

65-F-247

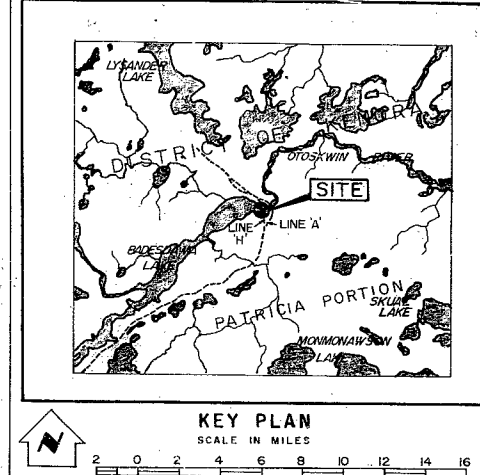
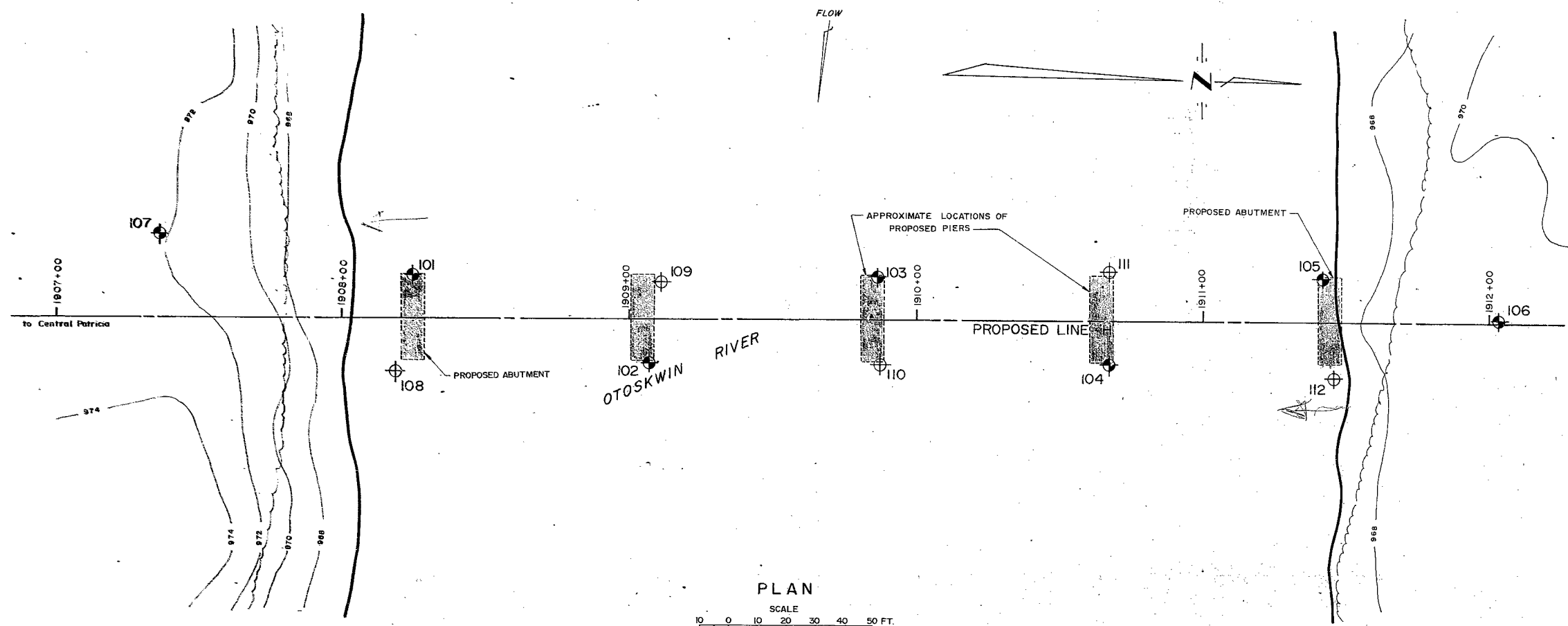
W.P. # 132-66

RESOURCES RD.

LINE 'H'

OTOSKWIN

RIVER CROSSING



LEGEND

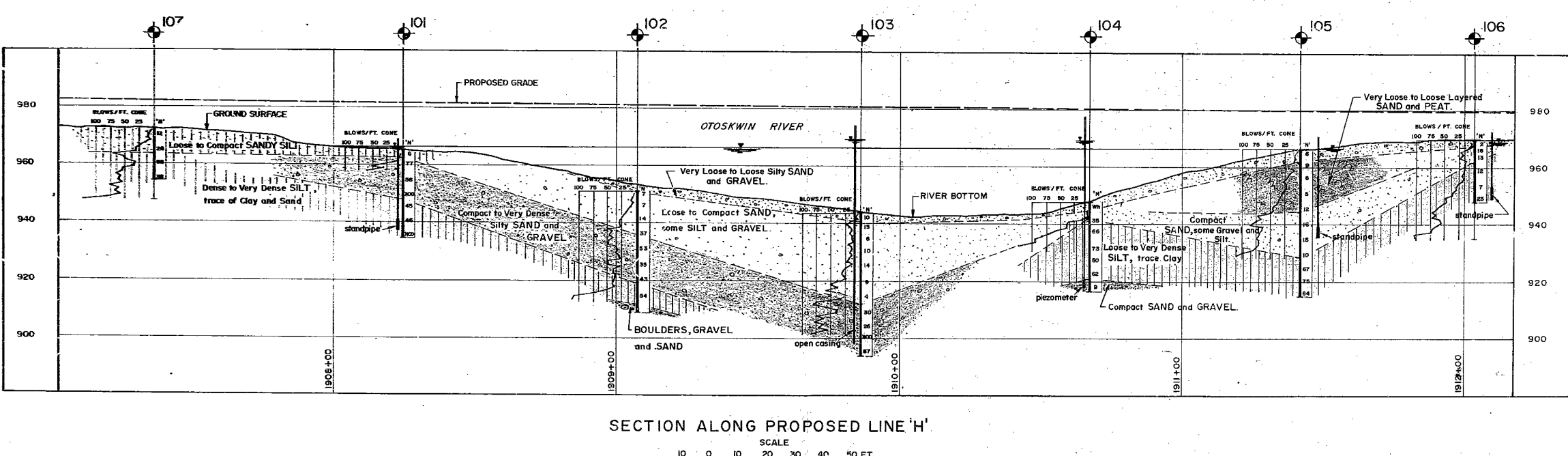
- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation.

NO.	ELEVATION	STATION	OFFSET
101	965.0	1908+25	15' LEFT
102	965.1	1909+07	15' RIGHT
103	965.8	1909+86	15' LEFT
104	965.8	1910+67	15' RIGHT
105	966.7	1911+42	15' LEFT
106	969.7	1912+03	1' LEFT
107	972.4	1907+36	25' LEFT
108	966.0	1908+19	16' RIGHT
109	965.1	1909+11	13' LEFT
110	965.8	1909+87	15' RIGHT
111	965.8	1910+67	17' LEFT
112	966.7	1911+46	19' RIGHT

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

PRINT RECORD

NO.	FOR	DATE



REVISIONS		DESCRIPTION	
DATE	BY		
H Q GOLDER AND ASSOCIATES LIMITED			
DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & TESTING DIVISION - FOUNDATION SECTION			
OTOSKWIN RIVER			
PROPOSED RESOURCES ROAD, LINE 'H'		DIST. NO. 20	
DISTRICT OF KENORA, PATRICIA PORTION			
"UNSURVEYED TERRITORY"			
BORING PLAN AND SOIL STRATIGRAPHY SECTION			
SUBM'D	CHECKED	W.P. NO. 132-66	M.T. DRAWING NO.
DRAWN M.W.	CHECKED F.J.H.	JOB NO. 65087-1	
DATE OCT. 7, 1965	SITE NO.	BRIDGE DRAWING NO.	
APPROVED <i>[Signature]</i>		CONT. NO.	

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

**H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN
J. L. SEYCHUK**

**2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201**

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED OTOSKWIN RIVER CROSSING

LINE "H"

NORTH OF CENTRAL PATRICIA, ONTARIO

Distribution:

- 12 copies - Department of Highways, Ontario,
Toronto, Ontario.**
- 2 copies - H. Q. Golder & Associates Ltd.,
Toronto, Ontario.**

October, 1965

65087-1

TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	1
INTRODUCTION	2
PROCEDURE	2
SITE AND GEOLOGY	3
SOIL CONDITIONS	5
GROUNDWATER CONDITIONS	8
DISCUSSION	9
General	9
Foundations	9
Approach Embankments	13
ABBREVIATIONS	In Order
RECORDS OF BOREHOLES	Following
	Page 15.
FIGURES 1	- Boring Plan and Soil Stratigraphy Section
2-7	- Grain Size Distribution Curves

ABSTRACT

The results of an investigation to determine the subsoil conditions at the site of a proposed bridge crossing over Otoskwin River north of Pickle Crow, Ontario are reported and recommendations are made for the foundation design of the proposed structure and approach embankments.

It was found that the site is underlain by a complex succession of granular deposits comprised of silts to sands and gravels. At depth the subsoil is generally a dense silt. On the river banks the dense silt is overlain by surficial deposits of peat and sand or sandy silt. In the river area, the silt has been eroded to various depths and sands and gravels have been subsequently deposited. On the south side of the river the silt is overlain by compact to very dense silty sand and gravel some 15 feet thick and then by loose to compact sand which ranges in thickness from a few feet near the south bank to 27 feet at the central pier. On the north side of the river the dense silt is overlain by 5 to 15 feet of compact sand and also at the north shore location by 16 feet of loose layered sand and peat. A slight artesian head, about 4 feet above river level, was encountered at depth in two of the borings in the river.

It is recommended that the piers and abutments of the proposed bridge structure be founded on either timber or steel pipe displacement piles. The piers and abutments may also be founded on spread footings placed within the granular subsoil. This will necessitate the provision of sheet piled cofferdams during the construction of the pier and abutment footings for groundwater control as discussed in the report.

There should be no overall stability problem with approach embankments, having 2 horizontal to 1 vertical side slopes, if the surficial peat covering is removed and the slopes are protected against erosion scour.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario, to carry out a subsurface investigation at the site of a proposed bridge crossing for the line "H" alignment of the Resources Road over the Otokwin River in the District of Kenora, Ontario. The purpose of this investigation was to determine the subsurface conditions across the site and to make recommendations for the foundation design of the proposed structure and roadway approach embankments.

PROCEDURE

The field work for this investigation was carried out between September 5 and September 29, 1965. During this period 7 boreholes (numbered 101 to 107, inclusive) with accompanying dynamic penetration tests and 5 additional dynamic penetration tests (numbered 108 to 112, inclusive) were put down to a maximum depth of about 70 feet. The borings were carried out using a skid-mounted machine drillrig supplied and operated by Boyles Bros. Drilling (Eastern) Ltd. of Port Arthur, Ontario. For the borings put down in the river, a raft supplied by the drilling contractor was used to mount the drillrig. The field work was supervised throughout by an engineer from our staff.

The locations of the borings put down during this investigation are shown on Figure 1. A detailed record of each borehole and dynamic penetration test is presented on the Records of Boreholes following the text of this report. A section of the inferred soil stratigraphy along the centreline of the alignment "H" crossing is given on Figure 1.

The samples obtained during the investigation were shipped to our Toronto laboratory for detailed examination and testing. The results of the laboratory testing are presented on the Records of Boreholes and on Figures 2 to 7.

The elevations of the boreholes given in this report are referred to an assumed datum established for the Resources Road and the elevations were obtained from a benchmark consisting of a nail in the top of a 0.4 foot spruce stump located 76 feet left of station 1907+50. The elevation of this benchmark was given as 974.07. The borehole locations and elevations given in this report were established in the field by our engineer.

SITE AND GEOLOGY

The site of the proposed bridge for the line "H" alignment of the Resources Road over the Otoskwin River is located some

30 miles north-east of Central Patricia in the Patricia Portion of the District of Kenora, Ontario. The Otokwin River flows east and narrows from Badesdawa Lake, immediately west of the site, to a channel some 320 feet wide and up to about 22 feet deep at the proposed crossing location. The site on either side of the river is generally covered by bush and is fairly level at an elevation some 3 feet above the river level. At the time of the investigation the Resources Road route was cleared to the south bank of the river.

From available geological information for this general vicinity it is known that on the retreat towards the north-east of the last continental glacier the area was inundated by a glacial lake which mantled the bedrock and overlying glacial till with sands, silts and clays.

A large end moraine which is oriented in a north-west and south-east direction is located some 20 miles north-east of the proposed river crossing. Melt water streams on the southern side of this moraine have deposited vast quantities of silt and very fine sand. Glacial features such as drumlins and eskers which trend in a north-easterly direction are numerous in the area.

The broad lowland around the northern end of Badesdawa Lake is mantled with silt. Varved clay sections are also exposed in

this area. Bedrock seldom outcrops north-east of Pickle Crow. The bedrock is understood to be part of the Canadian Shield and of Precambrian Age.

SOIL CONDITIONS

The detailed stratigraphy encountered in each boring is given on the Records of Boreholes. Following is a summary account of the inferred subsoil conditions at the site.

The central portion of the river bed is underlain by very loose to loose silty sand and gravel some 5 feet thick which has the appearance of recent alluvium. In addition some scattered boulders lie on the river bed surface. Grading curves for samples of this river bed deposit are shown on Figure 2.

Both river banks are only a few feet above normal river level and as such these banks are inundated during periods of flooding. As expected the soil on both banks appears to be a flood-plain deposit. The surface stratum at the north bank beneath a few feet of silty sand and gravel is a layered complex succession of silty sand with thin organic layers extending to a depth of 22 feet below ground surface at the location of borehole 105. The organic layers consist generally of fibrous peat and are up to 1/4 inch

thick and at $\frac{1}{2}$ to 1 inch spacing. Grading curves for samples from this deposit are shown on Figure 3. Standard penetration tests carried out in the layered sand and peat gave "N" values of 2 to 12 blows/ft. Based on these values and the results of the dynamic penetration tests, the relative density of the layered sand and peat is considered to be very loose to loose.

On the southern bank, the ground surface is underlain by a thin organic covering and then by up to 7 feet of loose to compact sandy silt. Grading results for samples of the sandy silt are shown on Figure 4.

The loose river bed deposit of sand and gravel and the layered sand and peat on the north bank of the river is underlain by a sand stratum which was found to be some 27 feet thick at the central pier location where the sand is well graded in nature, containing some gravel and a trace of silt. Elsewhere the sand is generally uniform in grading consisting predominantly of fine sand sizes with a trace of silt. Grading curves for samples of the sand are shown on Figure 5. Standard penetration tests carried out in the sand stratum gave "N" values ranging from 4 to 18 blows/ft. with an average of 12 blows/ft. Based on these values and the results of the dynamic penetration tests, the relative density of the sand stratum is considered to be loose to compact.

In boreholes 101, 102 and 103 on the southern side of the river, a stratum of silty sand and gravel 15 to 18 feet thick underlies the loose to compact sand. The material comprising this stratum is well graded consisting of gravel, fine to coarse sand sizes and some silt size particles. This well graded nature is illustrated on Figure 6, where the results of grain size distribution tests on representative samples of the silty sand and gravel stratum are shown. Standard penetration tests carried out in the stratum gave "N" values ranging from 26 to greater than 100 blows/ft. Based on these values and the results of the dynamic penetration tests, the relative density of the silty sand and gravel is considered to range from compact to very dense and to be generally dense.

A silt stratum, which is believed to be the principal stratum in the land area where it is overlain by a few feet of sand and sandy silt on either bank, was encountered below the above mentioned dense silty sand and gravel stratum and the loose to compact sand stratum below the river. The maximum thickness of the silt stratum where it was fully penetrated in borehole 104 is 21 feet. Typical grading curves for the silt, as shown on Figure 7, illustrate the uniform gradation characteristics of the stratum. Standard penetration tests carried out in the silt gave "N" values generally in excess of 40 blows/ft., with the exception of borehole 106 where

values of 12 and 7 blows/ft. were obtained in the upper portion of the silt stratum above elevation 950. Outside of this localized zone the relative density of the silt stratum is generally dense to very dense.

With depth in boreholes 102 and 104, the silt stratum is underlain by a deposit of sand, gravel and boulders which may be glacial till.

GROUNDWATER CONDITIONS

Standpipes were installed in three of the borings (boreholes 101, 105 and 106) which were either in shallow water or on land. A sealed piezometer was installed in borehole 104 after a slight artesian pressure was noticed at depth during the boring operations. Water levels were taken in these installations during the period of the field work. The installation details and the water level readings obtained are given on the Records of Boreholes.

The standpipes placed in the silt or the overlying sand gave water level readings at about existing river level. The piezometer sealed in the lower sand and gravel layer below the silt deposit in borehole 104 indicated an artesian water pressure some 3.5 feet above present river level. An artesian pressure was also

measured in the casing at borehole 103. The water rose in the casing some 3.5 feet above river level during drilling operations.

DISCUSSION

General

It is understood that the Otoskwin River bridge is to be a four span structure, with each span about 80 feet long. The abutments, which will be located at the river edge, will probably be of the retaining type. No other structural details of the proposed bridge are available at this time but the bridge will probably be a reinforced concrete structure. The proposed highway grade is at elevation 980, that is some 14 feet above present river level, necessitating roadway approach embankments about 12 feet high.

Foundations

This investigation has disclosed that the river bed is underlain by essentially granular deposits which are loose to compact near surface and dense with depth. It is recommended that a piled foundation be employed at this site due to the more favourable foundation conditions at depth and considering the relative ease of this type of construction in the river. It is considered that a driven displacement pile would be the most suitable pile type

for the subsoil conditions at the site. The selection of the type of displacement pile to be used would depend on economic factors such as the shipping costs to this remote area and the availability of suitable pile driving plant.

It is expected that the choice would be made finally between timber piles or steel pipe piles driven closed end and filled with concrete. Due to the essentially granular nature of the subsoil a reliable pile driving formula should provide a useful guide for determining the final set the piles should be driven to for a given design load. Taking a timber pile, with a 12 inch butt and 8 inch tip diameter, driven to a final set of about 5 blows/inch with a hammer having a rated energy of about 12,000 ft.lb./blow, a design load of 20 tons/pile may be obtained. It is estimated that the timber piles would penetrate into the upper portion of the compact to very dense sand and gravel stratum in the southern and central sections of the bridge site and generally into the silt deposit on the north side. Thus the length of timber piles required below existing ground surface would vary between about 15 feet at borehole 101 to about 35 feet at borehole 103.

For 12 inch diameter pipe piles driven closed end to a final set of 20 blows/inch with a hammer developing 20,000 ft.lb.

of energy/blow, the design load may be taken as 70 tons/pile. The penetration required for pipe piles to obtain the required driving resistance would in general be some 5 to 10 feet greater than for the timber piles.

An alternate foundation solution for piers and abutments would be the use of spread footings founded in the subsoil at a depth of about 5 feet below river bottom, except for the north abutment which should be carried down through the layered sand and peat to the compact sand stratum at about elevation 945. An allowable bearing value of up to 2 tons/sq.ft. may be taken for the compact sand at the south and central pier and north abutment locations. For the dense sand and gravel at the south abutment location and the dense silt at the north pier, the allowable bearing value may be increased to 4 tons/sq.ft. Provided that the in situ density of the subsoil at and below foundation grade is maintained, the total settlement of the footings imposing the above bearing pressures should be within about 1 to 2 inches and the differential settlement between individual piers and abutments should be less than 1 inch.

If a spread footing foundation is to be used for the support of the bridge structure the most important construction

problem at this site will be the proper control of the water in the granular subsoil. Excavations some 25 feet below the river water level will be required for the footings. Therefore groundwater and river water control is essential to ensure that the base of footing excavations do not experience "piping" resulting in loosening of the subsoil below foundation grade with a consequent significant decrease in the supporting capacity of the subsoil.

Control of groundwater seepage forces into the foundation excavations may be obtained by the construction of a interlocking steel sheet piled cofferdam around the perimeter of each pier and abutment location. The sheet piling should be driven below foundation level to a depth equal to about 0.5 times the excess head of water measured from the final excavation bottom to the anticipated highest river level during construction. The necessary penetration for the sheet piling would be difficult to obtain in the dense silt underlying the north pier location.

If spread footings are employed some scour protection will be required. In this regard the sheet piling used in the pier and abutment cofferdams could be cut off at river bed or existing ground surface level and the penetrated portion left in place.

In computation of sliding resistance between a rough concrete footing base and the undisturbed sand or sand and gravel subsoil, a coefficient of friction of 0.45 and a factor of safety of at least 1.5 may be used.

It is recommended that free draining and non-frost-susceptible granular backfill be provided behind the bridge abutments. The granular backfill should be compacted in horizontal lifts of about 9 inches and should extend horizontally from the back face of the abutment walls a minimum distance of 7 feet. Provision for drainage from this material should be made. With full effective drainage behind the walls it is recommended that a coefficient of earth pressure at rest, $K_c = 0.4$, and a total unit weight, γ , of 135 lb/cu.ft. be used for the compacted granular backfill in design of the walls. If some movement of the top of the abutment "retaining" walls can be tolerated an active earth pressure coefficient, $K_A = 0.3$, may be used.

Approach Embankments

At the south approach the access roadway fill and the underlying surficial peat cover should be removed from below the proposed embankment area. The south approach embankment could be constructed directly on the granular subsoil using 2 horizontal to

1 vertical side slopes, provided that suitable fill properly compacted in place is used in the embankment.

At borehole 105, at the southern limit of the north approach embankment, a 16 foot thick deposit of sand containing thin layers of organic material was encountered below surficial deposits of peat and loose sand and gravel. The organic material is essentially a fibrous peat present in the sand deposit in the form of layers up to about 1/4 inch thick at about 1/2 to 1 inch spacing. This deposit was not encountered in borehole 106, some 60 feet north of borehole 105. Based on the penetration resistance obtained and on detailed examination of the samples recovered from this deposit, which indicate that the material is essentially granular in nature with "N" values between 5 and 12 blows/ft., it is considered that the embankment may be constructed to the proposed grade of elevation 980 or some 13 feet above existing ground surface after removal of the surficial covering of peat. As for the south approach embankment, a 2 horizontal to 1 vertical side slope should be employed. Settlement of the north approach embankment could be of the order of 6 inches. As the peat layers are thin most of this settlement should take place during construction.

Rip-rap should be placed over the side slopes of the

approach embankments to at least 3 feet above the high water level in order to prevent erosion scour and undermining of the embankments. Above the rip-rap, the embankment slopes should be sodded or seeded and mulched to minimize surface water erosion and gully-ing.



F. J. Heffernan

F. J. Heffernan, P.Eng.

J. L. Seychuk

J. L. Seychuk, P.Eng.

FJH:HDG
65087-1
October 25, 1965.

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

<i>WH</i>	sampler advanced by static weight—weight, hammer
<i>PH</i>	sampler advanced by pressure—pressure, hydraulic
<i>PM</i>	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

<i>Consistency</i>	<i>c_u, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. 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

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	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

III. SOIL PROPERTIES

(a) Unit weight

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_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_C	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density $= (e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

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

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change $= -\Delta e / (1 + e) \Delta \sigma'$
C_c	compression index $= -\Delta e / \Delta \log_{10} \sigma'$
c_c	coefficient of consolidation
T_v	time factor $= c_v t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

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_i	sensitivity

in terms of effective stress
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress
 $\tau_f = c_u + \sigma \tan \phi_u$

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 101

LOCATION See Figure 1

BORING DATE SEPT. 24-26, 1965

DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX - BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE		COEFFICIENT OF PERMEABILITY k , CM./SEC.		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FT.		BLOWS/FT.	SHEAR STRENGTH C_u , LB./SQ. FT.	WATER CONTENT, PERCENT Wp W Wl			
966.0	RIVER BOTTOM RIVER LEVEL											
963.0	0.6' LOOSE GREY SANDY SILT, SOME ORGANIC MATTER		1	2'	6						MH	
948.5	DENSE TO VERY DENSE GREY SILTY SAND AND GRAVEL.		2	"	77	960					MH	
17.5			3	"	56							
			4	"	>100	950						
	DENSE TO VERY DENSE GREY SILT TRACE OF CLAY AND SAND.		5	"	45						H	
			6	"	46	940						
934.5			7	"	>100						H	
31.5	END OF HOLE					930						
						920						

80 BLOWS FOR LAST 2 INCHES
END OF PEN. TEST AT ELEV. 957.9

BENTONITE SEAL

CAVED MATERIAL

SAND FILL

STANDPIPE

W.L. IN STANDPIPE AT RIVER LEVEL SEPT. 26, 1965

15-10-5 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED *[Signature]*

RECORD OF BOREHOLE 102

LOCATION See Figure 1

BORING DATE SEPT. 6-8 1965

DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE		COEFFICIENT OF PERMEABILITY k , CM./SEC.		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	NUMBER	TYPE		BLOWS/FT.	BLOWS/FT.	BLOWS/FT.	BLOWS/FT.		
965.1	RIVER LEVEL									
0.0										
	WATER									
951.2	RIVER BOTTOM									
13.9		1	D.O.	7						
	LOOSE TO COMPACT GREY FINE SAND, SOME SILT.	2	"	7						
		3	"	14						
		4	WS	-						
936.1		5	D.O.	37						
29.0		6	WS	-						
	DENSE GREY FINE TO COARSE SAND, SOME GRAVEL, TRACE SILT.	7	D.O.	53						
		8	WS	-						
		9	D.O.	35						
919.1		10	"	43						
46.0		11	"	54						
912.1	DENSE GREY SILT, TRACE SAND.									
53.0		12	AXT	-						
908.9	BOULDERS, GRAVEL AND SAND.									
56.2	END OF HOLE									

END OF PEN. TEST AT ELEV. 914.1

15-10-5 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *mg*
CHECKED *JS*

RECORD OF BOREHOLE 103

LOCATION

See Figure

1

BORING DATE

SEPT. 15-21, 1965

DATUM

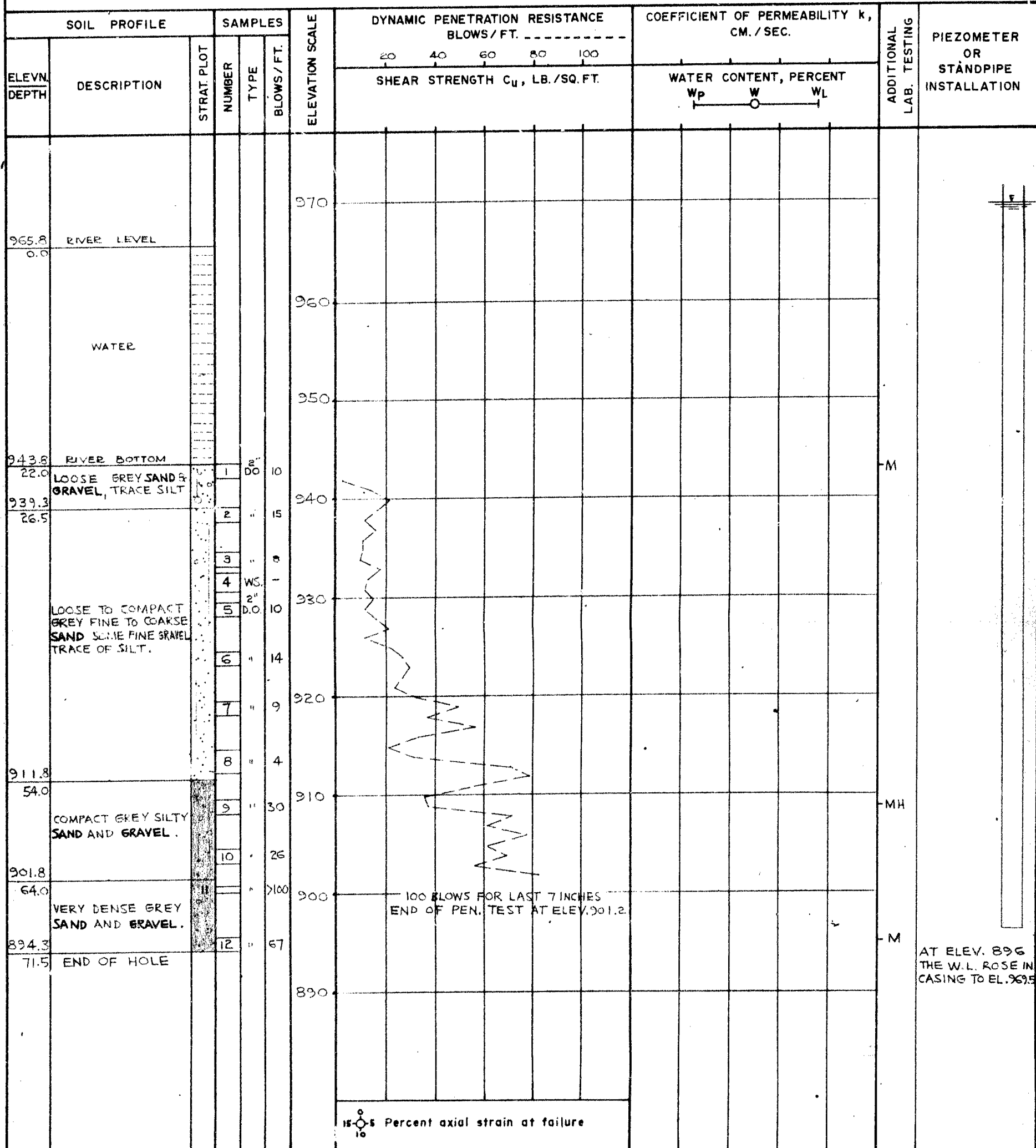
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN BY: [Signature]
CHECKED BY: [Signature]

RECORD OF BOREHOLE 104

LOCATION

See Figure

BORING DATE

SEPT. 16 & 17, 1965

DATUM

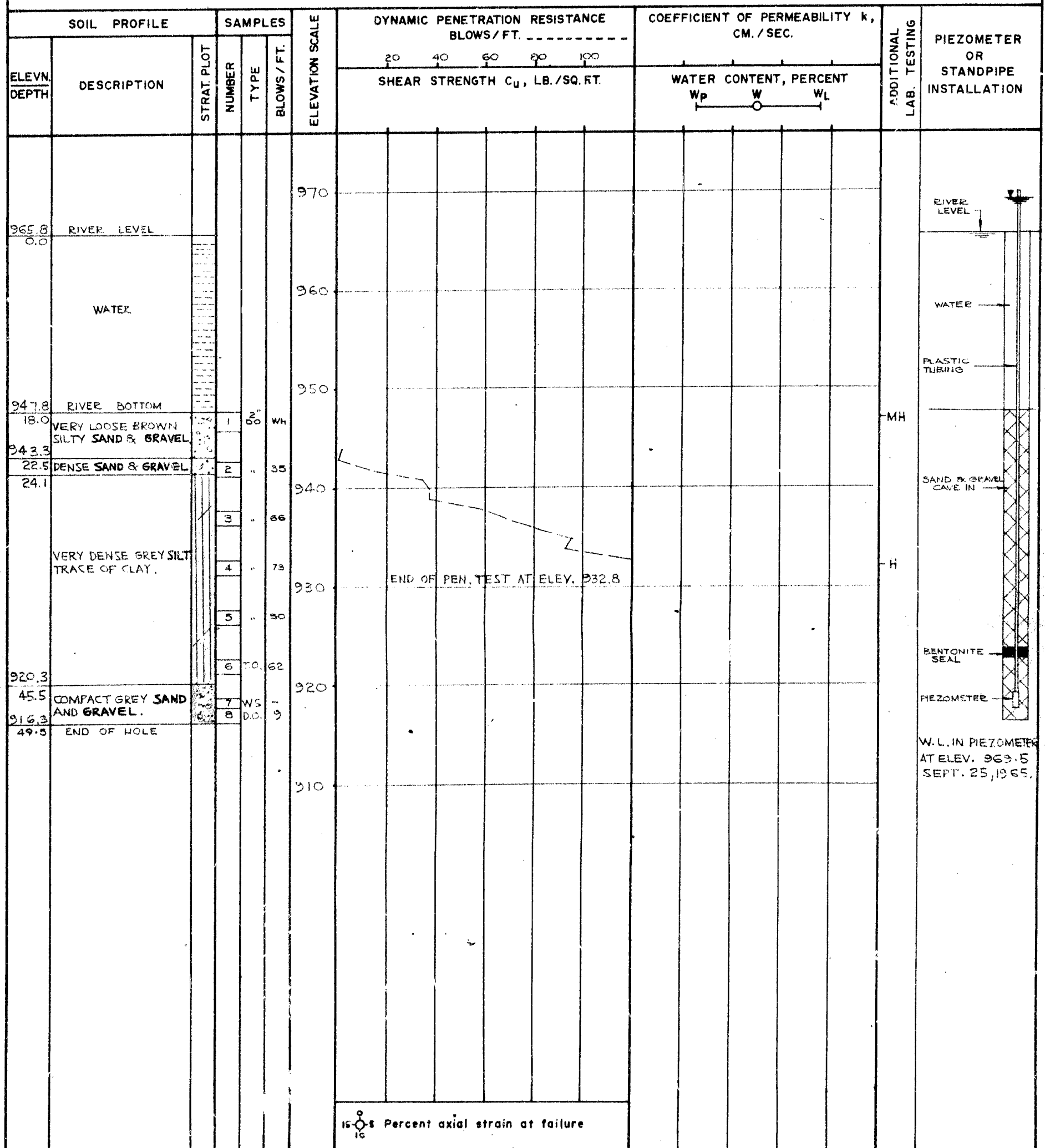
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN W. J. M.
CHECKED W. J. M.

RECORD OF BOREHOLE 105

LOCATION See Figure 1

BORING DATE SEPT. 12-14, 1965

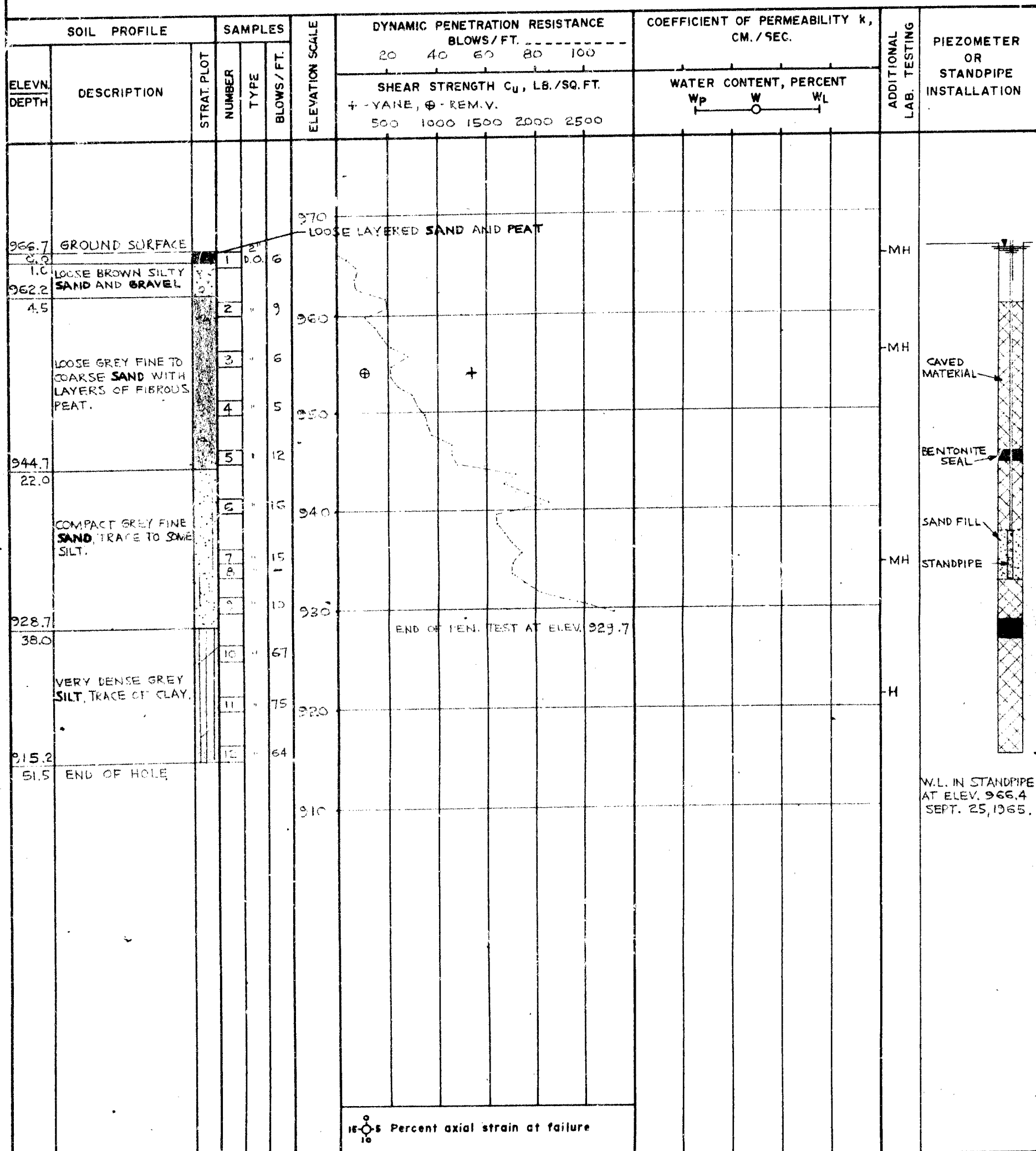
DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *ms*
CHECKED *3/3*

RECORD OF BOREHOLE 106

LOCATION See Figure 1

BORING DATE SEPT. 14-15, 1965

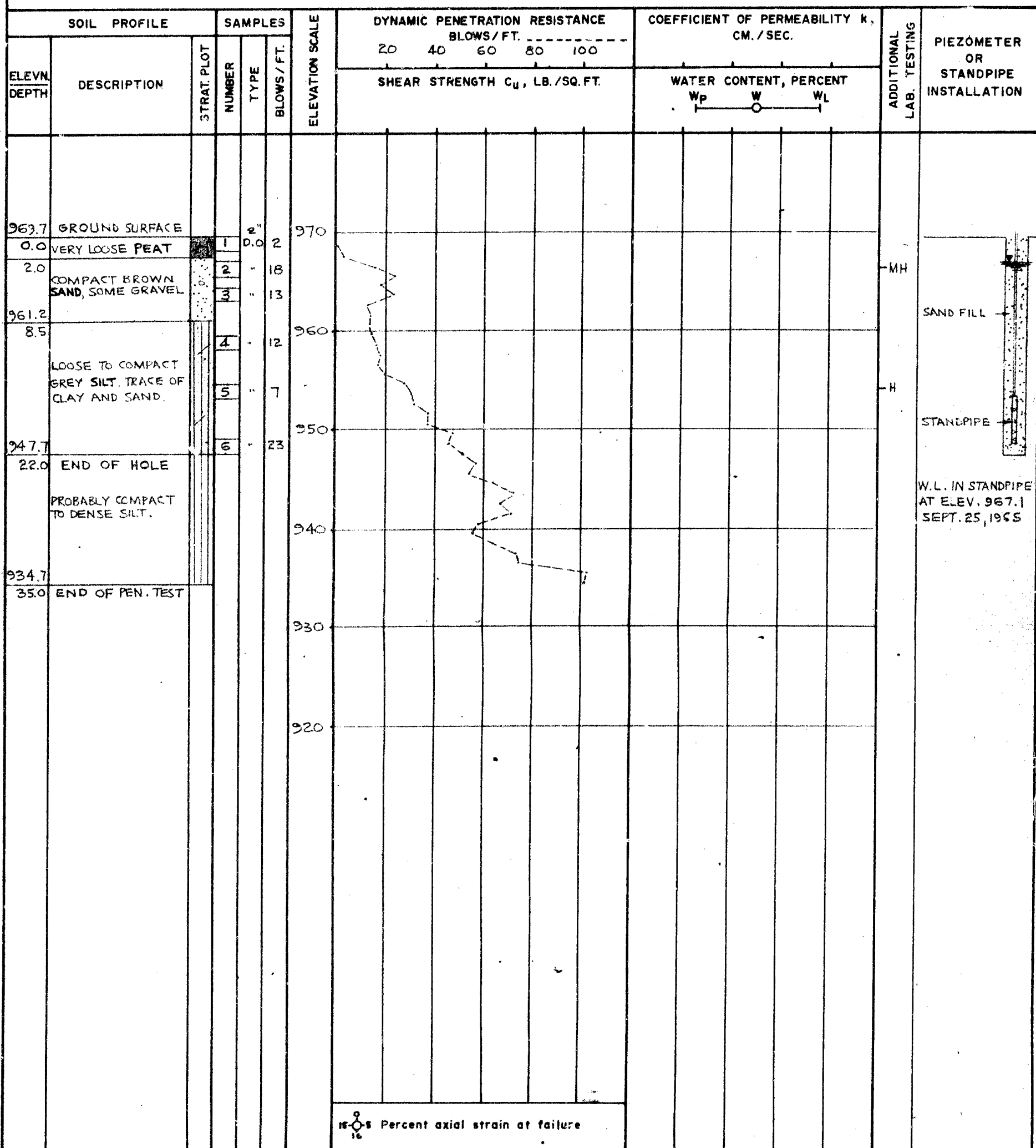
DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *ML*CHECKED *J. J.*

RECORD OF PEN. TEST 108

See Figure 1

SEPT. 26, 1965

DATUM

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT LB. DROP INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FT.	COEFFICIENT OF PERMEABILITY k, CM. / SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.	SHEAR STRENGTH C _u , LB./SQ. FT.	WATER CONTENT, PERCENT w _p w w _L					
	RIVER BOTTOM 966.0 RIVER LEVEL											
0.7 962.0 4.0	PROBABLY LOOSE SANDY SILT.											
	PROBABLY COMPACT TO DENSE SAND & GRAVEL											
946.0 20.0												
	PROBABLY DENSE SILT.											
938.0 28.0	END OF PEN. TEST											

15--5 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN _____
CHECKED _____

RECORD OF PEN. TEST 109

LOCATION

See Figure 1

BORING DATE

SEPT. 6, 1965

DATUM

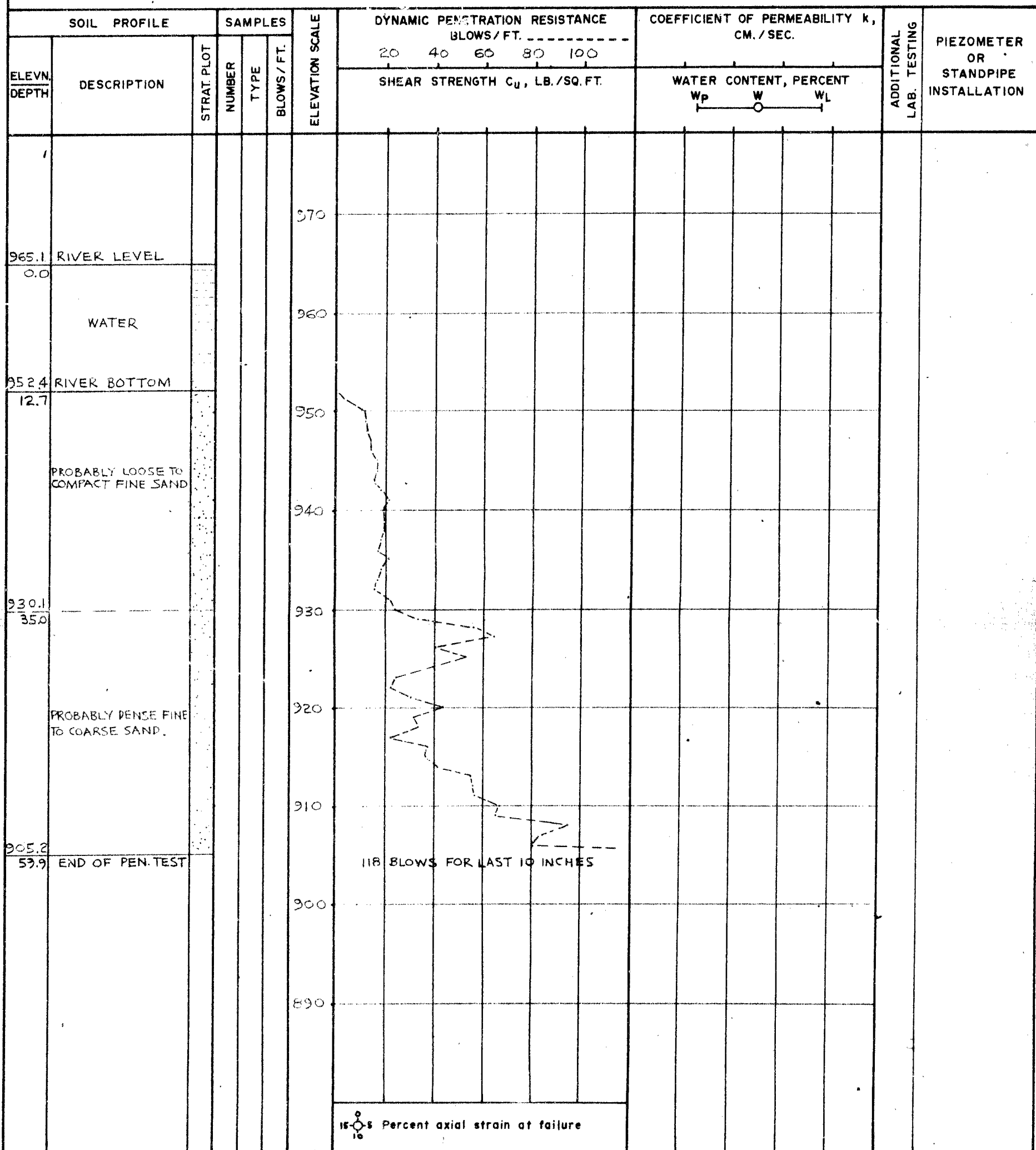
BOREHOLE TYPE

PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT 40 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN MW.
CHECKED F. J. H.

RECORD OF PEN. TEST III

LOCATION

See Figure 1

BORING DATE SEPT. 16, 1965

DATUM

BOREHOLE TYPE

PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. -----					COEFFICIENT OF PERMEABILITY k, CM./SEC.					ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20	40	60	80	100	WATER CONTENT, PERCENT Wp W Wl						
965.8	RIVER LEVEL																	
0.0																		
	WATER																	
946.6	RIVER BOTTOM																	
19.2																		
941.8	PROBABLY VERY LOOSE SILTY SAND & GRAVEL																	
24.0																		
932.8	PROBABLY VERY DENSE SILT.																	
33.0	END OF PEN TEST																	

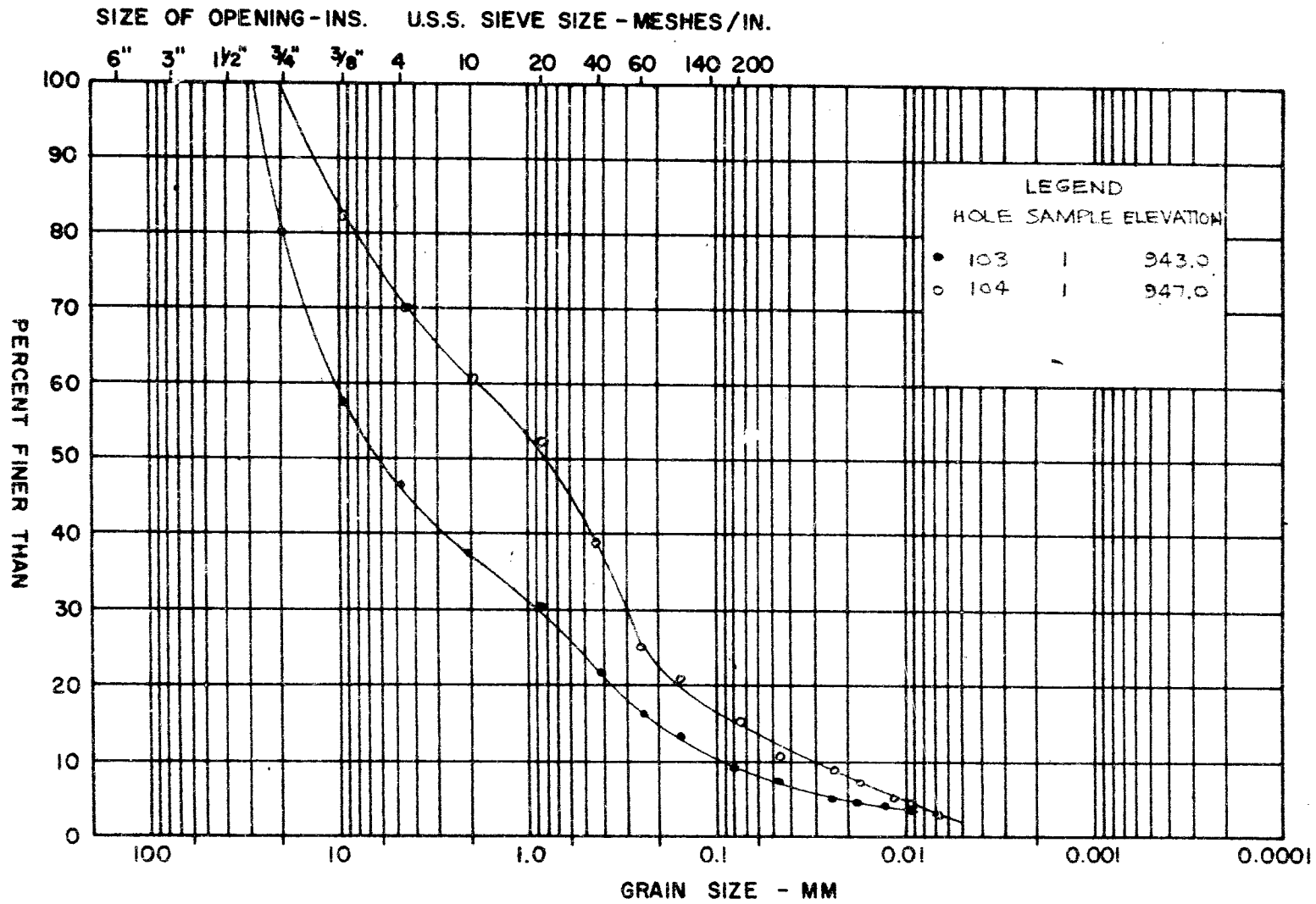
15-10 Percent axial strain at failure

VERTICAL SCALE
1 INCH = 0 10' - 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED *[Signature]*

M.I.T. GRAIN SIZE SCALE



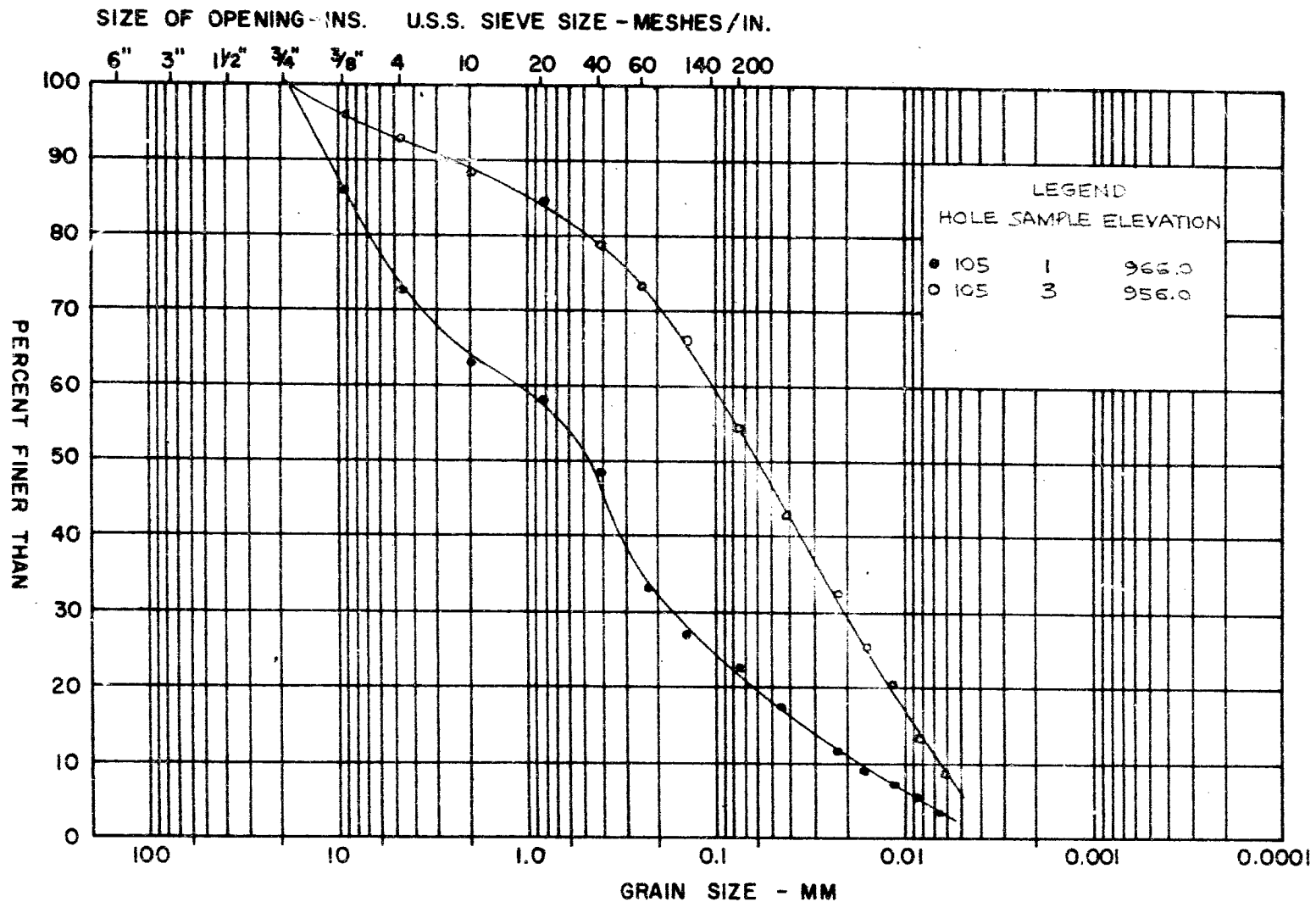
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SILTY SAND AND GRAVEL (RIVER BED DEPOSIT)

FIGURE 2

COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

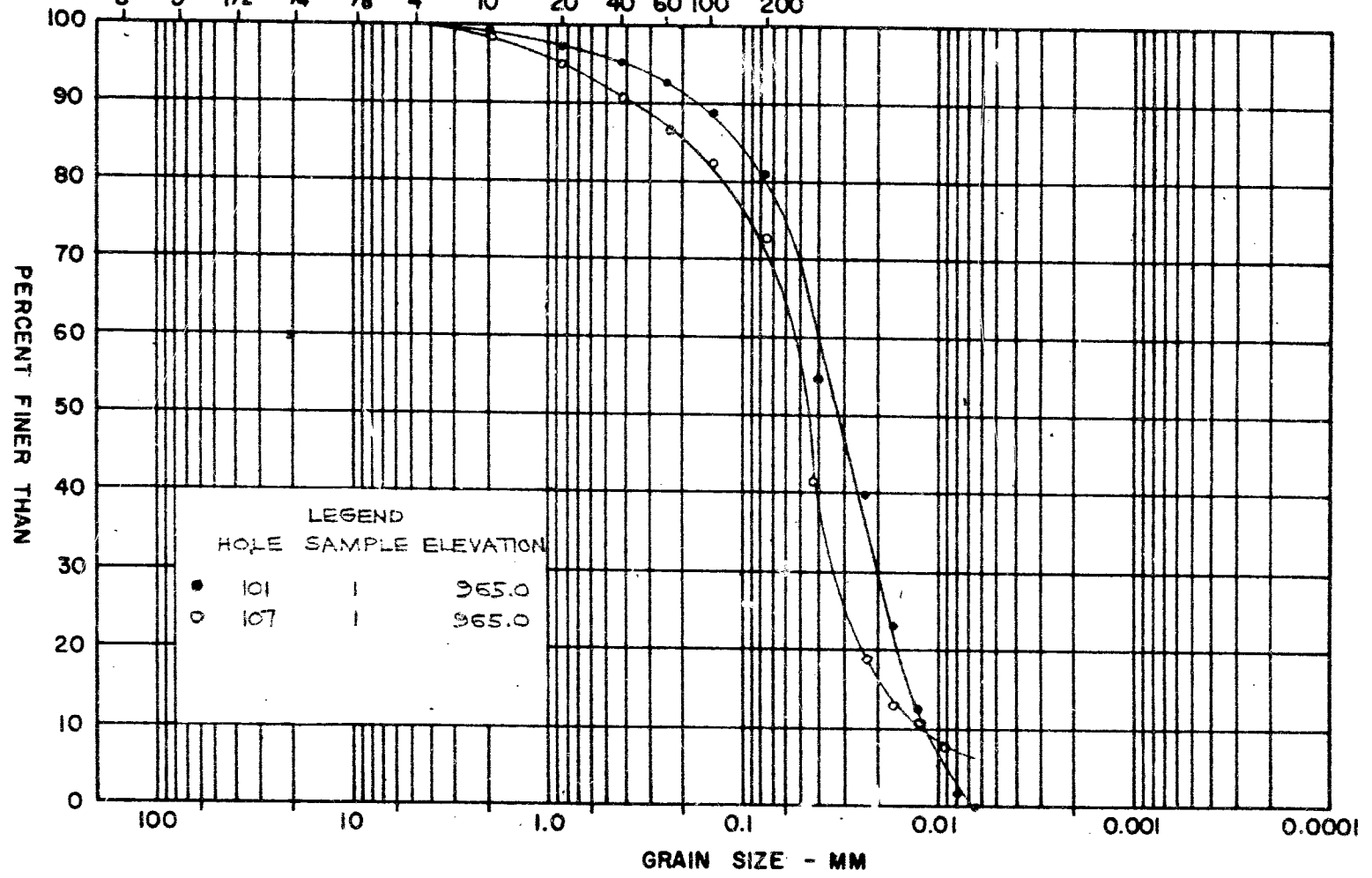
GRAIN SIZE DISTRIBUTION
LAYERED SAND AND PEAT

FIGURE 3

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200



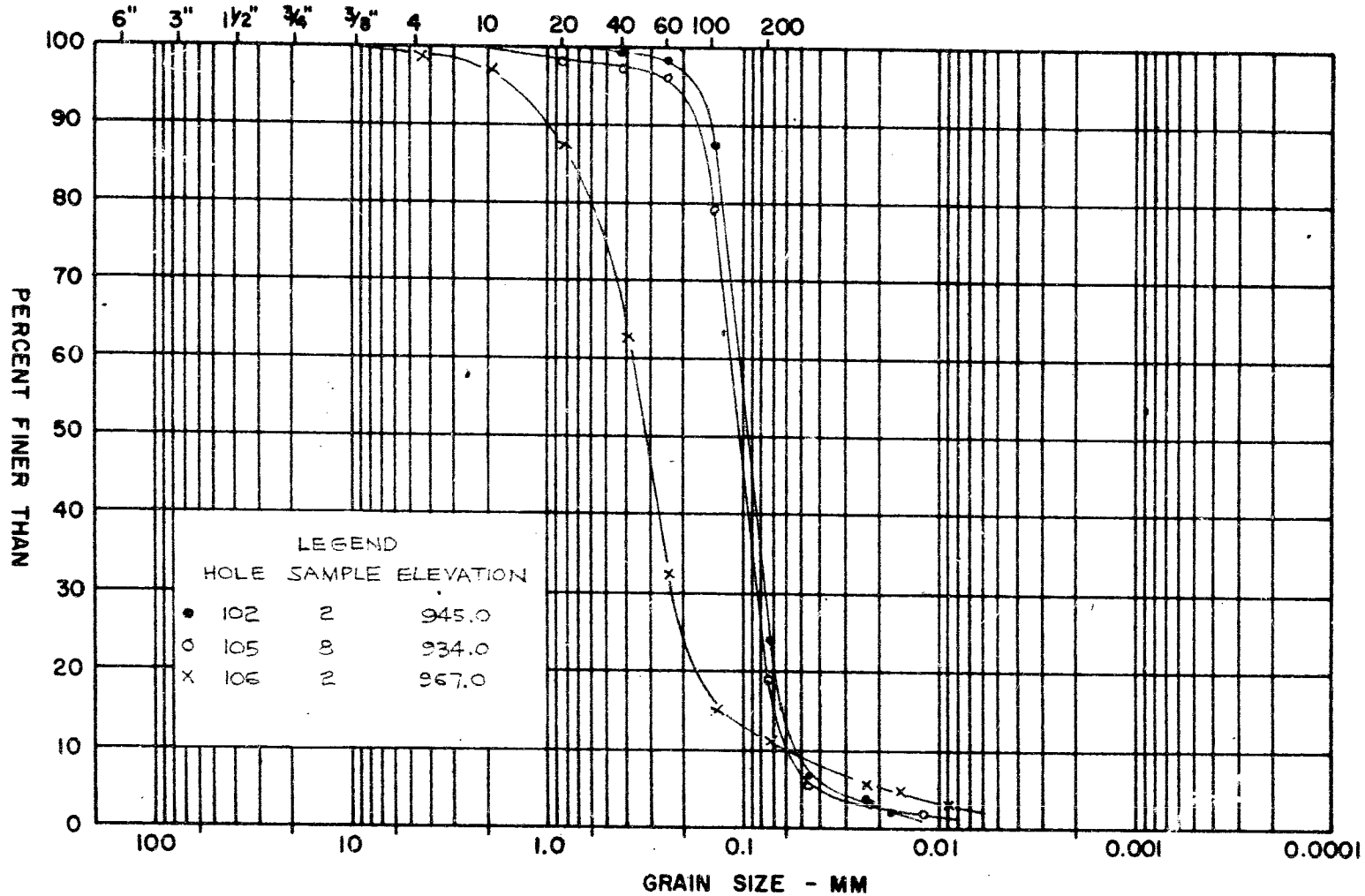
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SANDY SILT

FIGURE 4

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES / IN.



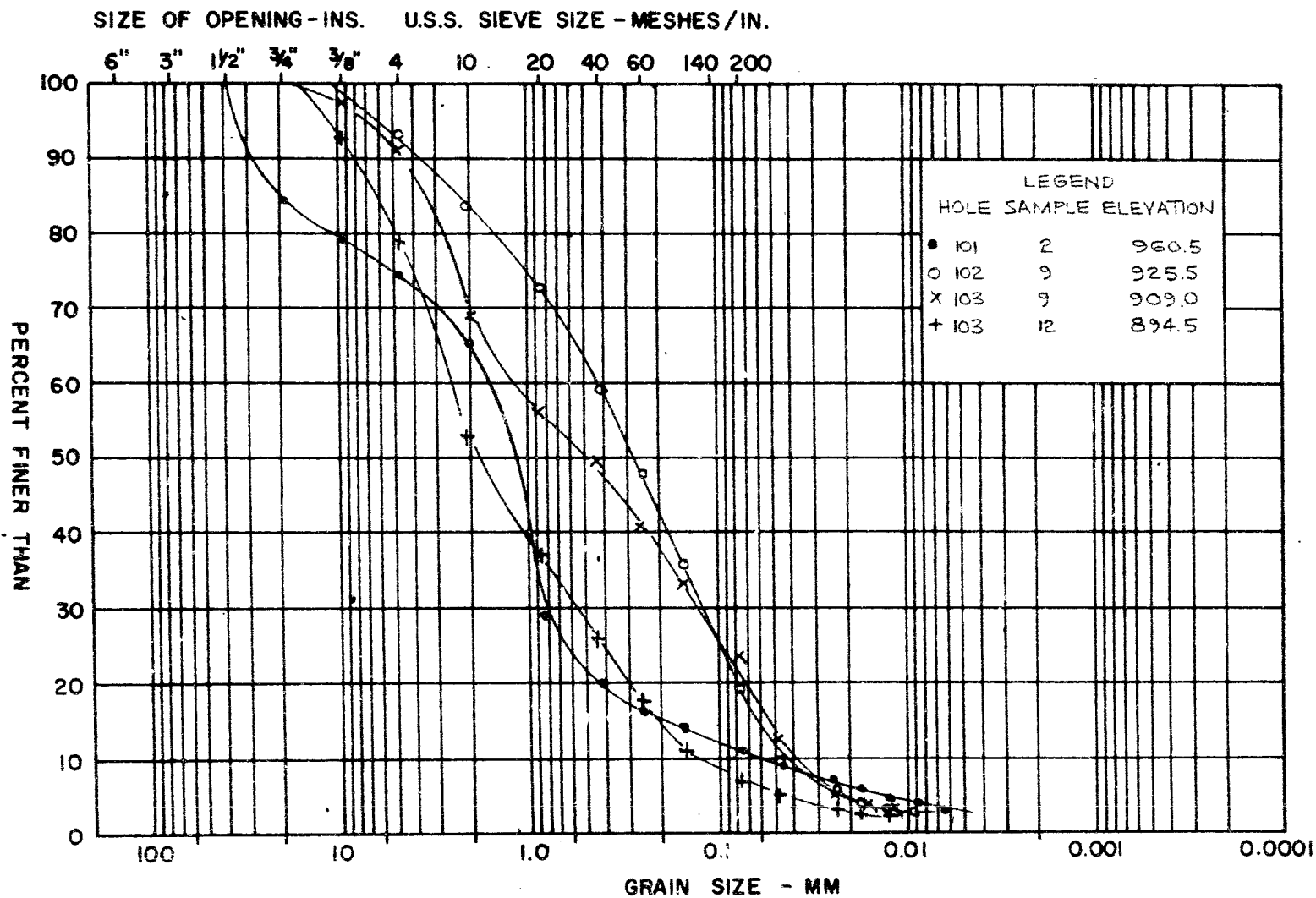
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SAND, SOME SILT AND GRAVEL

FIGURE

5

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SILTY SAND AND GRAVEL

FIGURE

6

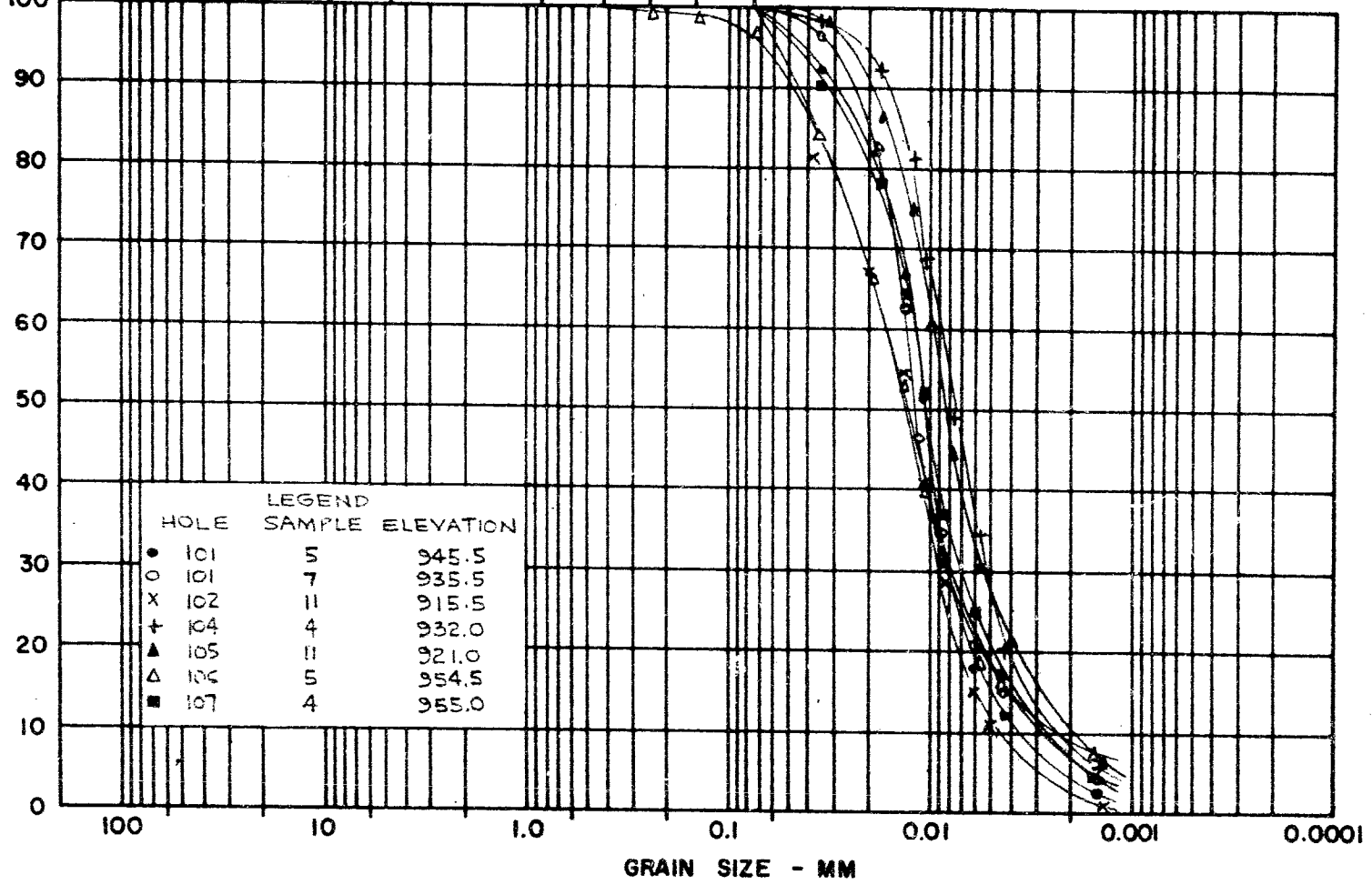
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES/IN.

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200

PERCENT FINER THAN



COBBLE SIZE	COARSE GRAVEL SIZE	MEDIUM GRAVEL SIZE	FINE GRAVEL SIZE	COARSE SAND SIZE	MEDIUM SAND SIZE	FINE SAND SIZE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION
SILT, TRACE CLAY AND SAND

FIGURE 7

GOLDER & ASSOCIATES

Mr. E. H. Davis,
Bridge Engineer,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. S. McCosbie

October 27, 1965

OCT 28 1965

FOUNDATION INVESTIGATION REPORT BY:
H. Q. Gelder & Associates, Limited -
Proposed Osooswin River Crossing,
Line 'H', North of Central Patricia, Ont.
Site 41-103, District 19 (Fort William).

Attached, please find the above-mentioned report submitted by the Consultant, H. Q. Gelder & Associates. We have reviewed the report and found the factual information adequate and well presented.

In view of the seourable nature of the subsoil, we would suggest that the Consultant's alternative recommending foundations on piles, be adopted. Because of the remoteness of the site, consideration should also be given to a trestle-type structure.

We feel that the report contains all the information necessary for your further design work. However, should you require any additional information, please feel free to call on our Office.

cc: H. Q. Gelder
attach.

cc: Messrs. E. H. Davis (2)

H. A. Tregaskes

D. W. Farren

H. W. Hurrell

V. A. Snell

F. Norman

F. DeVissser

A. Watt

Foundations Office

Gen. Files

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

Bay. 401 & Keele St.,
Downsview, Ontario.

Materials and Testing Division

August 6, 1965

**H. C. Calder and Associates Ltd.,
2644 Bloor Street West,
Toronto, Ontario.**

Attention: Mr. J. Szweduk

Utoskwin
**Re: Proposed Utoskwin River Crossings, Site 41-101,
Access Road North of Central Patricia,
District 19 (Fort William).**

Dear Sir:

This is to authorize you to carry out the necessary foundation investigations at the two proposed crossings over the Utoskwin River for the Access Road North of Central Patricia.

The necessary plans for the northerly crossing were given to your representative, Mr. Leo Lanti on August 3, 1965, and this site should be investigated first. Your Engineer will be given the plans for the southerly crossing at the District Office in Fort William.

As mentioned to you on the phone, the above jobs are most urgent and it will therefore, be necessary for your Engineer to phone you the findings at the two sites as soon as they become available. It is thus hoped that a verbal recommendation could be made on the suitability of the two respective crossings. The stability of the approach fills to the two sites, also has to be investigated.

For any additional information or guidance, your Engineer should contact Mr. Frank De Vries, Regional Bridge Location Engineer, Fort William.

It is understood that you will have a fully qualified Engineer supervising the investigation at all times.

Twelve (12) copies of the final report should be submitted to the Foundation Section not later than September 15, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

August 6, 1965

Because the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you requested to prepare them in accordance with the D.E.C. standards. To enable you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide us with Cronaflex copies of the drawings.

Charges for the work performed will be in accordance with your Schedule of Rates, dated September 10, 1962, and invoice to be addressed to the attention of the undersigned.

We are attaching Purchase Order J 34791, covering the purchase of any new material required for this work, in order that you may use this as a basis for exemption from the Federal Tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

al

cc/MSF
Attach.

A. Polka,
MATERIALS & TESTING ENGINEER

cc: Messrs. S. McComb
H. W. Murrell
V. A. Small
P. Brown
P. E. Vignar
S. A. Smith (2)
P. Honings
Foundations Office (2)
Gen. Files (2)

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN
J. L. SEYCHUK

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201

October 27, 1965.

Department of Highways, Ontario,
Materials & Testing Division,
Hwy. 401 and Keele Street,
DOWNSVIEW, Ontario.

Attention: Mr. A. G. Stermac, P.Eng.,
Principal Foundation Engineer.

RE: SITE INVESTIGATION,
PROPOSED OTOSKWIN RIVER CROSSING,
LINE "H",
NORTH OF CENTRAL PATRICIA, ONTARIO.

Dear Sirs:

Twelve copies of our report covering the above investigation were delivered to you yesterday by messenger.

We trust that this report contains sufficient information for your purposes. If you have any questions, please call us.

As discussed on the telephone today, the results of the investigation along the line "A" crossing of Otoskwin river will be presented to you within the next few days. Since it is understood that line "A" has been abandoned in favour of the line "H" crossing, the report for the line "A" crossing will be factual in nature.

Yours truly,
H. Q. GOLDER & ASSOCIATES LTD.,



J. L. Seychuk, P.Eng.

JLS:HDG
65087-1