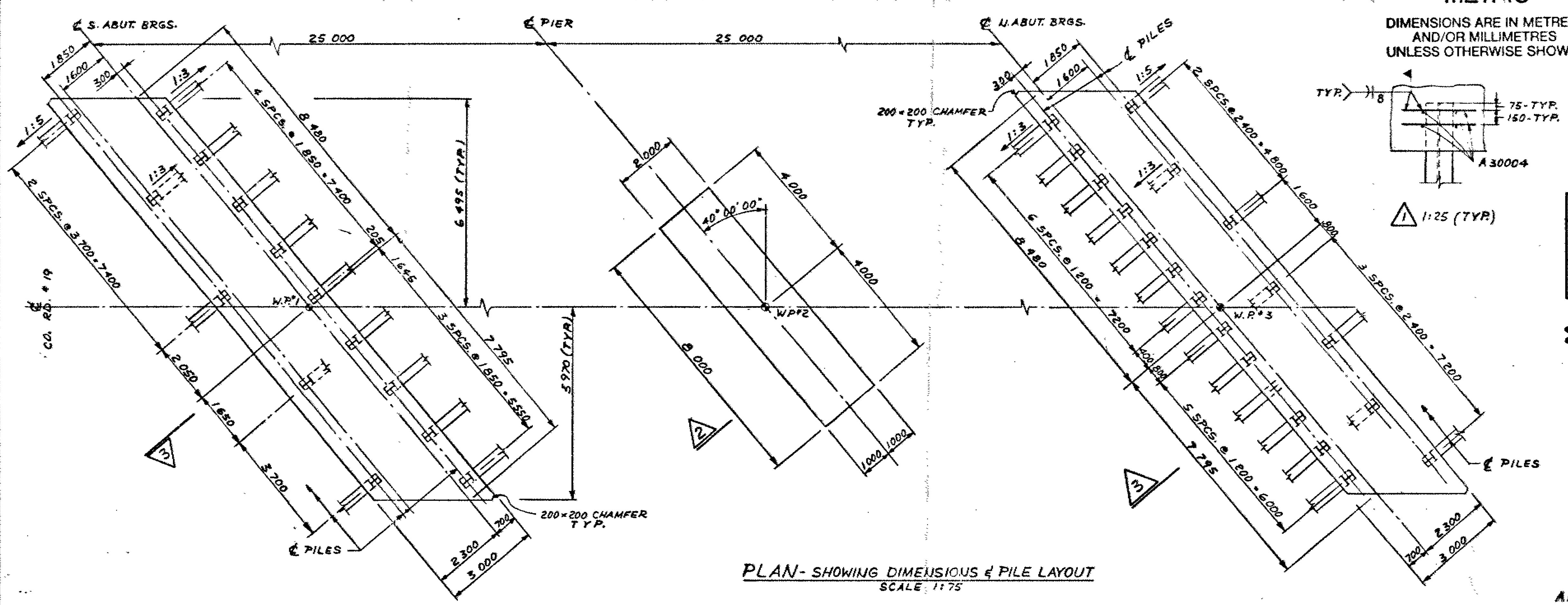
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No ·
WP No 87-80-04

MUSKRAT RIVER BRIDGE
CO. RD. Nº 19
FOOTING DETAILS

SHEET



PLAN - SHOWING DIMENSIONS & PILE LAYOUT
SCALE 1:75

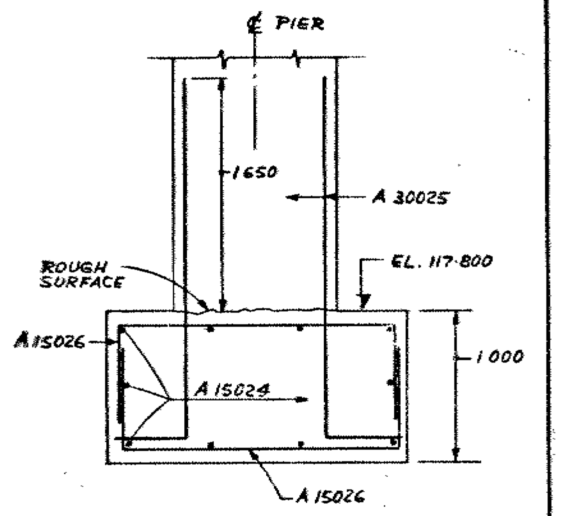
1:25 (TYP.)

PILE DATA

LOCATION	Nº	BATTER	LENGTH	Nº OSLO POINTS REQ'D
S. ABUT.	3	1:5	4 000	—
	11	1:3		11
N. ABUT.	16	1:3		16
	4	1:5	3 750	—

PILE DESIGN DATA

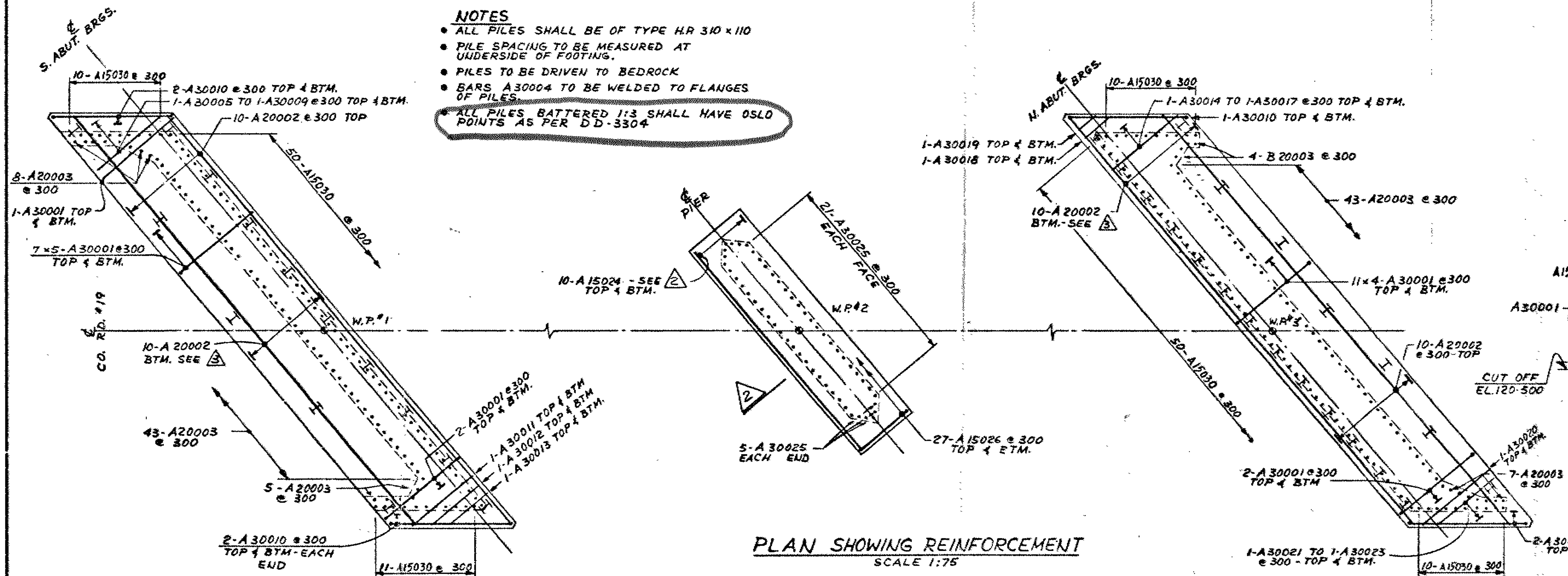
- DESIGN LOAD AT S.L.S. TYPE II = 1150 kN
- FACTORED CAPACITY AT U.L.S. = 1600 kN



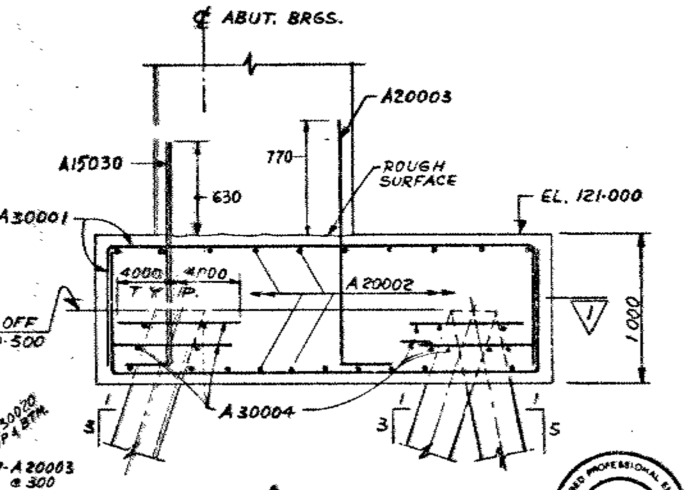
2 1:25

NOTES

- ALL PILES SHALL BE OF TYPE H.P. 310 x 110
- PILE SPACING TO BE MEASURED AT UNDERSIDE OF FOOTING.
- PILES TO BE DRIVEN TO BEDROCK
- BARS A30004 TO BE WELDED TO FLANGES OF PILES.
- ALL PILES BATTERED 1:3 SHALL HAVE OSLO POINTS AS PER DD-3304



PLAN SHOWING REINFORCEMENT
SCALE 1:75



3 1:25

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS DATE	BY	DESCRIPTION

DESIGN R.T.	CHECK	LOADING O.H.B.D.C.-A-83	DATE 84 SEP
DRAWING M.M.	CHECK	SITE No 29-33	DWG 3

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO

MINISTRY OF TRANSPORTATION AND
COMMUNICATIONS

GEOTECHNICAL INVESTIGATION
PROPOSED BRIDGE REPLACEMENT
OVER MUSKRAT RIVER
BRIDGE #29-33
W.P. 87-80-04

PEMBROKE ONTARIO

Distribution:

10 copies - Ministry of Transportation and
Communications
Downsview, Ontario

2 copies - Golder Associates
Mississauga, Ontario

July, 1983 831-1150

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APPENDIX A - FIELD WORK PROCEDURES

EXPLANATION OF TERMS USED IN REPORT

RECORD OF BOREHOLE SHEETS

DRAWING NO. 878004-A

FIGURES 1 TO 5, INCLUSIVE

ABSTRACT

A subsurface investigation was carried out by Golder Associates for the Ministry of Transportation and Communications at the site of a proposed two lane bridge to replace the existing bailey bridge (Bridge No. 29-33) on County Road 19 at Muskrat River in the Township of Pembroke, Ontario.

The borings indicate that, at the crest of the river banks, the site is underlain by a surficial layer of fill which overlies a deposit of grey/brown to grey silty clay. This stratum is up to 4.3 m thick at the crest of the river banks. The silty clay has a consistency which ranges from very stiff to soft with depth. Underlying the silty clay, 1.7 to 2.7 m of a till material was encountered and the till ranges in composition from a clayey silty sand with some gravel to sand with some silt and gravel and a trace of clay. The river bed materials encountered consisted of silty sand and gravel, sand till, and sand and gravel for a total thickness of about 1.1 m. Limestone bedrock was encountered at elevations of between 116.8 and 117.2 m. The groundwater level across the site ranged from elevation 118.7 to 119.2 m at the time of the investigation. The river water level at this time was at elevation 118.7 m.

The central pier of the proposed structure and the abutments may be supported on spread footings founded on bedrock. Consideration was also given to supporting the abutments at a higher elevation within the bank on short steel H piles. Construction may be carried out within a sheeted cofferdam or within a till cofferdam built out from the shoreline.

It is anticipated that some settlement of the fill placed for the widening of the approach ramp will take place. It is recommended that the construction of the approach slabs and the final road surfacing be postponed for about one year following fill placement to reduce the differential settlement between the existing and new fills to a minimum.

1.0 INTRODUCTION

Golder Associates has been retained by the Ontario Ministry of Transportation and Communications to carry out a geotechnical investigation at the existing Muskrat River Bridge No. 29-33 in the Township of Pembroke, Ontario.

The purpose of the investigation was to determine the subsurface conditions at the site and based on the assessment and interpretation of this data, to provide geotechnical recommendations for the replacement of the existing bailey bridge structure.

The field work was carried out and this report was prepared in accordance with the terms of reference as outlined during telephone conversations between Mr. M. S. Devata of MTC and Mr. F. J. Heffernan of Golder Associates. At that time, it was proposed to put down three boreholes along the existing bridge in order to determine the elevation of the bedrock across the site, assuming a two-span bridge was to be constructed. Since the final design of the proposed bridge had not been established at this time, further boreholes were to be put down to determine the subsurface conditions at the locations of the existing piers if subsurface conditions were irregular over short distances. This latter information would be utilized in the event that the replacement would involve a three span structure.

2.0 SITE AND PROJECT DESCRIPTION

The project site is located on County Road No. 19 at Bridge No. 29-33 over Muskrat River. The natural river banks are approximately 3 m in height sloping at about 20 degrees on the east side of the river while the west banks range from 5 to 5.5 m in height with slopes of about 30 to 45 degrees. The banks are generally well vegetated and do not display evidence of recent movement.

A drainage gully, present immediately south of the existing bridge structure on the west bank, has been protected by placement of rip rap and shows signs of recent erosion.

The soils exposed in the west slope consist of grey silty clay overlying grey silty sand till which outcrops at about river water level. No signs of groundwater seepage were apparent in the slopes at the time of the investigation.

The existing bridge at the site is a three span steel girder bailey bridge (51.7 m in length). The concrete in the piers and abutments exhibit considerable deterioration at about river water level. The wooden deck had been replaced recently and it is understood that such replacement is required at regular intervals.

It is understood that the proposed two lane bridge structure will be constructed along the same centreline as the existing structure and the abutments will be placed at approximately the same location. It is not known at present whether the proposed structure will consist of two or three spans, however, it is understood that the length will remain approximately the same. It is further understood that the grade will be raised by about 0.6 m and that the existing fill material will be left in place. The depth of fill at the site was not determined during this investigation as it is understood from a conversation with Mr. S. Chen on site that, prior to construction, further auger holes would be put down by MTC personnel, for this purpose.

3.0 SUBSURFACE CONDITIONS

3.1 Site Geology

From published geological information*, the site lies within the physiographic region known as the Petawawa Sand Plain and is adjacent to an area which is part of the Muskrat Lake Ridges. The former is a delta in origin built in the Champlain Sea during the Fossmill stage of Lake Algonquin. The latter is an area of fault blocks which generally are not overlain by clay but instead by a thin veneer of sand and gravel.

3.2 Soil Stratigraphy

The detailed stratigraphy encountered in each of the boreholes put down during this investigation, is given on the attached Record of Borehole sheets. It should be noted that the stratigraphic boundaries indicated on the Record of Borehole sheets and stratigraphic section are not intended to define exact planes of geological change but represent transitions from one soil type to another. Subsurface conditions have been established at the borehole locations only and may vary between the boreholes. The locations of the boreholes and stratigraphic sections showing the inferred subsurface conditions are given on the attached Dwg. No. 878004-A. The results of laboratory testing carried out on representative samples are given on the Record of Borehole sheets and on Figures 1 to 4 inclusive.

In general, at the crest of the river banks, the site is underlain by sand and gravel fill which overlies up to 4.0 m of silty clay. The silty clay is underlain by up to 2.7 m of a till material which consists generally of sand with

---

* The Physiography of Southern Ontario, Chapman & Putnam, 1973.

some gravel and silt. The river bed material grades from silty sand and gravel to a sand till which overlies about 0.5 m of sand and gravel. Limestone bedrock was encountered at approximately constant elevation (elevation 117) across the site at all borehole locations.

3.2.1 River Banks

In Borehole 3, about 0.3 m of brown sand and gravel fill was encountered at ground surface.

About 4.0 to 4.3 m of grey/brown to grey silty clay with a trace of sand (Figure 1) was encountered below the fill in Borehole 3 and at ground surface in Borehole 1. The silty clay has a soft to very stiff consistency as measured by in situ field vane tests. The undrained shear strength, C_u, generally ranged from 80 to greater than 95 kPa in the upper 2.7 m of the deposit and this upper zone was grey/brown in colour indicating the presence of a weathered crust. Below the crustal zone, the silty clay became grey in colour and the C_u values decreased to about 20 kPa at the base of the deposit in Borehole 3. The vane strengths obtained throughout the deposit indicated a sensitivity of about 3 to 9 indicating the material to be moderately to highly sensitive.

Atterberg Limits tests gave liquid limits and plasticity limits of 41 to 50 and 23 to 27, respectively (see Figure 2) indicating a clay of intermediate plasticity. The natural water content of the samples of the silty clay ranged from 37 to 45 per cent and were generally close to or in excess of the liquid limit, reflecting the sensitivity noted in the vane tests.

Underlying the silty clay in Boreholes 1 and 3 and at ground surface in Borehole 4, 1.7 to 2.7 m of grey till was encountered. The composition of the till is variable and ranges from clayey silty sand with some gravel (Figure 3) to sand with some silt and gravel and a trace of clay (Figure 4). The natural water content of samples of the sand till ranged from 12 to 9 per cent. The till is in a loose to dense state of packing with 'N'* values ranging from 8 to 34 blows per 0.3 metres and these values increase with depth.

3.2.2 River Bottom

Borehole 2 was advanced through the bridge deck and B size casing was extended to the river bottom. Below the wooden plank bridge deck about 0.35 m of concrete was encountered. The upper 0.15 m was composed of a dirty sandy, highly weathered, slightly cemented material containing some wood fibres. About 0.15 m of wood timber was encountered below this and the wood was underlain by about 0.05 m of well cemented concrete.

At the river bed, about 0.6 m of grey silty sand and gravel was found to overlie about 0.25 m of sand till with a trace of gravel. These upper layers are in a dense state of packing with one 'N' value obtained of 31 blows per 0.3 metres. The natural water content of the sample of the till was measured at 8 per cent.

The till is underlain by about 0.4 m of gravel and grey sand with a trace of silt and clay. (See Figure 5.) One 'N' value of 34 blows per 0.3 metres was obtained indicating a dense state of packing. The water content of this sample was 9 per cent.

---

* 'N' value - Standard Penetration Index - see Explanation of Terms Used in Report.

3.3 Bedrock

Boreholes 2 and 4 were advanced 3 metres into the bedrock which underlies the site. Borehole 1 and 3 were terminated at refusal to the auger and the smooth grinding at this point indicated probable bedrock, although no attempt was made to drive the spoon sampler into bedrock.

The bedrock core samples obtained are grey fine grained to aphanitic limestone with occasional 50 to 200 mm thick shaley interbeds and minor solution cavities up to 25 mm in diameter. Generally core recovery values ranged from 70 to 100 per cent and RQD* values ranged from 15 to 52 per cent. The core samples from Borehole 4 reflect the upper range of RQD values while the top 1.5 m of bedrock in Borehole 2 is highly fractured with RQD values of about 15 per cent. Occasional chlorite filled fractures were noted throughout the bedrock at both borehole locations.

3.4 Groundwater Conditions

Following completion of Boreholes 1 and 4, a piezometer and a standpipe was installed in each of the holes, respectively. The details of the piezometer installations are given on the Record of Borehole sheets.

The water level in the piezometer in Borehole 1 was monitored on the day of and after the installation. The elevation of the water level on the day following installation was at about 118.7 m which coincides with the base of the silty clay deposit. The water level in the standpipe in Borehole 4 was at about elevation 119.2 m on the day of installation.

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* RQD - Modified Recovery - See Explanation of Terms Used in Report.
It should be noted that the RQD values of the bedrock are based on Bx core size.

The river water level was at elevation 118.7 m and the readings in the installation are consistent with a groundwater table sloping to river level. However, an additional reading will be taken to determine if the recorded levels have stabilized and the results will be reported by letter.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 Bridge Foundations

This section of the report provides geotechnical recommendations for a two span bridge structure with a central pier located at about Sta. 20+713 along Line 'A'. A small variation from about 116.8 to 117.2 m in the elevation of the bedrock was noted across the site. Based on estimated river bottom contours, the depth of overburden may vary from about 1.0 to 1.5 m between the locations of the existing piers. It is considered, therefore, that these recommendations are considered applicable as well to a three span bridge structure with central piers placed at the locations of the existing piers.

Construction of the bridge piers and abutments should be carried out at times of the year when the river level is low. Construction of the central pier may be carried out within a sheeted cofferdam. The use of a glacial till cofferdam built out from the shoreline may also be considered. Because of space restrictions and river flow channel requirements, this construction scheme may be more feasible for a three span structure.

Bearing capacity calculations have been carried out in accordance with the current Ontario Highway Bridge Design Code (OHBDC).

4.1.1 Pier Foundation

About 1.0 m of overburden was encountered in Borehole 2 at the location of the proposed central pier. The elevation of the bedrock at the proposed location for the central pier is about 117.2 m and it is anticipated that it steps down slightly to both the east and west. The RQD values (based on Bx core size) ranged from 15 to 35 per cent in the upper 3.1 m with the upper 1.5 m being highly fractured.

The bridge pier may be supported on spread footings founded on competent limestone bedrock. The sand, till and gravel above the bedrock must be removed and the bedrock surface must be cleared of all loose material before pouring concrete. It is estimated that a minimum of 0.4 m of bedrock may have to be removed to reach sound bedrock. It is essential that the foundations are inspected before placing concrete. Should jointing or solution cavities be extensive over the area of the footing, the founding elevation may have to be lowered. A soil cover of about 1.5 m should be provided to the footings for frost and scour protection purposes.

The factored bearing capacity at Ultimate Limit States for spread footings on limestone bedrock is 3000 kPa. The bearing capacity at Serviceability Limit States, Type III, is not relevant since the stresses required to produce detrimental settlement of the structure will be much larger than the recommended value for the factored bearing capacity at Ultimate Limit States.

A coefficient of friction of 0.5 may be assumed between the bedrock surface and the concrete footings, provided that the footing bases are adequately cleaned and prepared to ensure a good contact between the concrete and the bedrock.

4.1.2 Abutment Foundations

The bridge pier foundation will provide rigid support to the bridge. Abutment foundations must also be rigid or significant differential settlement will occur. The abutments may be supported on spread footings or on steel H piles founded on bedrock.

4.1.2.1 Spread Footings

The construction drawings for the existing bridge indicate that the bridge abutments are founded on stiff clay at about 5.2 to 5.4 m below the base of the bridge deck. Based on the results of this investigation, it is considered that the footings are, in fact, founded within the till below the silty clay.

The design recommendations as discussed in Section 4.1.1 will apply, as well, to the abutment spread footings founded on sound bedrock. The elevation of the bedrock is expected to be at about elevation 116.8 to 117.0 m. The RQD values for the core samples obtained in Borehole 4 at the proposed east abutment were higher than those obtained below the river bed and ranged from 40 to 52 per cent.

Excavations at the proposed abutment locations would involve undercutting of the adjacent slope and would extend into the silty clay and till materials. Temporary slopes into these materials may be carried out with side slopes of 1.5 to 1. Groundwater seepage into the excavation is anticipated to be minimal, however, due to the proximity of the abutments to the river edge, a cut off wall, such as sheet piling driven to bedrock will be required at the edge of the excavation adjacent to the river. A glacial till cofferdam can also be used to direct the river flow around the abutment foundation area. The river banks should be regraded where altered during construction to an angle no steeper than their original slope and erosion protection should be provided.

4.1.2.2 Steel H Piles

Considerable regrading of the river banks would be required for construction of spread footings at the abutment locations; in particular, at the west abutment. Alternatively, the abutment footings may be placed at a higher elevation within the bank and supported on short steel H piles driven to bedrock. Although boulders were not encountered during this investigation, the possibility of their presence within the till cannot be overlooked.

As a guide, an HP 310 x 110 driven to a set of 20 blows per 25 mm of penetration for the final 75 mm of driving, using a hammer energy approaching but not exceeding 60 kJ, will have a capacity of Ultimate Limit States of 1150 kN. On reaching the required set, the pile should be subjected to two further groups of 20 blows. The average set measured for each group should not be greater than the set measured for each of the previous groups. Serviceability Limit States does not apply to piles founded on limestone bedrock since settlement will be negligible.

4.2 Bridge Abutments as Retaining Walls

Where the abutments are required to act as retaining walls the lateral earth loads will depend on the type and method of placement of the fill materials. The following recommendations are made for the design of the abutment retaining walls:

i) Selected granular fill, such as M.T.C. Granular 'B' should be used as backfill immediately behind the structures. The granular fill should be placed in the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the structure's footing or pile cap.

ii) All granular fill should be compacted in 200 mm thick lifts to 95 per cent of the Standard Proctor dry density of the material. Heavy compaction equipment should not be used behind any structure within a lateral distance equal to the current height of the fill above the base of the structure.

iii) Provided that the above criteria are satisfied, and the abutments are less than 10 metres high, an equivalent fluid pressure of 8 kPa/metre and 6.5 kPa/metre may be used to calculate earth pressure at Ultimate and Serviceability Limit States respectively. A coefficient of friction equal to 0.5 may be assumed between the concrete footings and the bedrock, provided that the footing bases are adequately prepared to ensure a good contact between the concrete and the rocks.

An adequate drainage system should be provided behind the abutments to prevent build-up of hydrostatic forces during drawdown. The drainage system should include a properly designed filter to prevent clogging of the pipes. Provisions should be made to allow cleaning or rodding of the pipes, should they become clogged.

4.3 Approaches

It is understood that the existing roadway fill is to be left in place. It is anticipated that settlement of the existing embankment materials and the subsoils has, by now, reduced to very minor amounts. Additional fill will be required for the widening of the approach ramps. Some settlement will be expected to occur within both the underlying silty clay and the additional fill. It is understood that the proposed bridge design includes an approach slab to the abutment to compensate for longitudinal differential settlement. It is recommended that the approach slab construction and final paving should be postponed one year following fill placement to allow a large percentage of the settlement to take place. It is anticipated that further differential settlement after this period will be less than 250 mm.

All topsoil and other organic material should be removed and the subgrade proofrolled prior to placing of suitable fill. The adjacent new fill should be keyed into the existing fill in lifts not exceeding 200 mm and compacted to a minimum of 95 per cent of the Standard Proctor dry density.

GOLDER ASSOCIATES

REGISTERED PROFESSIONAL ENGINEER
A. S. POSCHMANN
PROVINCE OF ONTARIO

A.S. Poschmann, P.Eng.

F.J. Heffernan, P.Eng.

ASP/FJH/cg

APPENDIX A

FIELD WORK PROCEDURES

July, 1983 831-1150

FIELD WORK PROCEDURES

The field work for this investigation was carried out on July 7 and 8, 1983, at which time a total of 4 boreholes (numbered 1 to 4) were put down at the locations shown on Drawing No. 878004-A. A bombardier mounted CME 55 power auger (supplied by F.E. Johnston Drilling Ltd.) was used with 114 mm inside diameter hollow stem augers and Bw casing size. Bedrock samples were obtained in Bx core size. A total of 22 m of sampled and/or vane test borings and corings were put down to depths of between 4.5 to 6.1 m.

Samples were obtained at 0.75 to 1.5 m intervals of depths using a conventional 50 mm O.D. split barrel sampler in conjunction with Standard Penetration tests. One 75 mm O.D. Shelby sample was taken in Borehole 1. In Borehole 3, continuous vane testing was carried out within the soft cohesive soils. In Boreholes 2 and 4, bedrock was cored for a depth of 3 m. Details of the drilling and sampling operations are summarized on the Record of Borehole sheets.

The field work was supervised throughout by a member of Golder Associates engineering staff who located the borings in the field, directed the drilling and sampling operations, and logged the borings.

The borehole locations and elevations were surveyed by Golder Associates. The elevations were referenced to Geodetic datum (B.M. Elevation 125.752 m - South end of concrete window sill, 21.8 m Right of Station 20+762.2).

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

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Ontario

RECORD OF BOREHOLE No 1

W P 87-80-04 LOCATION Sta. 20+736.1 o/s 5.0m Lt. Line 'A' ORIGINATED BY ASP

DIST ____ HWY ____ BOREHOLE TYPE Hollow Stem Auger COMPILED BY MKW

DATUM Geodetic DATE July 7, 1983 CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
122.64	Ground Surface							
0.00	Silty Clay, trace sand					122		
	Firm to very stiff	1	SS	6				
	Grey/brown	2	TW	PH				0 0 50 50
		3	SS	4		120	+5	
		4	SS	3			+5.5	0 1 50 49
118.65 / 3.99	Clayey Silty Sand Till, some gravel. Dense Grey	5	SS	34		118		14 34 26 26
116.79 / 5.85	End of Hole. Refusal to Auger at Probable Bedrock				July 8, 1983	116		

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

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2

W P 87-80-04 LOCATION Sta. 20+712.5 o/s 0.9m Lt. Line 'A' ORIGINATED BY ASP

DIST ___ HWY ___ BOREHOLE TYPE Hollow Stem Auger, Bx Rock Core COMPILED BY MKW

DATUM Geodetic DATE July 7, 1983 CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
118.69	River Level										
0.00 / 118.07	Water						118				
0.62	Silty Sand and Gravel		1	SS	31				○		
0.87	Sand Till Grey										
1.07 / 117.18	Gravel and Sand, trace silt and clay. Dense Grey		2	SS	34				○		50 45 5 1
1.51	Limestone Bedrock, fine grained to aphanitic, very to moderately close jointing, very poor to poor RQD. Occasional solution cavities (less than 25 mm)		3	Bx RC	REC 70%		116				
			4	Bx RC	REC 100%						
114.21 / 4.48	End of Hole						114				

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

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

OFFICE REPORT ON SOIL EXPLORATION

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Ontario

RECORD OF BOREHOLE No 3

W P 87-80-04 LOCATION Sta. 20+740.9 o/s 6.0m Rt Line 'A' ORIGINATED BY ASP
DIST ____ HWY ____ BOREHOLE TYPE Hollow Stem Auger, N Vane COMPILED BY MKW
DATUM Geodetic DATE July 7, 1983 CHECKED BY ASP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (+ FIELD VANE)	WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
123.05	Ground Surface										
0.00 122.75	Fill - sand and gravel Brown										
0.30	Silty Clay, trace sand Very Stiff to Soft Grey/Brown to Grey						122	$+^8$; >95 + 6kPa; $+^5$; >95 + 6kPa; $+^9$; $+^8$; $+^3$			
							120				
118.73 4.32	Silty Sand Till, some gravel, trace clay					▽	118				
116.99 6.06	End of Hole Refusal to Auger at Probable Bedrock					July 8, 1983	116				

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

$+^3$, $\times^5$: Numbers refer to Sensitivity   15 –◇– 5 (%) STRAIN AT FAILURE (20)

OFFICE REPORT ON SOIL EXPLORATION

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RECORD OF BOREHOLE No 4

W P 87-80-04 LOCATION Sta. 20+697.7 o/s 4.8m Rt Line 'A' ORIGINATED BY ASP

DIST ____ HWY ____ BOREHOLE TYPE Hollow Stem Auger, Bx Rock Core COMPILED BY MKW

DATUM Geodetic DATE July 8, 1983 CHECKED BY BSF

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
119.55	Ground Surface							
0.00	Sand Till, some silt and gravel, trace clay	1	SS	8			o	
	Loose to Dense	2	SS	18		118	o	19 45 17 9
116.84	Grey/Brown to Grey	3	SS	45 @ 250mm			o	
2.71	Limestone Bedrock, fine grained to aphanitic, occasional shaley interbeds, very close to moderately close jointing, poor to fair RQD. Occasional solution cavities (5 25 mm).	4	Bx RC	REC 88%		116		
		5	Bx RC	REC 52%		114		
113.81								
5.74	End of Hole				July 8 1983			

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 o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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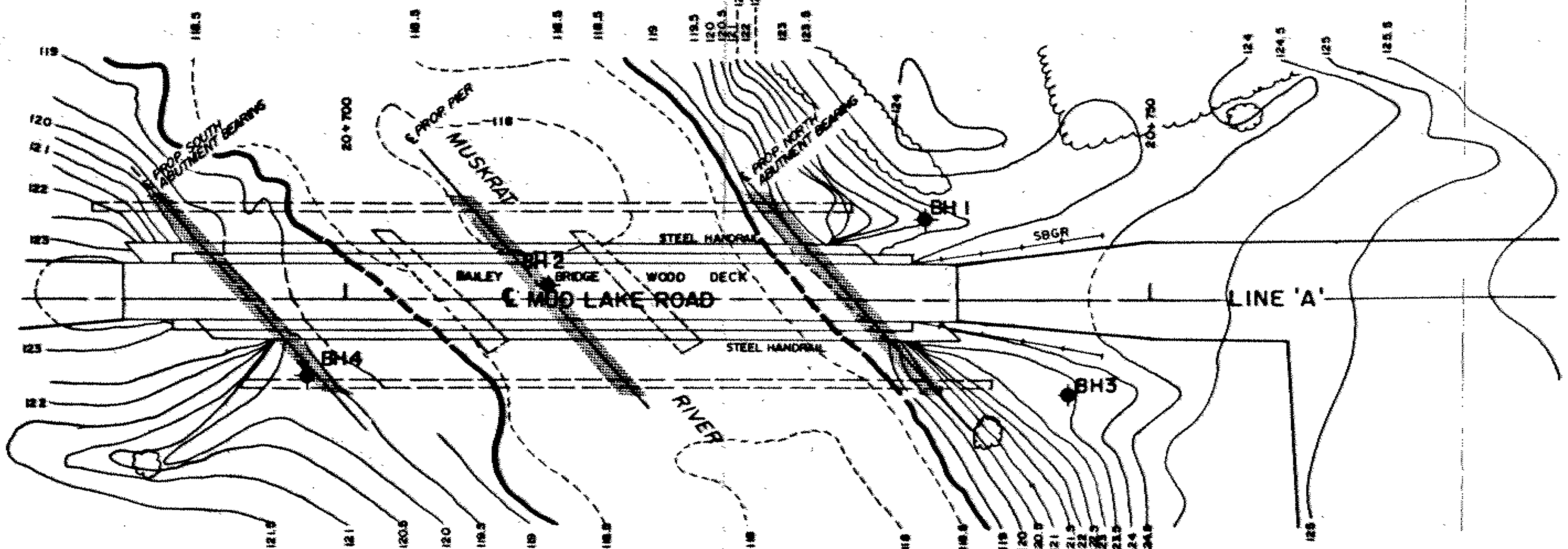
CO RENFREW
TWP STAFFORD
CON I
LOT 28

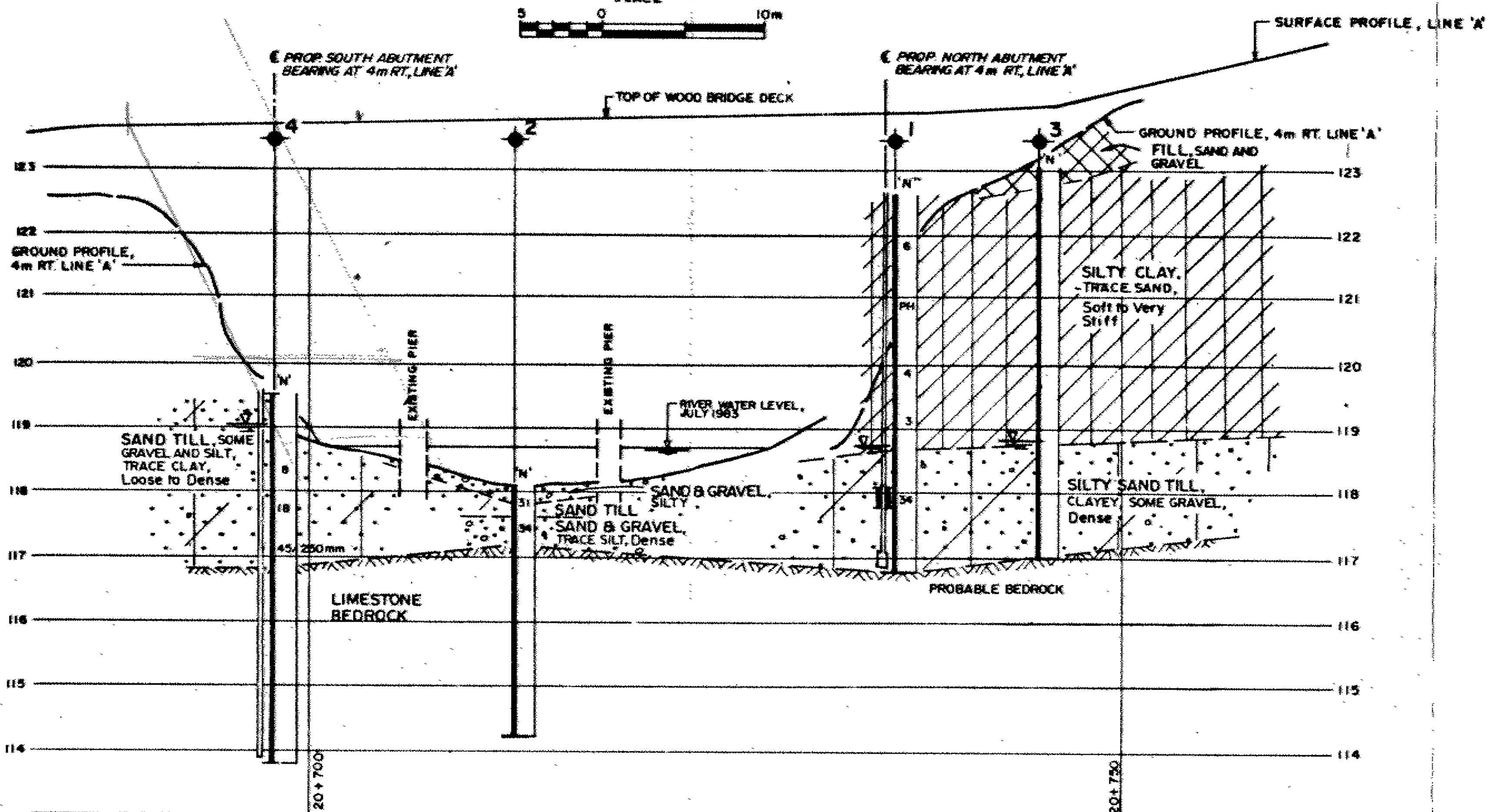
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No
WP No 87-80-04

MUSKRAT RIVER BRIDGE
No. 29-33
BORE HOLE LOCATIONS & SOIL STRATA

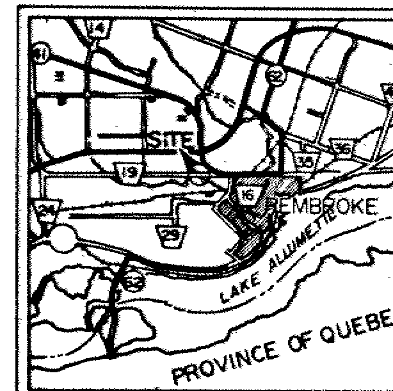
SHEET





SECTION ALONG LINE 'A'

SCALE
5 0 5 10 15m HORIZ.
1.0 0.5 0 1 2 3 4m VERT.



KEY PLAN
SCALE
1 0.5 0 1 2 km

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Bentonite Seal
- Piezometer
- Standpipe

No	ELEVATION	STATION	OFFSET
1	122.64	20+736.1	5.0m LT., 'A'
2	118.69	20+712.5	0.9m LT., 'A'
3	123.05	20+740.9	6.0m RT., 'A'
4	119.55	20+697.7	4.8m RT., 'A'

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

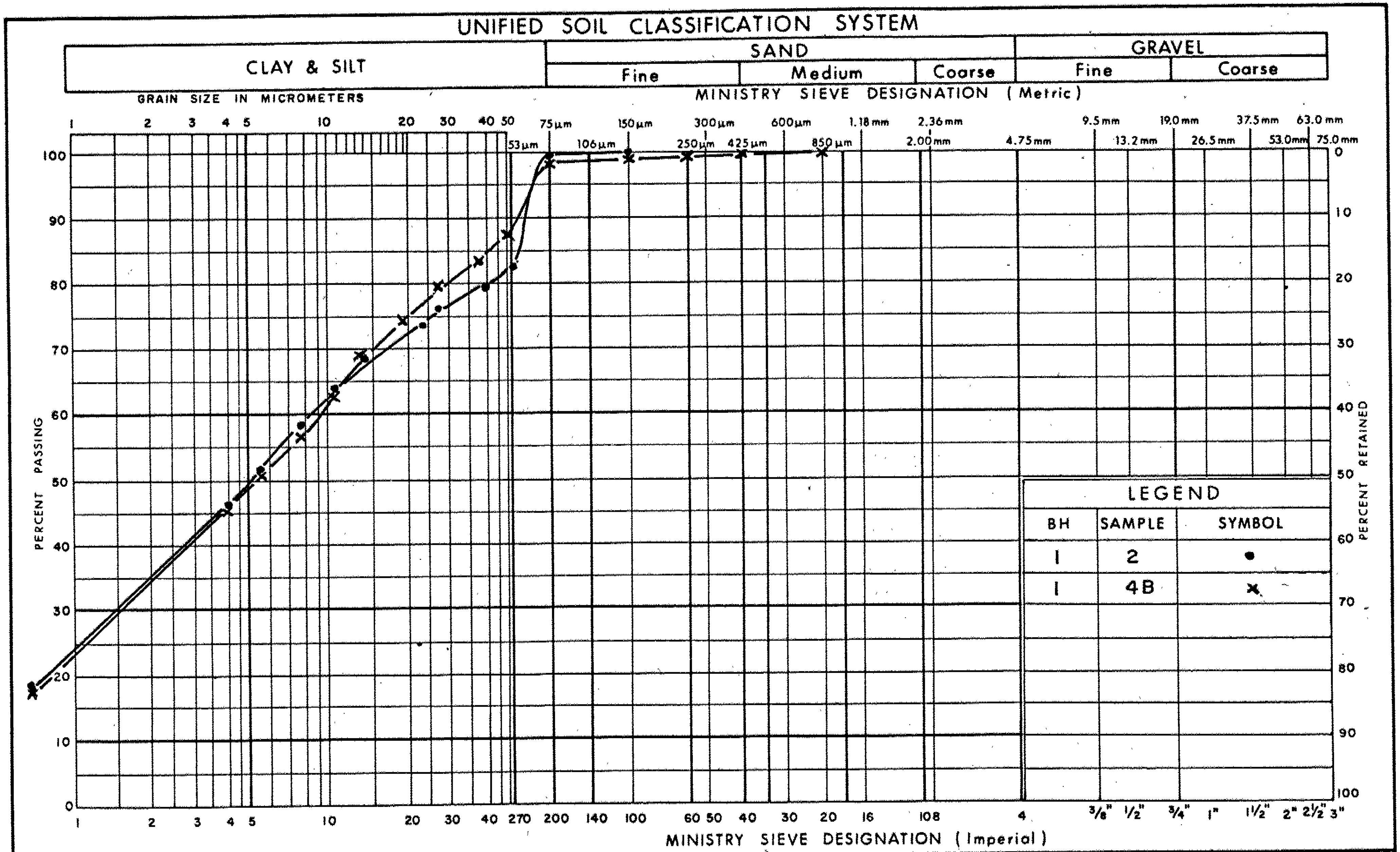
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2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

REV	DATE	BY	DESCRIPTION

Geocres No

HWY No			DIST
SUBM'D AP	CHECKED JB	DATE JULY 25, 1983	SITE
DRAWN MW	CHECKED AP	APPROVED	DWG 878004-A

78 12 M



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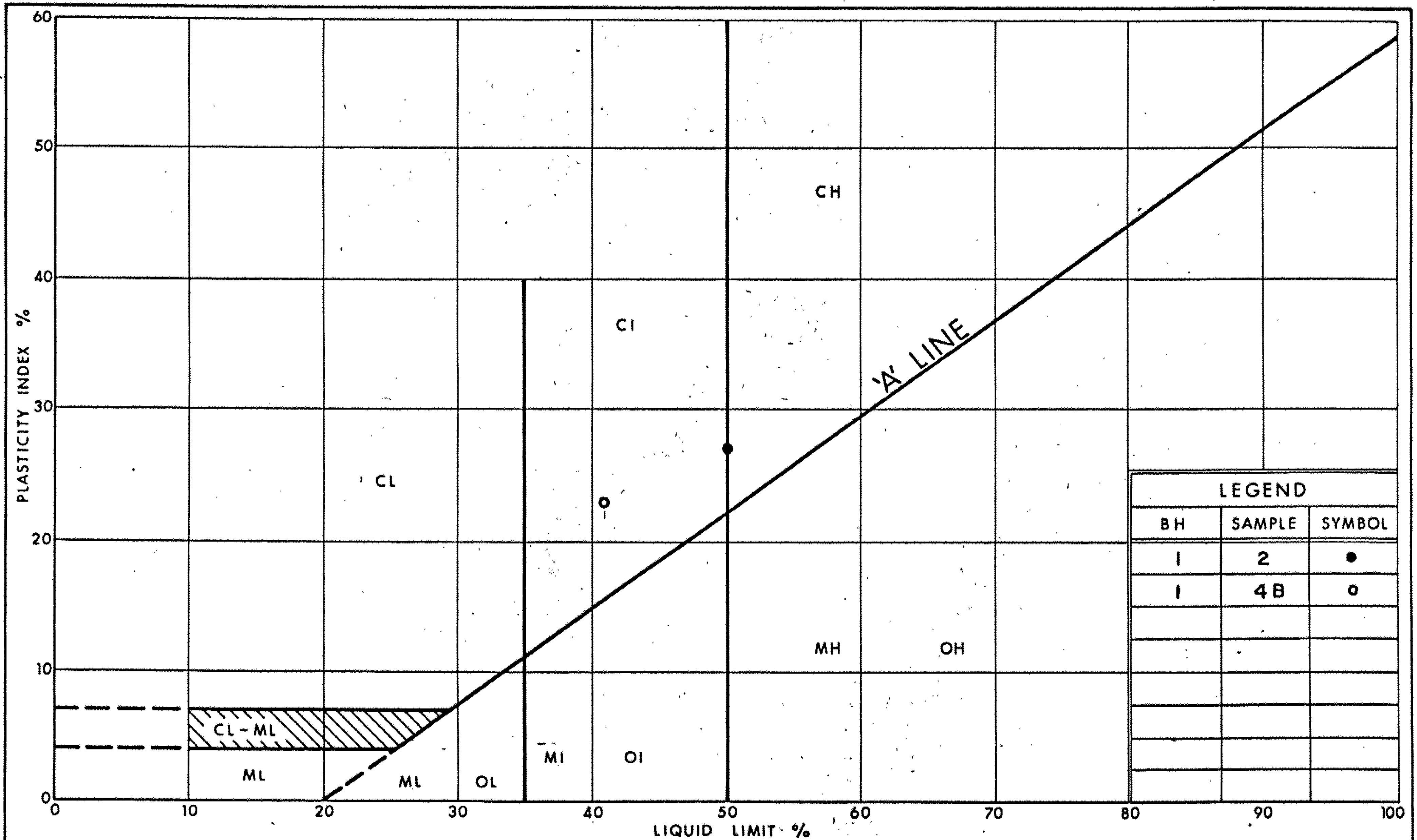
GRAIN SIZE DISTRIBUTION
SILTY CLAY

FIG No 1

W P 87 – 80 – 04



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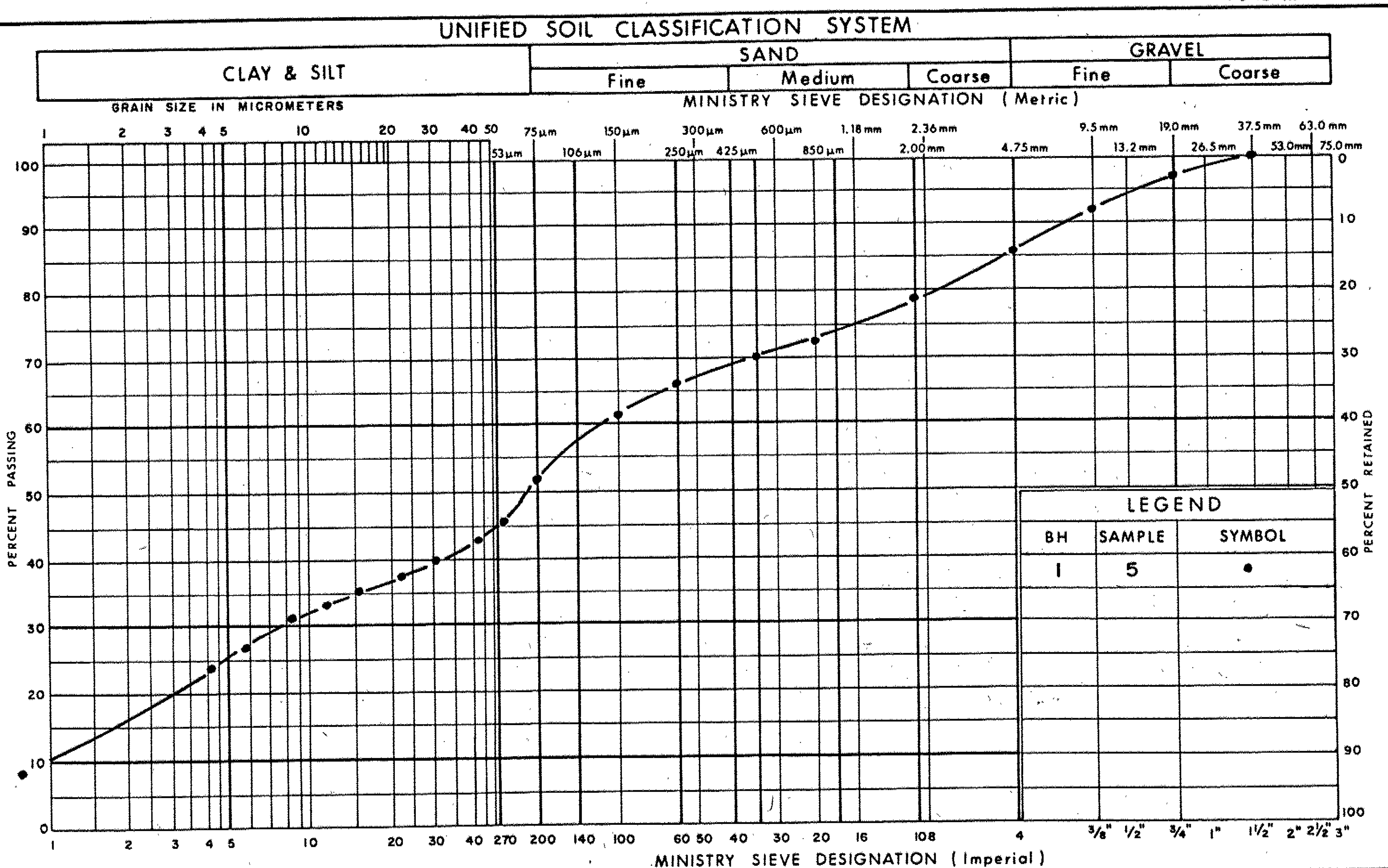
PLASTICITY CHART
SILTY CLAY

FIG No 2
WP 87 - 80 - 04

78 12 M



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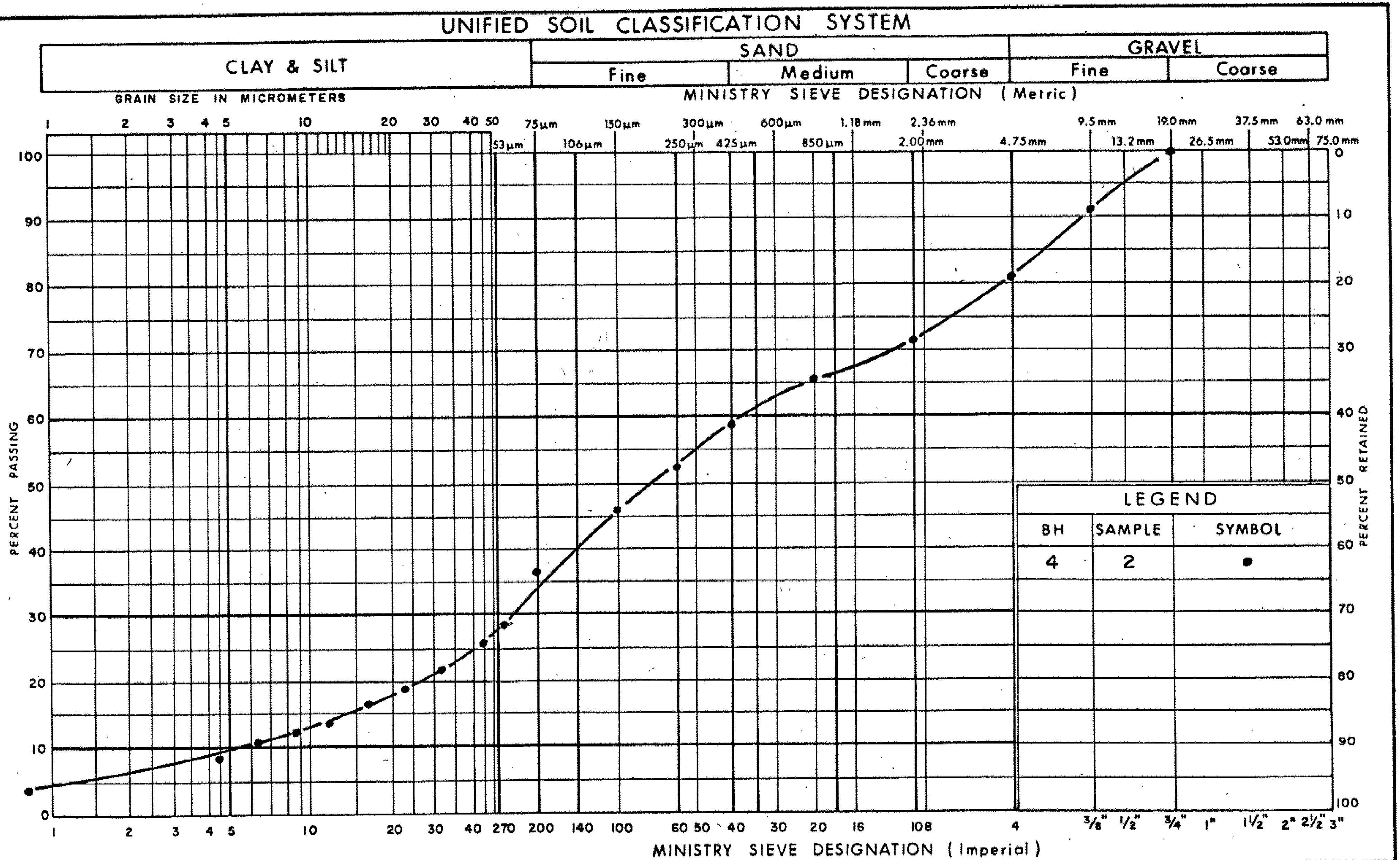
GRAIN SIZE DISTRIBUTION
CLAYEY SILTY SAND TILL

FIG No 3
W P 87 – 80 – 04

78 12 M



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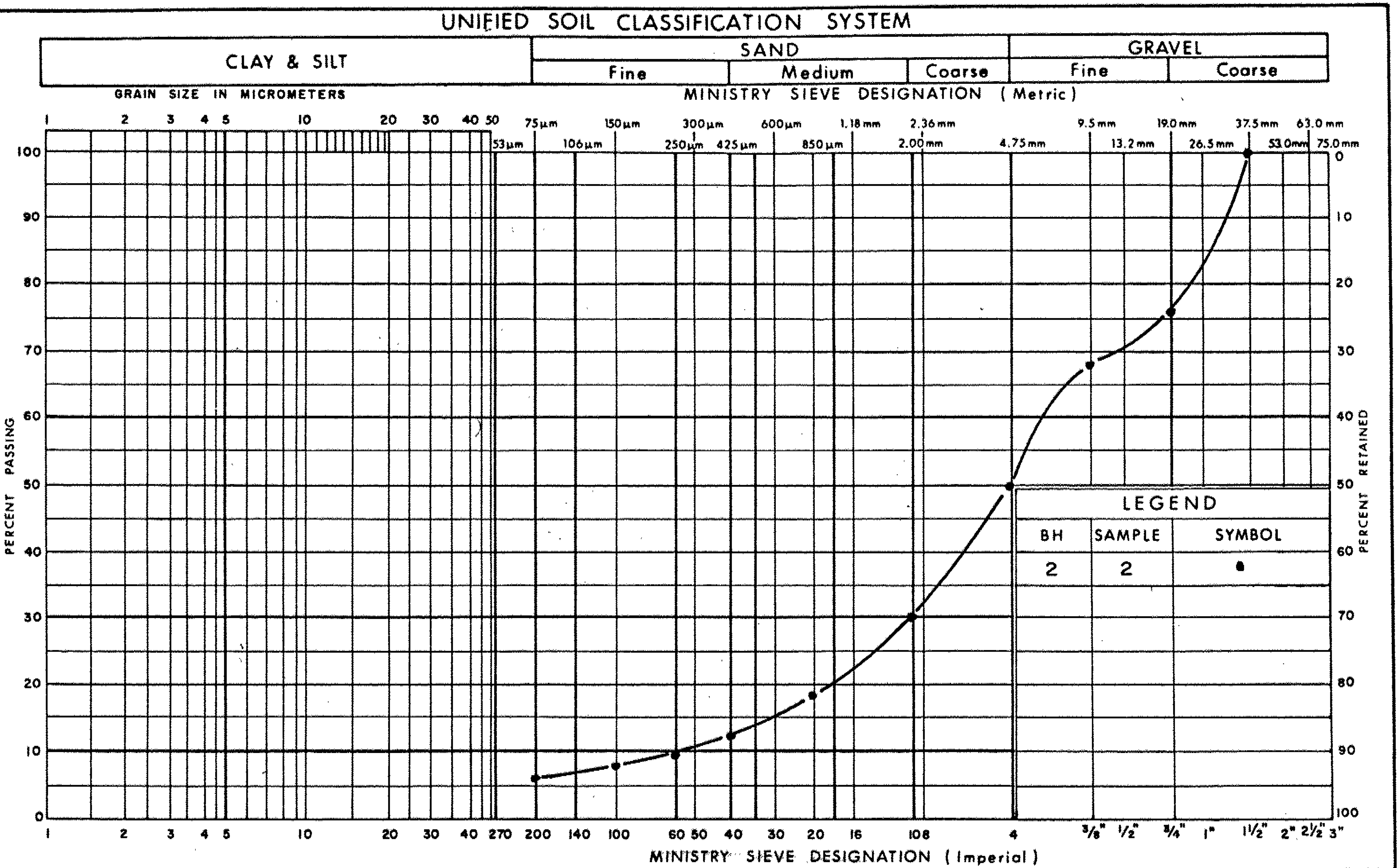
GRAIN SIZE DISTRIBUTION
SAND TILL

FIG No 4

W P 87 – 80 – 04

78 12 M



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