

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31E-37

DIST. 13 REGION Northern

W.P. No. 138-73-61

CONT. No. 77-66

W.O. No. _____

STR. SITE No. 44-50

HWY. No. 522

LOCATION South River Bridge

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. 2

REMARKS: 2 documents to be unfolded
before microfilming

DIST. 13

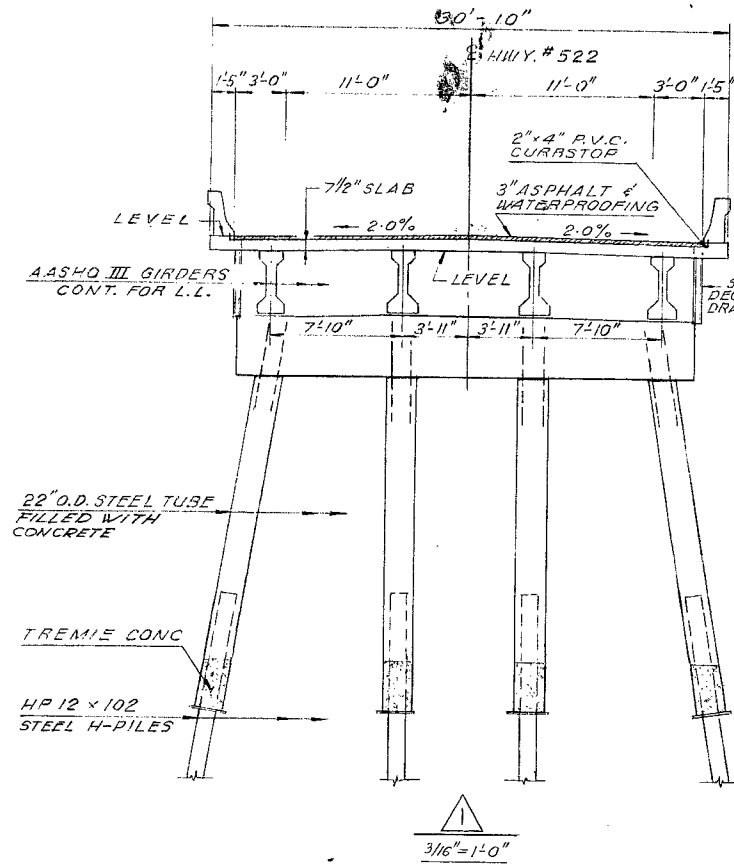
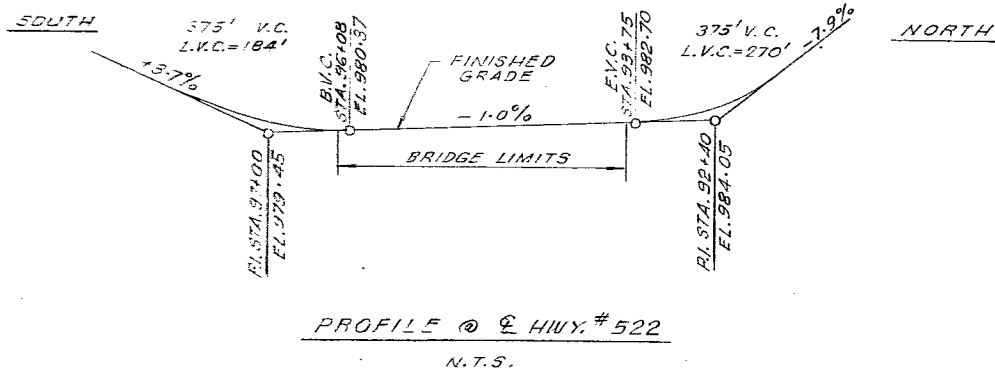
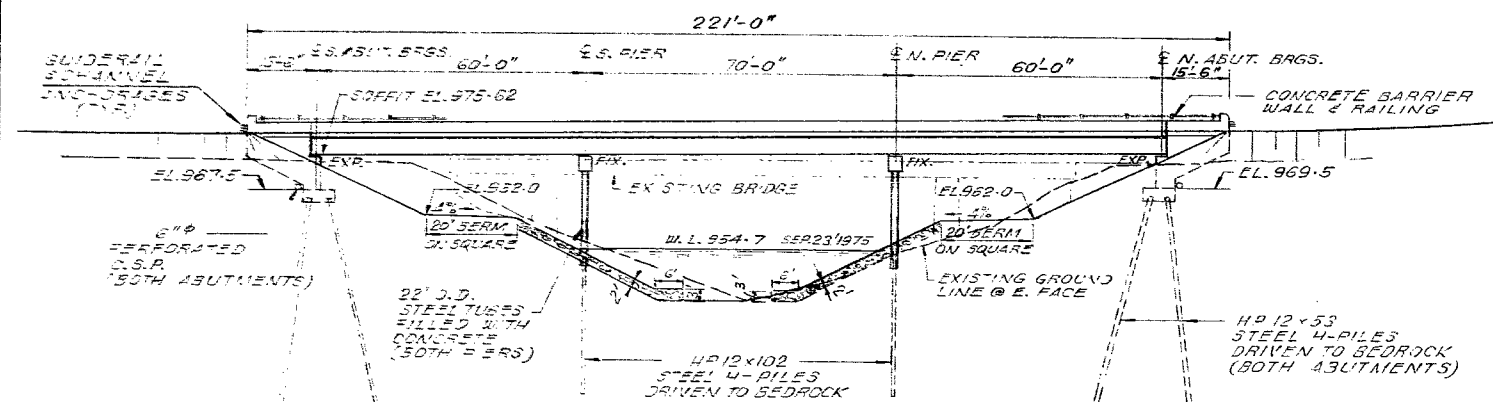
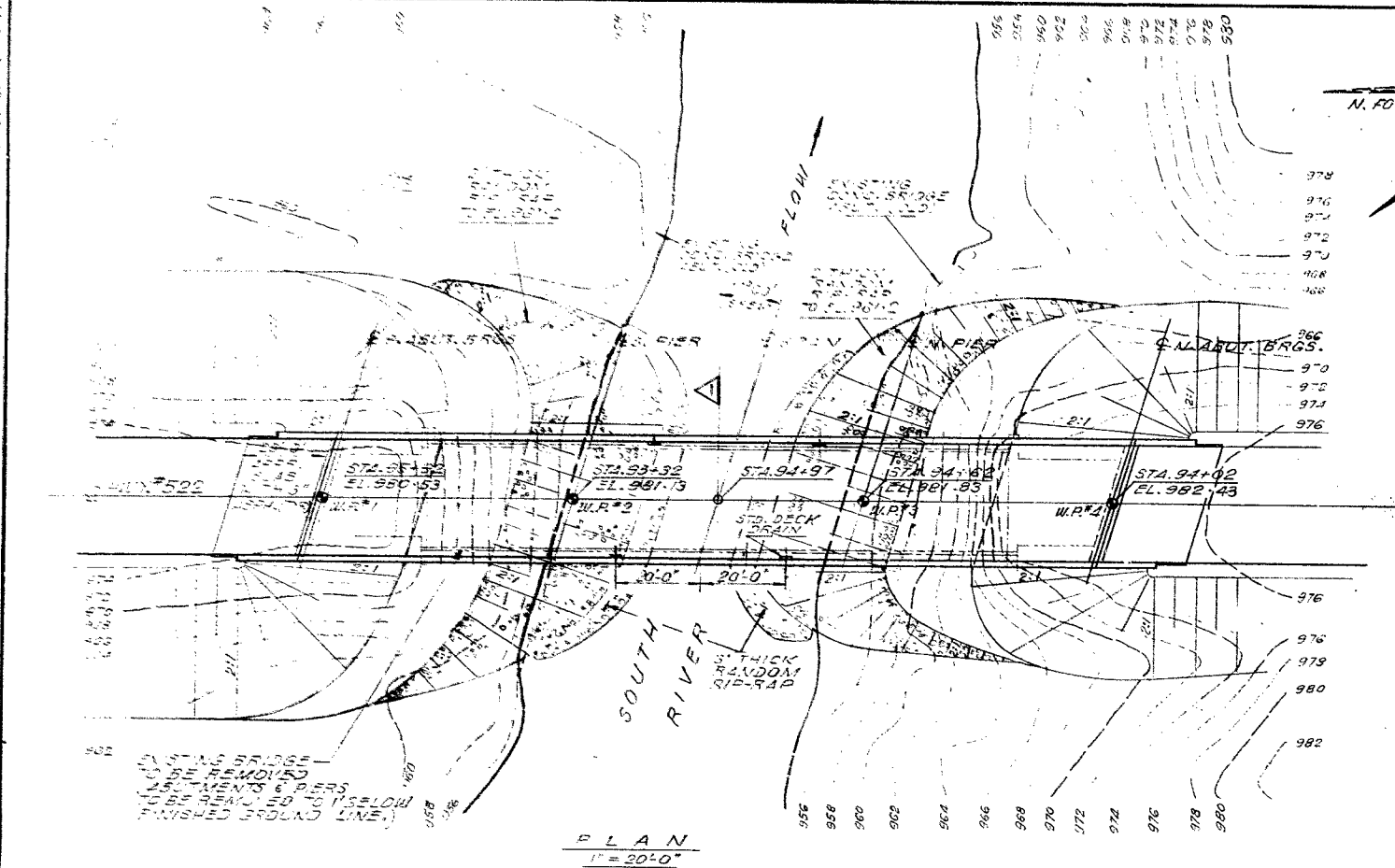
CONT No
WP No 139-73-01

SOUTH RIVER BRIDGE
1.9 MI. WEST OF HWY. #11
GENERAL LAYOUT

SHEET



17°00' SKEW
SIN = .292 37
COS = .956 30
TAN = .305 73



CONCRETE QUANTITIES

CONCRETE IN PIER CAPS, ABUTMENTS & WINGWALLS 126 C.Y.
CONCRETE IN DECK & DIAPHRAGMS 162 C.Y.
CONCRETE IN BARRIER WALLS 39 C.Y.
CONCRETE IN APPROACH SLABS 36 C.Y.

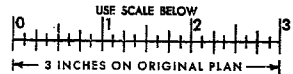
NOTE RE

SEQUENCE OF CONSTRUCTION
BERMS & EARTH GRADING
BETW. ABUTMENTS TO BE
COMPLETED BEFORE PILE
DRIVING BEGINS.

B.M. EL. 976.22
GEODETIC DATUM
N. & W. IN S. ROOT
0.9' SPRUCE
98.0' RT. 93+84



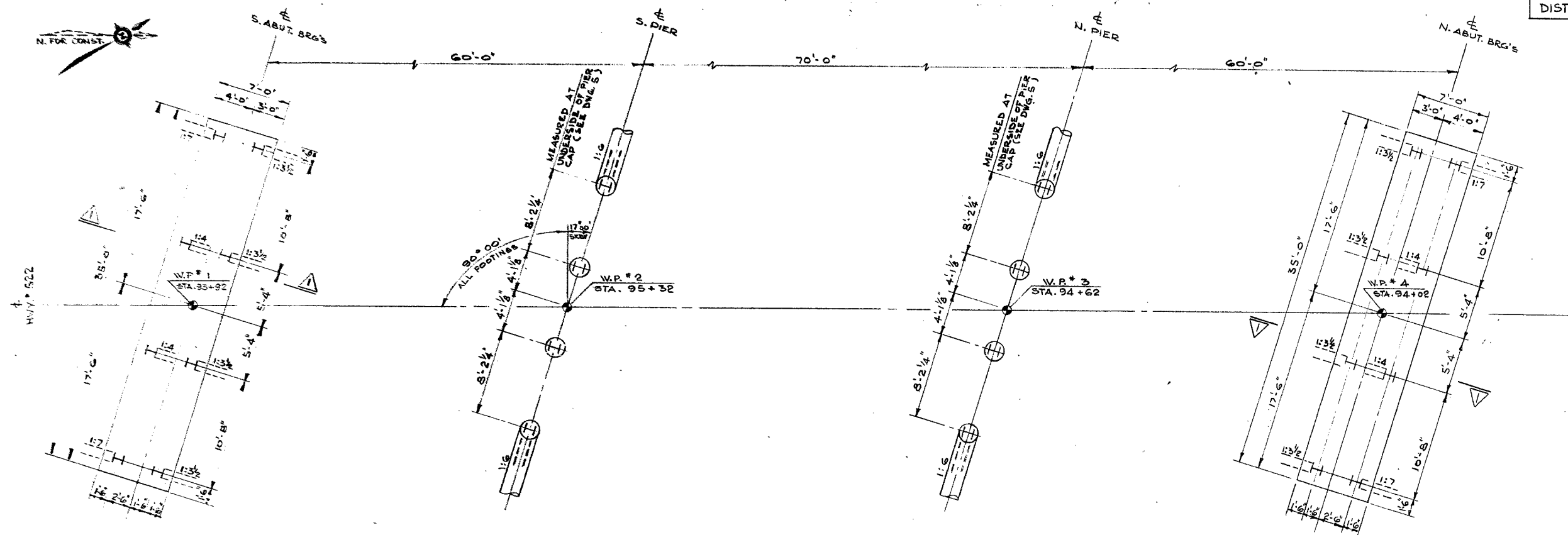
FOR REDUCED PLAN



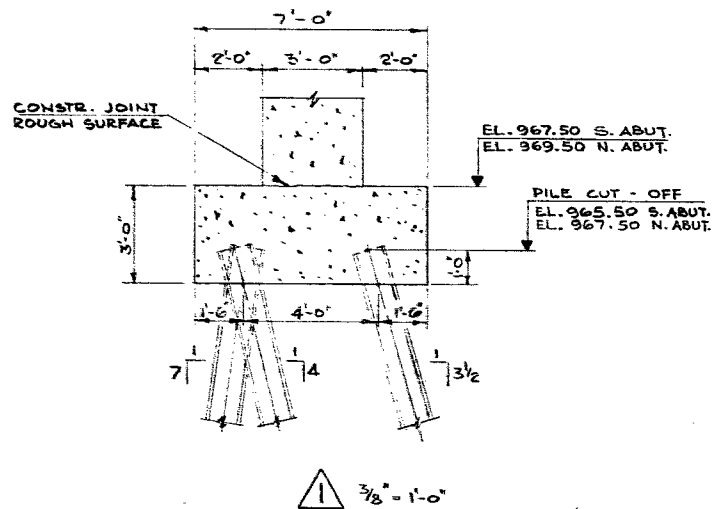
REVISIONS	DATE	BY	DESCRIPTION



31E-37
GROSS 112



FOOTING LAYOUT
SCALE: 3/16" = 1'-0"



LIST OF STEEL H-PILES

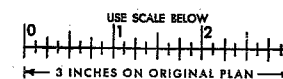
LOCATION	TYPE	No REQ'D	LENGTH
S. ABUT.	HP 12 x 53	8	52'-0"
S. PIER	HP 12 x 102	4	38'-0"
N. PIER	HP 12 x 102	4	38'-0"
N. ABUT.	HP 12 x 53	8	52'-0"



31E-37
REVISED No.



FOR REDUCED PLAN



REVISIONS	DATE	BY	DESCRIPTION
1			
2			
3			

DESIGN R.O.L. CHECK F.C. LOADING HS 20-44 DATE SEPT/76
DRAWING S.M. CHECK R.O.L. SITE No 44-50 DWG 3



Memorandum

To: J. McAllister (2)
Head, Structural Section
Northern Region, North Bay

From: Soil Mechanics Section
Geotechnical Office
West Building, Downsview

Attention:

Date: January 21, 1976

Our File Ref.

In Reply to

Subject:

FOUNDATION INVESTIGATION REPORT

W.P. 138-73-01
Site No. 44-50
Hwy. 522 District 13
South River Bridge
1.9 Miles West of Hwy. 11

Attached we are forwarding to you our detailed Foundation Investigation Report on the subsoil conditions existing at the above mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your requirements. Should additional information be required, please do not hesitate to contact our Office.

M. Devata
M. DEVATA
Supervising Engineer

cc: M.J. Bernhardt
C.G. Campbell
R.A. Dorton
B.J. Giroux
S. McCombie
R.S. Pillar
G.A. Wrong

R. Hore
J. Anderson)
R. Murphy) memo only
G. Sloan)

Files
Record Services

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FOUNDATION INVESTIGATION REPORT

for

W.P. 138-73-01
Site No. 44-50
Hwy. 55 District 13
South River Bridge
1.9 Miles West of Hwy. 11

1. INTRODUCTION

It is understood that the present structure at the above mentioned site was designed for H15 loading and the structural analysis carried out for HS20-44 AASHO loading, indicated that the existing superstructure is overloaded. Thus, a replacement structure is being considered. Mr. J.C. McAllister, Regional Structural Planning Supervisor, Northern Region, requested the Soil Mechanics Section, in a memorandum dated August 20, 1975, to carry out a foundation investigation for the proposed structure, which is to be built along the same horizontal alignment as the existing one. However, the pier and abutment locations for the new three span structure are not finalized, but it is believed that they may be located in the proximity of the existing piers and abutments.

This report contains the results of the investigation and our recommendations pertaining to the design of the proposed structure foundations, as well as the approach embankments.

2. SITE AND GEOLOGY

The site is located on Hwy. 522, about 1.5 miles west of Hwy. 11, near the village of Trout Creek in the Township of South Himsworth, and in the District of Parry Sound.

In the vicinity of the site, the topography is rolling to gently undulating. The surrounding area is heavily wooded. South River meanders in this area and flows in a northerly direction draining into Lake Nipissing. At the time of the investigation, however, no noticeable flow was observed. The remnants of the old bridge abutments can be seen just north of the existing structure.

Geologically, the area is part of the Precambrian Shield. In the

"Physiography of the Georgian Bay - Ottawa Valley Area of Southern Ontario" by Chapman, the site is located in an area known as 'Sand Plains'.

3. EXISTING STRUCTURE

The existing structure is a three span (24' - 90' - 24': total 138') low steel truss bridge of 26' width and was built in 1940. According to the information gathered from the local patrolman, periodic padding was required due to the settlement of the embankment since the completion of the structure and approaches. Transverse cracks are visible on the underside of the deck. Due to the spalling of the concrete in the deck at various places, reinforcing steel is exposed. Water seepage through the deck has resulted in the rusting of some portions of the top flanges of the stringers and beams and also the exposed reinforced steel.

The existing embankment has been constructed to elevation 974.5 - 975.0 with 2:1 side slopes. It was established from the old drawing that the original ground surface was at about elevation 959.0. The immediate approaches have settled as discussed previously. This is attributed primarily due to:

- (a) poor compaction of the fill material
- (b) the compressible nature of the lower portion of the cohesive fill material (clayey silt to silt)
- (c) and the consolidation of the cohesive portion of the in situ material due to embankment loading

From the drawings of the existing bridge (D-509-11-1), it appears that 1.5:1 slopes were constructed in the longitudinal direction. Referring to the recent Plan E-5058-1, the present longitudinal slopes are much flatter (3:1). From this it appears that the approaches in the longitudinal direction were unstable with 1.5:1 slopes and consequently fills in this direction were flattened to 3:1 in order to ensure stability. In our opinion, the instability was caused mainly due to the following:

- (a) The surficial run-off was not adequately controlled and as a result of this a major amount of water follows the path along the forward slopes.
- (b) Poor slope protection of river banks up to the high high water level.
- (c) The frost susceptible nature of the fill material (sandy silt to silty sand).

Gabion walls were placed in front of both abutment walls several years ago and their purposes of installation are not known, but they have settled and moved laterally towards the river since then. Movements of the abutment walls are not believed to have taken place.

The present abutments are believed to be founded on spread footings at about elevation 952 ft. and 949 ft. for the north and south abutments respectively. The bottoms of the pier caps are located at about elevation 949.4. The piers are believed to be supported by 25' long timber piles (reference to Dwg. D-509-11-1) whose tip elevations are at about elevation 924.0. According to Contract 39-120, no precise details of the type of piles are available.

4. FIELD AND LABORATORY INVESTIGATION PROCEDURES

Four sampled boreholes, one of which was accompanied by a dynamic cone penetration test, were put down during the course of the field investigation. The borings and the penetration test were advanced by means of a continuous flight auger machine (C.M.E. 45) adapted for soil sampling purposes.

Samples of the overburden were obtained at required depths by means of a 2" O.D. split-spoon sampler. The sampler was driven into the soil with a driving energy in accordance with the specifications for Standard Penetration Test. The same method was used to advance the dynamic cone penetration test. When it was not possible to recover the sampled material in the split-spoon sampler due to the very moist and loose nature of the granular soil, slotted tube samples were taken. Few samples of the slightly cohesive stratum were obtained in 2" I.D. Shelby tubes which were hydraulically pushed into the soil. In situ vane tests were also carried out within this zone to determine the undrained shear strengths. Bedrock was proven at three boring locations by obtaining BXL size rock core samples. The groundwater conditions were observed by recording the water levels in the open boreholes during the period of the field investigation.

The soil, bedrock, and groundwater conditions encountered in the borings are presented on the Record of Borehole Sheets. Surveying of the

boreholes in the field was carried out by personnel from the Engineering Surveys Office, Northern Region. All elevations are referenced to a Geodetic Datum. The boring locations and elevations, together with the estimated stratigraphical profile are shown on Drawing No. 1387301-A.

All samples were subjected to careful inspection and classification both in the field and in the laboratory. Laboratory tests were performed on selected samples to determine the engineering properties of the various soil types; namely:

- Natural Moisture Contents
- Grain-Size Distributions
- Atterberg Limits
- Unconfined Shear Strengths
- Bulk Densities

The results of the laboratory testing are plotted on Record of Borehole Sheets, and on Figs. 1 to 3.

5. SUBSOIL CONDITIONS

(5.1) General

The subsoil conditions at the site are somewhat variable. In general, it may be described that the natural deposits at this location consist of a stratum 4 to 13.5 ft. of firm to stiff clayey silt to silt followed by 11.5 to 29 ft. of very loose to compact sandy silt to silty sand. This granular deposit generally changes to compact sand and gravel immediately above the sound granite bedrock. The bedrock surface ranges between elevations 918.1 to 922.1.

In certain locations where the approaches were constructed, the natural deposit is overlain by fill material of about 15 ft. in thickness.

(5.2) Fill Material

Fill material was placed in order to construct the approaches to the present structure. The upper portion of the fill material consists of very loose sand with some silt, trace to some gravel,

and traces of organics. The thickness of this layer varies from 3.8 to 10 ft. The lower portion of the fill material is composed of a 5 to 11.2 ft. thick layer of clayey silt to silt with traces of sand and organics, and its consistency is estimated to be firm to stiff. From this, it can be inferred that the fill material was not properly compacted.

(5.3) Clayey Silt to Silt, Traces of Sand, Organics

This natural deposit was observed over the entire area except in the vicinity of the south bank of the river (B.H. No. 1). In addition, a thin cover of silty sand approximately 2.5 ft. in thickness was encountered immediately above this stratum at Borehole No. 4. Elsewhere this deposit is overlain by roadway embankment material (B.H. No's 2 and 3).

The thickness of this slightly cohesive material ranges from 4 to 13.5 ft. and its consistency is estimated to be firm to stiff.

Laboratory testing was carried out in the overall deposit (both lower portion of the fill and the natural material). The results of the Atterberg Limit tests are plotted on the Plasticity Chart (Fig. 1), where it can be seen that the material is of very low plasticity. In general, this material can be treated as a cohesive type as opposed to that of noncohesive type. The results of the grain-size distributions are presented in an envelope form (Fig. 2).

(5.4) Sandy Silt to Silty Sand

This stratum was observed in all the boreholes immediately below the clayey silt to silt deposit (B.H.'s No. 2, 3, 4) or below ground surface (B.H. No. 1). The thickness of this deposit varies from 11.5 ft. to 29 ft. and its relative density is estimated to be very loose to compact. Grain-size distributions carried out on samples within this deposit are presented in an envelope form on Fig. No. 3.

(5.5) Sand and Gravel

Below the sandy silt to silty sand deposit and immediately above the bedrock, a coarse granular deposit consisting of sand and gravel was encountered in all the boreholes carried out at this location. The thickness of this stratum ranges from 7 ft. to 12.6 ft. and its relative density is estimated to vary from compact to very dense.

(5.6) Bedrock - Granite

The overburden is underlain by granite bedrock and it was proven in Boreholes No. 1, 2 and 4 by obtaining from 2.8 ft. to 6.3 ft. of BXL size rock core samples. At Borehole No. 3, refusal to augering was considered as the surface of the bedrock. The bedrock varies from elevations 918.1 to 922.1, which indicates that the depth to bedrock ranges from 34.9 to 55.6 ft. below ground level.

The bedrock can be identified as hard granite and generally sound, as evidenced by the quality and the percentage recovery from the rock cores. However, the upper foot of the bedrock surface at Borehole No. 2 was found to be slightly weathered. Detailed descriptions of the rock cores as described by Mr. B. Glassford, Geologist for M.T.C., are presented on the Diamond Drill Record Sheet included in the Appendix.

The boundaries between the various deposits, as determined at the boring locations, are shown on the accompanying Record of Borehole Sheets. An estimated stratigraphical profile across the site, inferred from the boring data, is plotted on Dwg. No. 1387301-A. The results of the various testings carried out within the overburden are presented on the Record of Borehole Sheets and on Figs. 1 to 3 which are attached to the Appendix.

6. GROUNDWATER CONDITIONS

Groundwater level observations were carried out during the course of the field investigation by recording the water levels in the open

boreholes. The observations are plotted on the Record of Borehole Sheets and also on Dwg. No. 1387301-A. The groundwater levels obtained by these observations vary from elevations 954.3 to 958.0 and their depths from ground surface range from 2 to 19 ft. The river water level at the time of the investigation was found to be 954.1.

7. DISCUSSION AND RECOMMENDATIONS

(7.1) General

A replacement structure for the present three span bridge (24' - 90' - 24') is proposed. The horizontal alignment for the proposed structure will be maintained. However, the grade for the proposed structure will be revised to about elevation 981.5 which indicates that the vertical alignment will be 6 to 7 ft. higher than the existing one. The new bridge also will be a three span structure. At this stage, the precise details such as proposed length of spans and footing locations of the structure are not known. However, it is believed that the piers and abutments will be located in the proximity of the existing foundations.

The subsoil, bedrock and groundwater conditions encountered in this area have been discussed elsewhere in this report. An inferred stratigraphical profile along the proposed centreline is shown on Dwg. No. 1387301-A.

Recommendations for the structure foundations and the approach embankments are given in the following subsections.

(7.2) Approach Fills

The proposed grade is such that additional fills of up to 7.5 ft. will be required. Thus, the slopes in the transverse direction will be 22-23 ft. above the original ground surface (approx. elevation 959.0). However, in the longitudinal direction, maximum height of the approaches measured to the bottom of the river bed will be 37.5 to 38.5 ft.

Analyses, in terms of total stresses have been carried out, both in the longitudinal and transverse directions, to determine

the stability of the proposed fill geometry. The following data and assumptions were used:

<u>Soil Type</u>	<u>Elev. (Ft.)</u>	<u>γ (pcf)</u>	<u>γ' (pcf)</u>	<u>ϕ°</u>	<u>Cu (psf)</u>
1. Granular Type (New Fill)	982-975	130	130	30	0
2. Silty Sand to Sandy Silt (Existing Fill)	975-971	130	130	30	0
3. Clayey Silt to Silt (Existing Fill and Natural Material)	971-955	120	120	0	1,000
4. Clayey Silt to Silt (Natural Deposit)	955-943	57.6	57.6	0	700
5. Sandy Silt to Silty Sand (Natural Deposit)	943-940	67.6	67.6	30	0

Water Level - Elevation 955.0

The results indicated that the embankment constructed at the proposed grade will be stable in the transverse direction if constructed with 2:1 slopes (2 horizontal to 1 vertical). However, the longitudinal slopes will require a midheight berm (elevation 962.0) of 20 ft. width.

The new embankment should be 'keyed' into the existing one in accordance with current M.T.C. practices.

It should be noted that there will be some differential settlements within the width of the new embankment because of the configuration of the new fill and the existing embankment. In order to minimize such problems, paving should be delayed as long as possible after the placement of the new fill.

(7.3) Structure Foundations

The subsoil conditions are such that it is not practical to utilize spread footing type of support for the proposed structure. Therefore, it is recommended that the piers and abutments be supported on end-bearing piles founded on bedrock. The elevations of the surface of the bedrock could be inferred from Dwg. No. 1387301-A. The piles can be designed using the maximum

allowable load of the pile section chosen.

The construction of the pile caps at the pier locations will require a dewatering scheme to enable construction to be carried out in a relatively dry condition. The specific details can be provided once the precise location and elevation of the pier foundations are known.

If tubular piles are used, these may be extended as pier columns to the underside of the deck. This would obviate the need for a dewatering scheme which is mentioned above.

No bouldery or rock fill should be placed where piles are to be driven. Adequate frost cover for the footing bases should be provided.

Protective measures against scour due to river erosion and surface runoffs, should be provided. Recommendations pertaining to this aspect should be obtained from the Hydrology Section.

8. MISCELLANEOUS

The field work for this investigation was carried out during the period of September 8, 1975 to September 12, 1975 under the supervision of Mr. H. Shah, Project Engineer.

The equipment used for subsoil sampling was owned and operated by Master Soil Investigation Ltd.

This report was written by Mr. H. Shah and was reviewed by Mr. M. Devata, Supervising Engineer.

Harmath Shah

H. SHAH, P. Eng.
Project Engineer

M. Devata

M. DEVATA, P. Eng.
Supervising Engineer



APPENDIX

ENGINEERING SERVICES BRANCH - GEOTECHNICAL OFFICE - SOIL MECHANICS SECTION

W.P. 138-73-01

LOCATION - Sta. 95 + 34 25.7' Rt.

ORIGINAL ED BY HS

DIST. 13 HWY. 522

BORING DATE September 8 & 9, 1975

COMPILED BY NT

DATUM

BOREHOLE TYPE Hollow Stem Augers, BX CASING, BXL Core

CHECKED BY

15 $\frac{20}{10}$ 5 % STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

W.P. 138-73-01

LOCATION Sta. 95 + 76 16.0' Lt.

ORIGINATED BY HS

DIST. 13 HWY. 522

BORING DATE September 10, 1975

COMPILED BY NT

DATUM Geodetic

BOREHOLE TYPE Hollow Stem Augers, BX Casing, EXL Core

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ PCF	REMARKS % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
973.7	Ground Level															
0.0	Fill															
	Sand, some silt, trace to some gravel, trace of organics.		1	SS	2	970										6 72 (22)
	Very Loose		2	SS	2/18"											
	Clayey silt to silt, traces of sand, orgs.		3	SS	6											
958.7	Firm to Stiff		4	SS	5	960										0 1 86 1
15.0	Clayey silt to silt, traces of sand, orgs.		5	TW	PH											Org. 0.26%
954.7	Firm															
19.0	Sandy silt to silty sand.		6	SS	4											
	Very Loose to Compact		7	SS	1	950										0 53 (47)
			8	SS	8											
			9	SS	14	940										0 79 (21)
			10	SS	7											0 42 (58)
930.7						930										
43.0	Sand and Gravel		11	SS	19											
	Compact		12	SS	20											
918.1	Dense					920										
55.6	Granite weathered sound		13	RC	91%											
915.3	Bedrock															
58.4	End of Borehole					910										

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

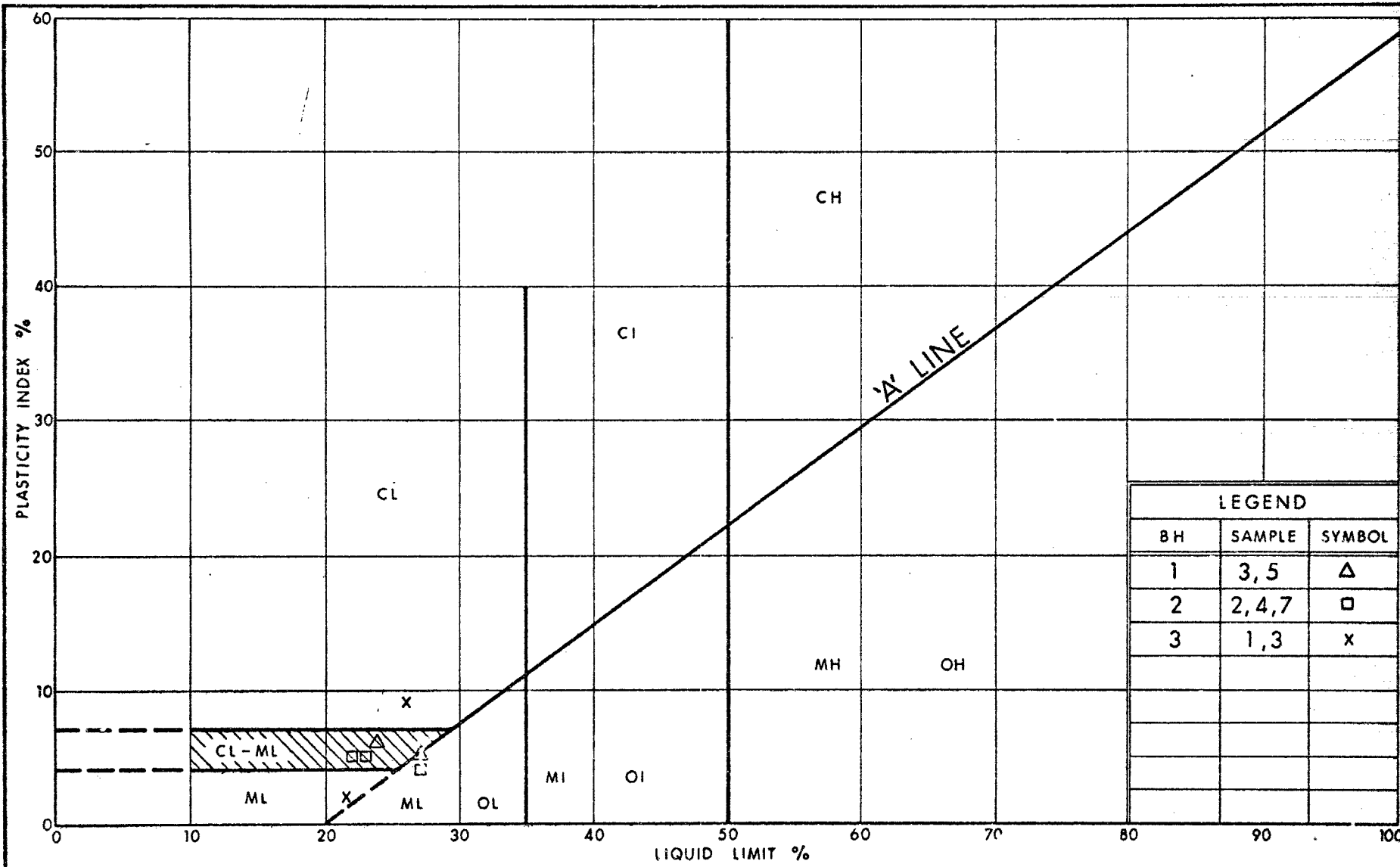
WP 138-73-01 LOCATION Sta. 94 + 16 18.5' Rt. ORIGINATED BY HS
DIST 13 HWY 522 BORING DATE September 11, 1975 COMPILED BY NT
DATUM Geodetic BOREHOLE TYPE Hollow Stem Augers CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ PCF	REMARKS % GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	w_p	w	w_L		
974.0	Ground Level															
0.0	Fill Sand, some silt, trace to some orgs. Very loose		1	SS	4	970										
	Clayey silt to silt, traces of sand, orgs.		2	SS	6											Org. 0.13%
	Firm to Stiff		3	TW	PH											
959.0			4	SS	14	960										Org. 0.39% 0 1 88 11
15.0	Clayey silt to silt, traces of sand, orgs.		5	SS	7											
	Firm to Stiff		6	SS	3											Org. 0.32%
			7	SS	6											
945.5			8	SS	13	950										0 2 89 9
28.5	Sandy silt to silty sand, traces of organics, gravel.		9	SS	4											Org. 0.07% 0 67 (33)
	Loose to Compact		10	SS	13	940										
932.5			11	SS	17											2 13 (85)
41.5	Sand and gravel		12	SS	17	930										
	Compact Very Dense		13	SS	70											
920.5			14	SS	124	920										
53.5	End of Borehole Refusal to auger Probable Bedrock															

RECORD OF BOREHOLE NO 4

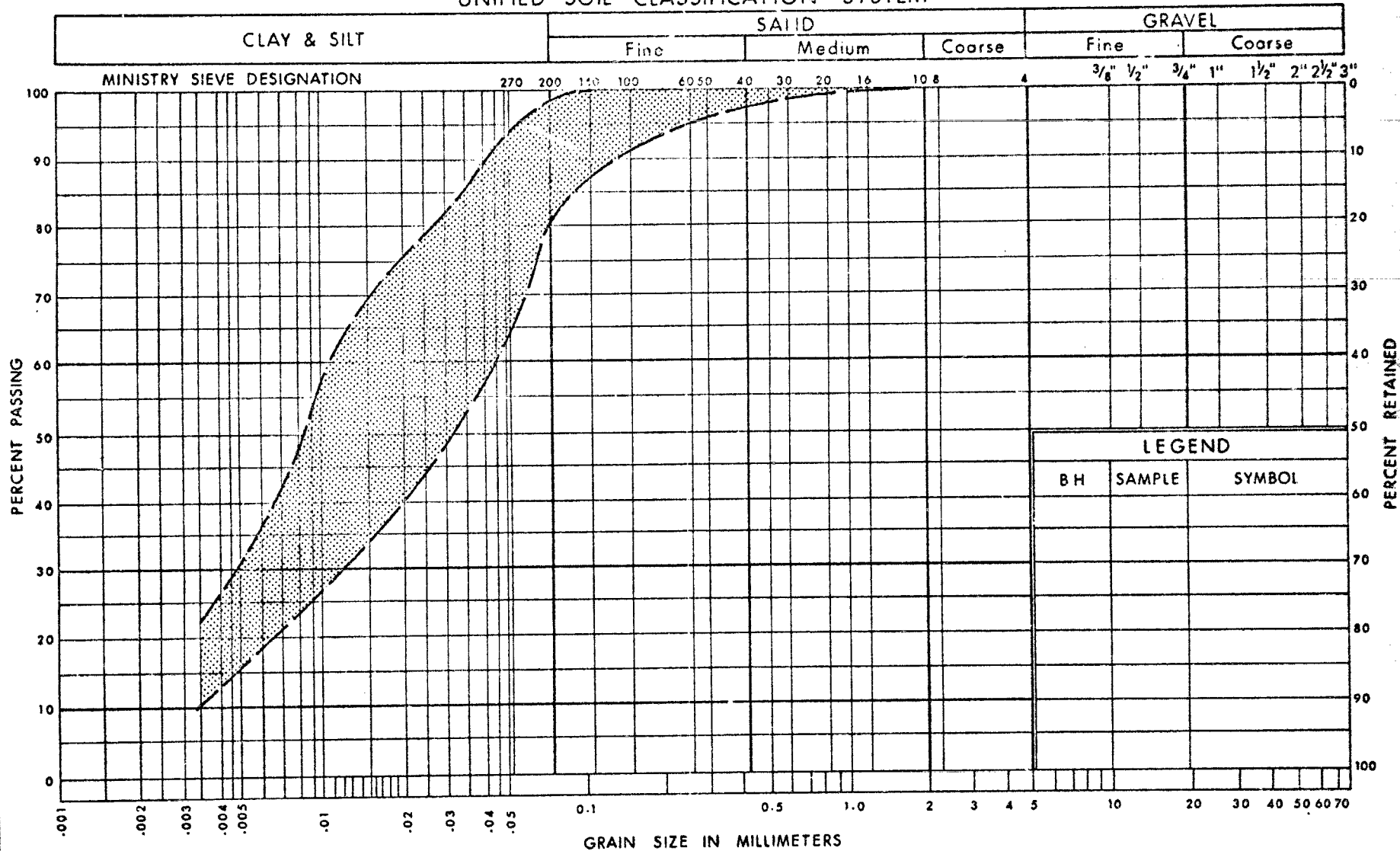
WP 138-73-01 LOCATION Sta. 94 + 60 39.5' Lt
 DIST 13 HWY 522 BORING DATE September 12, 1975
 DATUM Geodetic BOREHOLE TYPE Hollow Stem Augers, BX Casing, BXL Core
 ORIGINATED BY HS
 COMPILED BY NT
 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
957.0	Ground Level															
954.5	Silty sand. Very Loose		1	SS	3											
2.5	Clayey silt to silt org. silt, some sand trace of sand, orgs.		2	SS	2											
			3	SS	8											
941.5	Firm															
15.5	Sandy silt to silty sand.		4	SS	10											
			5	SS	8											
930.0	Loose		6	SS	10											
27.0	Sand and gravel. Compact to Dense		7	SS	15											
922.1	Granite Bedrock		8	BXL	100											
917.2	Sound		9	BXL	93%											
39.8	End of Borehole															



LEGEND		
BH	SAMPLE	SYMBOL
1	3, 5	△
2	2, 4, 7	□
3	1, 3	x

UNIFIED SOIL CLASSIFICATION SYSTEM



**Ministry of
Transportation and
Communications**

Ontario

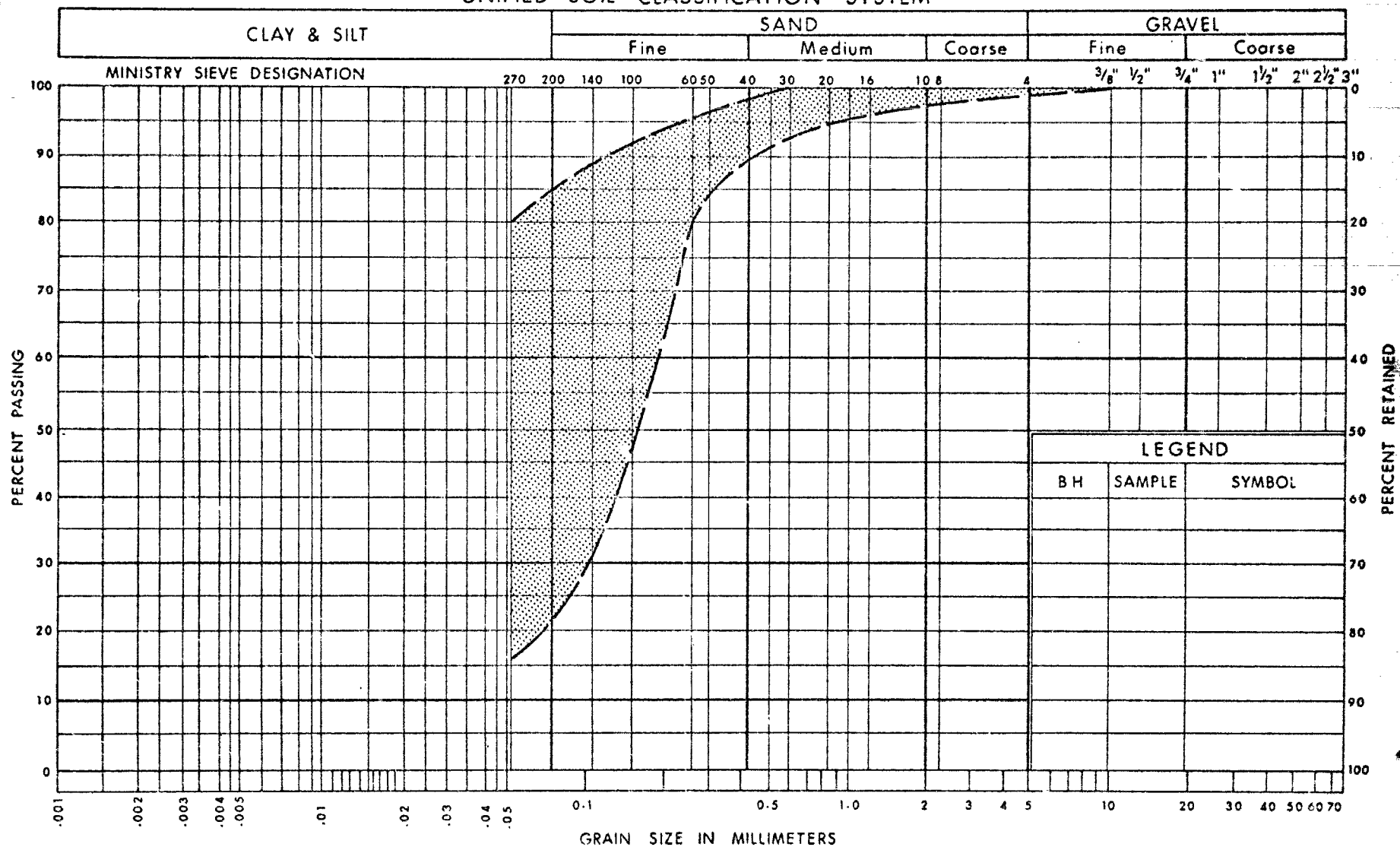
ENGINEERING SERVICES BRANCH

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILT
TRACE OF SAND

FIG No 2

W P 138-73-01

UNIFIED SOIL CLASSIFICATION SYSTEM



ENGINEERING SERVICES BRANCH

Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SANDY SILT TO SILTY SAND

FIG No	3
--------	---

W F 138 - 73 - 01



Ministry of
Transportation
and
Communications

DIAMOND DRILL RECORD

HOLE NO. _____ SHEET NO. _____

DIP

90°

PROPERTY _____ W.P. 138-73-01 _____
LOCATION _____

LATITUDE _____
DEPARTURE _____
BEARING _____

TOTAL FOOTAGE 39'10" HOLE # 4

ELEV. COLLAR _____
 DATUM _____
 DATE STARTED _____
 DATE COMPLETED _____
 DRILLED BY _____
 LOGGED BY _____

FOOTAGE		FORMATION	SAMPLE NUMBER			REMARKS
FROM	TO					
		Hole No. 1				
36'	42'4"	Granite pinkish, medium grained, hard with large hornblende crystals				39'6" - vertical joint
		Hole No. 2				
55'6"	58'5"	Granite pinkish, medium grained, hard with large hornblende crystals. Weathered zones 55'6" to 56'6"; 58'0" to 58'3".				
		Hole No. 4				
34'11"	39'10"	Granite, pinkish, medium grained, hard with large hornblende crystals. First 2" slightly weathered.				

DATE OF EXAMINATION September 22, 1975

Z. Koniuszy

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTPENETRATION RESISTANCE

'N'-STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>c LB./SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS :-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CIU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" " ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTSOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
w_s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

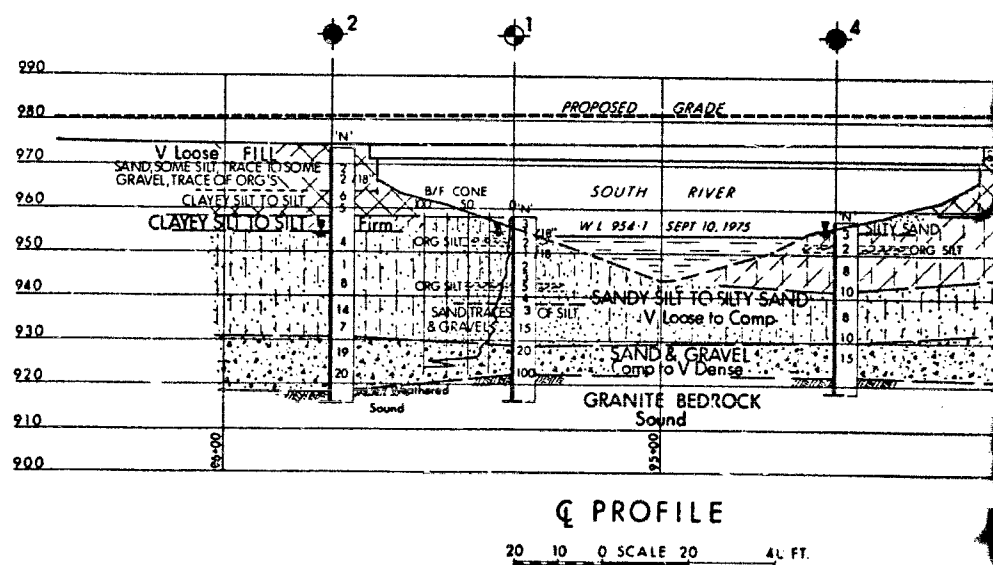
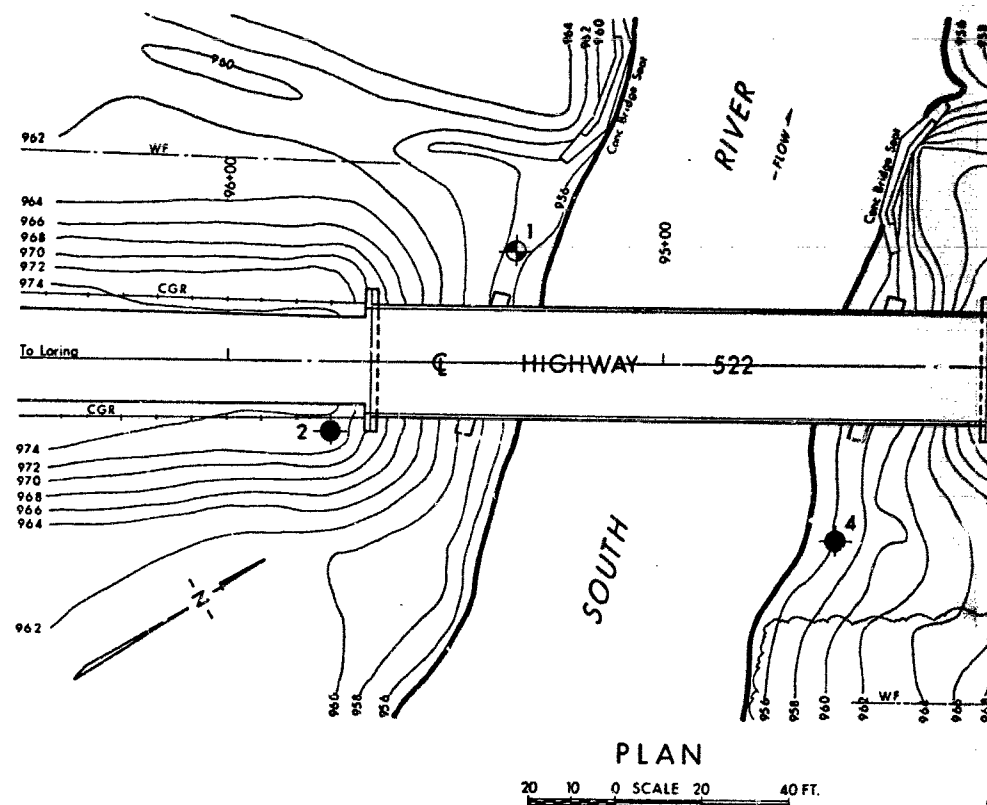
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

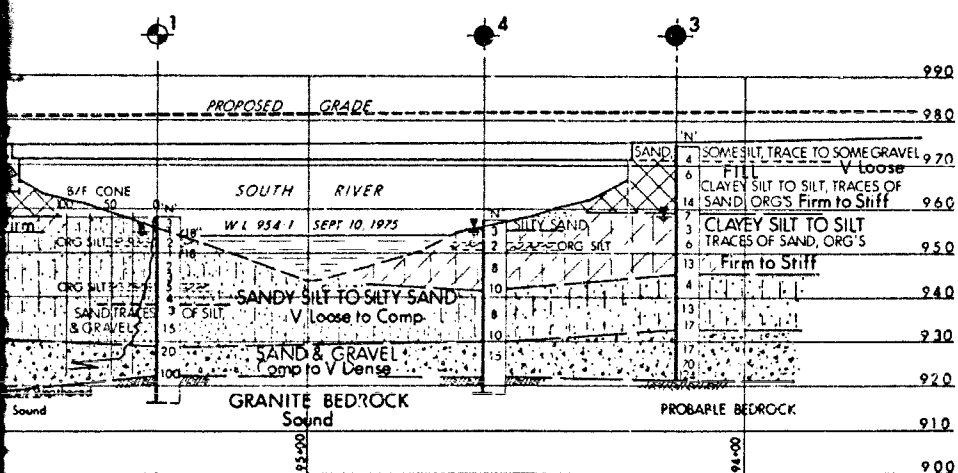
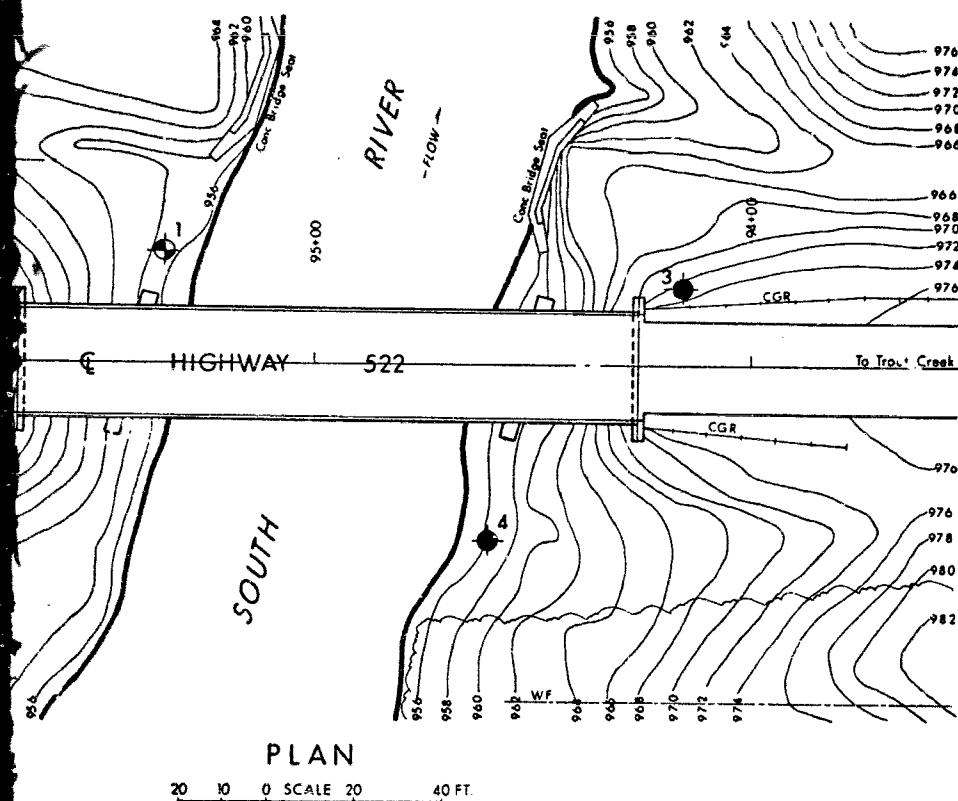
FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

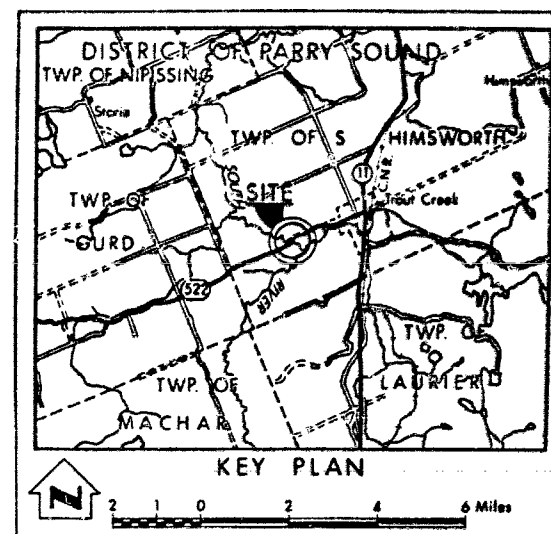
SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL





Q PROFILE



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Resistance Test
B/F CONE - Blow/Ft. Cone Test (350 Ft. lbs. energy/blow)
- ⊕ Bore Hole & Cone Test
- ⊕ Water Levels established at time of field investigation, SEPT. 1975

NO.	ELEVATION	STATION	OFFSET
1	958.0	95+34	25.7 RT
2	973.7	95+76	14.0 LT
3	974.0	94+16	18.5 RT
4	957.0	94+60	39.5 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 FOR CONTRACT DOCUMENT

The complete foundation investigation report for this structure may be examined at the Structural Office and Foundations Office, Downsview, and at the NORTH BAY District Office.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
ENGINEERING SERVICES BRANCH—GEOTECHNICAL OFFICE—SOIL MECHANICS SECTION

SOUTH RIVER

HIGHWAY NO. 522 DIST. NO. 13
DIST. PARRY SOUND
TWP. SOUTH HIMSWORTH LOT 31 & 32 CON. 3

BORE HOLE LOCATIONS & SOIL STRATA

SUBMD H.S.	CHECKED H.S.	WP NO. 138-73-01	DRAWING NO.
DRAWN Sach	CHECKED Sach	WO NO.	1387301-A
DATE 13 JAN 1976	SITE NO. 44-SJ	BIDGE DRAWING NO.	
APPROVED	CONT NO.		



Memorandum

To: Mr. C. Mirza,
Manager,
Geotechnical Office,
Downsview.

From: Structural Planning,
Northern Region,
North Bay.

Attention:

Date: 20 August 1975.

Our File Ref.

In Reply to

Subject:

W.P. 138-73-01
Site 44-50
South River Bridge
Hwy. 522, District 13



Enclosed are plans, profiles, existing bridge drawings and a field reconnaissance report for the above project. Site plans will not be available for some time.

The proposed structure is to be built in the same location as the existing bridge with pier and abutment locations as shown on the attached sketch.

Please arrange to carry out a foundation investigation of this site.

As yet, there are no schedule dates established for this project.

C. Verhulst
C. VERHULST,
FOR:
J. C. McALLISTER,
REGIONAL STRUCTURAL
PLANNING SUPERVISOR.

CV:jem
Encl.
c.c. R. Murphy

Soil Mechanics Section
Geotechnical Office
West Building
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

Tel: (416) 248-3282

September 8, 1975

Master Soil Investigation Limited
104 Kenhar Drive
Weston, Ontario
M9L 1N4

Dear Sirs:

This letter confirms our request by telephone of September 4, 1975 for the supply of a Type III Auger Machine (M.V. mounted) (Item No. 5.3, 1), together with all necessary equipment, as per your Tender for Supply Contract, S-74-2685, at Highway 522 and South River crossing on September 8, 1975.

Mobilization will be from North Bay, Ontario.


Our Project Number is W.P. 138-73-01.

Yours truly,

M. Devata
Supervising Engineer

c.c. W.W. Fry
(Attn: V. Di Marco)

Files (2) ✓
Record Services





Memorandum

To: Mr. C. Mirza, Head,
Soil Mechanics Section,
West Bldg., Downsview.

From: Structural Planning,
Northern Region,
North Bay.

Attention: H. Shah

Date: 11 September 1975.

Our File Ref.

In Reply to

Subject:

W.P. 138-73-01, Site 44-50
South River Bridge
Highway 522, District 13

Enclosed is the available structural maintenance history for the above structure.

I checked into the other structure installations in the area and found that no formal foundation investigations were carried out at the two sites that I had in mind. The nearest site then would probably be the South River bridge investigation at Nipissing Village, further downstream.

It appears that the gabions were installed during 1970 and have been sliding forward at the east abutment ever since.

C. Verhulst
C. VERHULST,
FOR:
J. C. McALLISTER,
REGIONAL STRUCTURAL
PLANNING SUPERVISOR.

CV:jem
Encl.





Memorandum

To: Mr. C. Mirza, Head,
Geotechnical Office,
West Bldg., Downsview.

From: Structural Section,
Northern Region,
North Bay.

Attention: H. Shah

Date: 12 December 1975.

Our File Ref.

In Reply to

Subject:

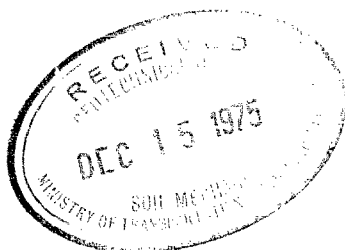
W.P. 138-73-01, Site 44-50
South River Bridge
Hwy. 522, District 13

Enclosed for your use are two copies of site plan E-5058-1
for the above project.

This plan has not been officially issued; however, no changes
that will affect your section are expected to be made.

C. VERHULST,
FOR:
J. C. McALLISTER,
HEAD,
STRUCTURAL SECTION.

CV:jem
Encl.





Memorandum

To: Mr. J. McAllister,
Head, Structural Section,
Northern Region, North Bay.

From: Structural Office,
West Building, Downsview.

Attention:

Date: May 4, 1976.

Our File Ref.

In Reply to

Subject:

South River Bridge,
W. P. 138-73-01, Site 44-50
District #13.

Attached herewith are prints of the Preliminary Bridge
Plan Drawing 44-50-P1 for the above mentioned structure.

The estimated cost of the proposed structure is \$252,000.00
which includes tender, materials, engineering and sundry
construction.

We have sent a copy of the Preliminary Plan to the Hydrology
Office for their comments.

Any comments or revisions you may have should be submitted
at your earliest convenience.

CSG/cf
Attch.

C. S. Grebski,
Structural Design Engineer.

c.c. R. Dorton
A. McKim
W. Birch
A. Radkowski
M. Stoyanoff
J. Harris
C. Mirza
J. Anderson
R. Murphy
S. Edwards

No comments.
M. Dorton
H. Shah





Ministry of
Transportation and
Communications

Memorandum

To: Mr. C. Mirza,
Head, Soil Mechanics Section,
West Building, Downsview.

From: Structural Office,
West Building, Downsview.

Attention:

Date: September 15, 1976.

Our File Ref.

In Reply to

Subject:

South River Bridge,
W.P. # 138-73-01 Site # 44-50
Highway # 522 District # 13

Attached herewith we are submitting the final bridge
drawings which show the foundation design for this structure.
Kindly give us your comments at your earliest convenience.

CSG/cf
Attch.

C. S. Grebski
C. S. Grebski,
Structural Design Engineer.

*Received on Sept 27/76
by MA*

Forwarded to Sept 76

Refer to memo dated Oct. 12/76



728



Memorandum

To: Mr. S. McCombie,
Regional Manager,
Planning & Design Office,
Northern Region, North Bay.

From: Structural Office,
West Building, Downsview.

Attention:

Date: September 22, 1976.

Our File Ref.

In Reply to

Subject: W.P. 138-73-01, Site 44-50,
South River Bridge,
Highway 522, District 13.

Please find enclosed four sets of prints of drawings 44-50-1 and 3 to -13 inclusive for your use.

Two sets of prints are being forwarded to the Construction Office, Northern Region.

One print of drawing 44-50-1 is being forwarded to the Systems Design Project Review Section.

One set of prints is also being forwarded to the following:

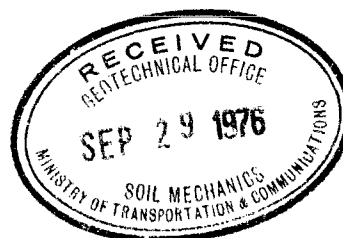
Estimating Section
Regional Structural Planning Engineer
Assistant Contruction Engineer (Structures)
Structural Maintenance Engineer
Soil Mechanics Section
Hydrology Section

The D4 and Special Provisions will follow.

NZ/cf
Encl.

N. Zoltay,
Structural Contract
Specifications Engineer.

c.c. J. Wear
B. Giroux
J. McAllister
A. McKim
M. Bernhardt
E. Van Beilen
C. Mirza
J. Harris
R. Murphy
J. Anderson



Mr. C.S. Grebski
Structural Design Engineer
Structural Design Section
West Building, Downsview

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

October 13, 1976

South River Bridge
W.P. 138-73-01, Site 44-50
District #13, North Bay

We have reviewed the final bridge drawings, Sheets 45-50-1 and 3
and have no further comments.

H. Shah
Project Engineer

For: M. Devata
Supervising Engineer

MD/HS/gs

cc: Files
Record Services



Memorandum

To: Mr S. McCombie
Regional Manager
Reg. Planning & Design
Northern Region North Bay

From: Structural Office
West Building, Downsview,

Attention: Date: 77 05 02

Our File Ref. In Reply to

Subject: South River Bridge
1.9 miles west of Hwy #11
W.P. 138-73-01, site 44-50
Hwy #522, Dist 13

The steel reinforcement in the slab of the above structure has been modified. The D4 has to be revised as follows:

"905 - Reinforcing Steel (Bridge)"

45 tons should be changed to read 33 tons.

P.O. Law
Structural Project Engineer
for A. Radkowski
Regional Structural Design Engineer

c.c. N. Zoltay
J. Wear
J. McAllister
M. Bernhardt
A. McKim
B. Giroux
E. Van Beilen
C. Mirza✓

