

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

Attention: Mr. S. McCombie

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

June 30, 1966

JUN 30 1966

FOUNDATION INVESTIGATION REPORT BY:
H. Q. Golder and Associates Limited -
Proposed Papineau Creek Crossing,
Highway No. 127 - Line 'S', Bayneoth, Ont.
N.P. 66-64, District 10 (Banerft).

Attached, please find the foundation investigation report for the above mentioned site, prepared and submitted by the consultant, H. Q. Golder and Associates Ltd.

We have reviewed the report and find the factual information adequate and well presented.

Regarding the recommendations, we would like to make the following comments:

In our opinion, H-piles driven to practical refusal can be used without reservation. Such piles have been used many times under comparable conditions, and as far as we know, their performance has been satisfactory.

Although a conical tip for tube piles would probably enable a deeper penetration of these piles, we feel that this may not be required if adequate bearing can be achieved at a higher elevation. Closed end (with plate at the pile tip) tube piles driven to practical refusal, will provide a satisfactory solution.

We do not recommend the use of timber piles because of the possibility of damage during driving and also because of the relatively small structural strength of timber as compared to steel. It should be borne in mind that all piles driven at this site will act as end-bearing piles and, therefore, the structural strength of the material is of primary importance.

cont'd. /2 ...

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

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

June 30, 1966.

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

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

RE: SOIL CONDITIONS AND FOUNDATIONS,
PROPOSED PAPINEAU CREEK CROSSING,
W.P. 66-64,
MAYNOOTH, ONTARIO.


Dear Sirs:

Eleven copies of our report on the above project, with a Cronaflex copy of Figure 1, were sent to you today, but messenger.

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

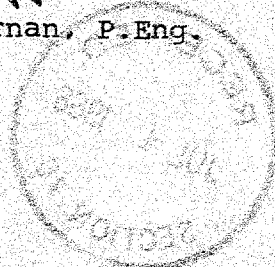
Yours truly,

H. Q. GOLDER & ASSOCIATES LTD.,



F. J. Heffernan, P. Eng.

FJH:hdg
66080



Mr. S. A. Davis,
Bridge Engr.,
Bridge Division.

- 2 -

Attn: Mr. S. McCombie

June 30, 1966

We feel that the information contained in the report and our comments will suffice for your further design work. However, should you feel that you would like to discuss any problem regarding this project, please feel free to contact this Office.

AGS/ndef
Attach.

A. G. Sterness
A. G. Sterness,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. S. A. Davis (2)
R. A. Tregaskes
D. W. Parren
R. S. Pillar
J. E. Callaghan
J. E. Graspier
G. Scott
A. Watt
Foundations Office
Gen. Files

Mr. G. Scott,
Regional Bridge Location Engr.,
Bridge Section,
Kingston, Ontario.

Attn: Mr. J. A. Fisher

Foundation Section,
Materials & Testing Div.,
Rm. 107, Lab. Bldg., Downsview.

November 29, 1966

Your Memo - Nov. 7/66

Proposed Papineau Creek Crossing --
Hwy. #127 - Line 'S', Maynooth, Ont.,
W.P. 66-64 - District #10 (Bancroft).

We have reviewed the Preliminary Plan #D-6021-P1
for the above mentioned structure.

It appears that the designer complied with the
recommendations contained in the foundation report
prepared by soil consultants, H. Q. Golder and Associates.

MD/MieF

cc: Foundations Office ✓
Gen. Files

M. Devata

M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Eng.,
Room 107 Lab. Building,
DOWNSVIEW, Ontario.

From: Mr. G. Scott,
Regional Bridge Location Eng.,
Bridge Section,
KINGSTON, Ontario.

Date: November 7, 1966.

Our File Ref.

In Reply To:

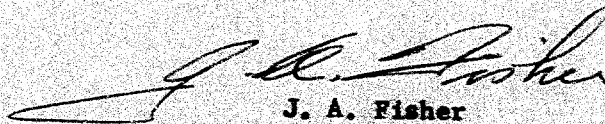
Subject:

RE: W.P. 32-65, Papineau Creek, Site #11-5, Hwy. #127,
District #10, Bancroft.

*← Report and also
their drawings indicate W.P. 66-64 - Why?*

We are sending you herewith one print of Preliminary
plan # D-6021-PI.

Would you kindly let us have your written comments?



J. A. Fisher
For: G. Scott,
REGIONAL BRIDGE LOCATION ENGINEER.

JAF/GS/lm
ATTACH

Report by H. B. Golder & Associates, filed in 64.

MEMORANDUM

To: Mr. J. E. Callaghan,
District Engineer,
BANCROFT, Ontario.

FROM: Bridge Division,
KINGSTON, Ontario.

DATE: May 31, 1967.

OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 66-64, Site 11-5, Papineau Creek South Bridge
Highway 127, District 10, BW-1045

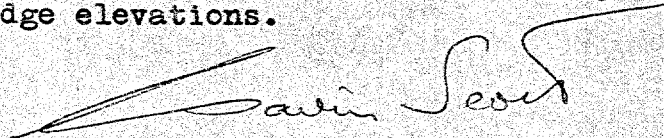
This will record our discussion at the above site concerning the proposed stream diversion for the subject structure, the following persons being present:

J. E. Callaghan	- District Engineer
Ralph Martin	- Construction Supervisor
J. D. Harris	- Bridge Hydrology Engineer
Frank Adams	- Student Engineer
G. Scott	- Bridge Location Engineer

In the interests of economy and to avoid possible difficult work through the swamp area, it has been agreed to move the structure location 100 ft. southward along the proposed highway (to Station 30+50 for CL bridge channel) as shown in red on the attached sketch.

It should be noted that these revised stream diversion channels are minimal and they should be improved if found practicable.

It was also agreed that the profile grade would be adjusted to conform to the bridge elevations.



Gavin Scott
REGIONAL BRIDGE LOCATION ENGINEER

GS/hl
Encls.
c.c.

Mr. S. J. Markiewicz

Mr. C. S. Grebski

→ Mr. A. G. Stermac

Mr. J. E. Gruspier

Mr. J. D. Harris

Bridge Office Files Section: Downsvievw

RY CONSTRUCTION
STREAM DIVERSION

T.S. 32+35.99

PROPOSED STREAM DIVERSION
(MINIMAL)

FOR DE
SEE SH

REMOVE
TOWNSHIP DUM

STA 30+50
& BRIDGE
CHANNEL

EAM DIV.

CL STREAM
DIVERSION

30+00

FLOW

PAPINEAU CREEK

ERECT STRUCTURE
BRIDGE DRAWINGS
D-6021 1 to 10

PROPOSED STREAM DIVERSION
(MINIMAL)

ERECT 75' STEEL BEAM GUIDE
RAIL DD-908 & DD-909 AT EACH
CORNER OF STRUCTURE

NOTE

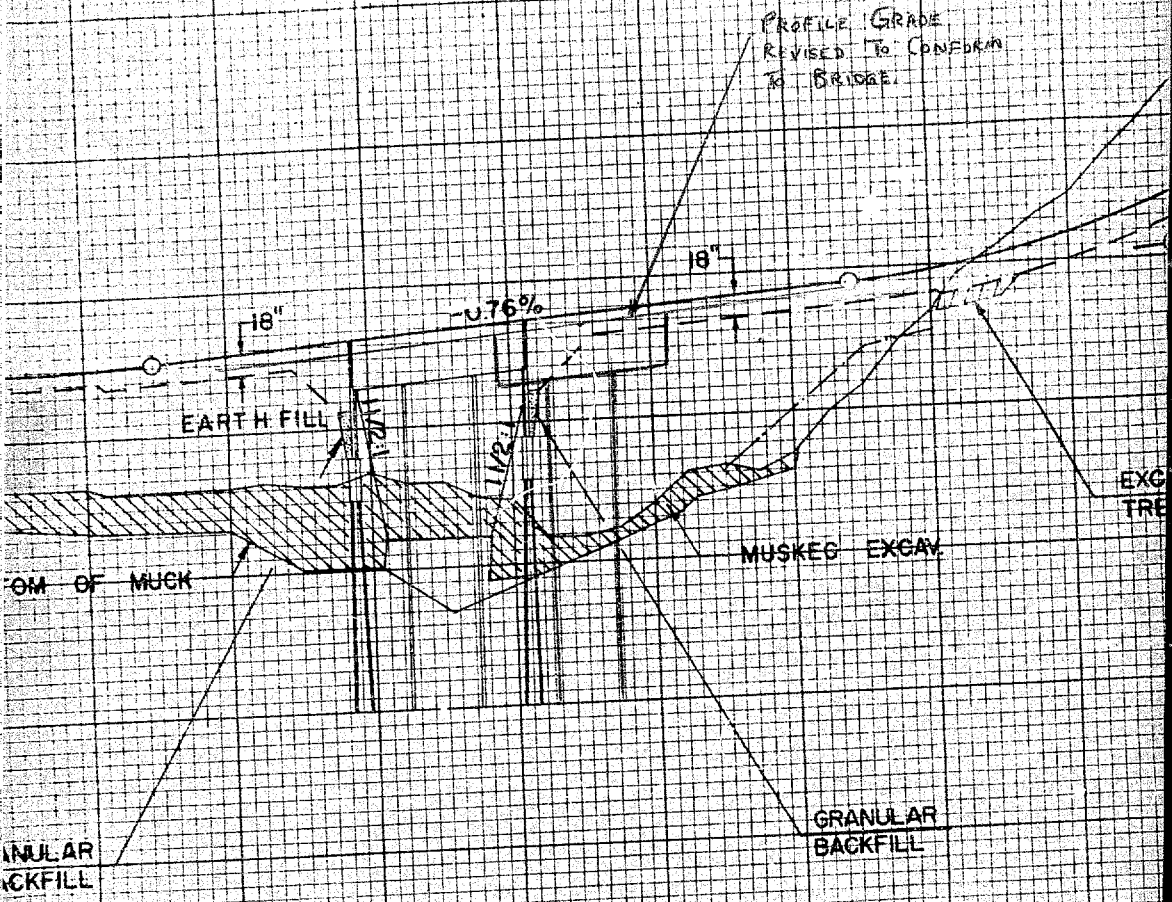
FOR DETAIL ON STREAM
DIVERSION SEE SHEET N^o 10

PLACE RANDOM RIP-RAP
STA. 28+50 TO STA. 30+
STA. 32+40 TO STA. 33+
LT. OF Q TO ELEV. 1293.
MINIMUM DEPTH = 2'

NOT 32+03.64

3+55.10 = STA. 0+00
SIDEROAD RT.

500' V.C.
L.V.C. 470



ERECT STRUCTURE
BRIDGE DRAWINGS
D-6021 1 to 10

4' 00' LT.
-230.00'
-265.68'

SC 33+85.99

TS 32+35.99

Es-2°15' Ls-150.00'

30+00

3 2 1

9 8 7

66-F-2212

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
763-4203
767-9201

W.P. 66-64

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED PAPINEAU CREEK CROSSING

HIGHWAY NO. 127 - LINE "S"

MAYNOOTH

ONTARIO

Distribution:

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

June, 1966

66080

TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	1
INTRODUCTION	2
PROCEDURE	2
SITE AND GEOLOGY	3
SOIL CONDITIONS	5
GROUNDWATER CONDITIONS	7
DISCUSSION	8
General	8
Foundations	8
Approach Embankments	11
ABBREVIATIONS	In Order
RECORDS OF BOREHOLES	Following
	Page 12.
FIGURES 1	- Boring Plan and Soil Stratigraphy Section
2 & 3	- Grain Size Distribution Curves

ABSTRACT

The results of an investigation to determine the sub-surface conditions at the site of a realigned crossing of Highway 127 over Papineau Creek, north of Maynooth, 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 covered by a soft fibrous peat ranging in depth from 2 feet to about 12 feet. The peat is underlain by loose to compact sand. The sand deposit ranges from about 10 feet in thickness at the southern extremity of the site to about 30 feet at the northern extremity. A very dense sandy glacial till containing numerous cobbles and boulders underlies the sand deposit directly above sound granite gneiss bedrock. The till mantle is of variable thickness, being about 25 feet thick at the proposed south abutment location and about 5 feet thick at the north abutment.

It is recommended that the piers and abutments of the proposed bridge structure be founded on end bearing piles, either steel pipe or H section. Low capacity timber piles may also be used. The piles should be driven to practical refusal in the glacial till or on the bedrock surface.

There should be no overall stability problem with approach embankments, having 2 horizontal to 1 vertical side slopes, provided the peat is completely removed down to the surface of the sand deposit.

INTRODUCTION

H. Q. Golder and Associates Ltd. have been retained by the Department of Highways, Ontario. to carry out a soil investigation at the site of a proposed bridge crossing along the re-alignment of Highway 127 over Papineau Creek, north of Maynooth, Ontario. The purpose of the investigation was to determine the soil and groundwater conditions across the site and to make recommendations for the foundation design of the proposed structure and the roadway approach embankments.

PROCEDURE

The field work for the investigation was carried out between May 31 and June 18, 1966. During this period 8 boreholes (numbered 1, 3, 5, 7, 8, 9, 10 and 11), four of which had adjacent dynamic penetration tests, and 3 additional dynamic penetration tests (numbered 2, 4 and 6) were put down to a maximum depth of about 50 feet. In addition in-situ falling head permeability tests were carried out in boreholes 3 and 5. The borings were carried out using a skid-mounted machine drillrig, supplied and operated by F.E. Johnston Drilling Co. Ltd. All borings were carried out from a raft supplied by the drilling contractor. The field work was supervised throughout by a member of our engineering staff.

The locations of the borings put down during this investigation are shown on Figure 1 located in a pocket near the rear of this report. A detailed record of each borehole and dynamic penetration test is presented on the Record of Borehole sheets following the text of this report. A section along the centreline of the proposed alignment is also given on Figure 1.

The soil and rock samples obtained during the investigation were shipped to our Toronto laboratory for detailed examination and selective testing. The results of the laboratory testing are presented on the Records of Boreholes and on Figures 2 and 3.

The elevations of the boreholes given in this report are referred to Geodetic datum. As all borehole locations were at or below the creek water level, all depths in the boreholes were referenced to this level. A detailed record of the variations of the creek water level was kept throughout the boring programme by reading on a depth board. The elevation of the depth board was determined by Department of Highways, Ontario personnel.

SITE AND GEOLOGY

The site of the proposed bridge for the line "S"

alignment of Highway 127 over Papineau Creek is located some 0.5 miles north of Maynooth and 15 miles north of Bancroft, Ontario. Papineau Creek flows from west to east at the proposed crossing and is some 200 feet wide and from 2 to 6 feet deep.

The proposed crossing is some 400 feet west of the present single lane Highway 127 bridge over Papineau Creek. To the north of the creek at the proposed site for some 500 feet there is a flat, low lying peat deposit covered with small bushes and grass. The south bank is covered by peat for a distance of only some 60 feet, from the waters edge to where the ground rises steeply to a well wooded glacial deposit. The surface of the peat is fairly level at an elevation some 2 to 3 inches above the present creek water level. Granitic bedrock outcrops at the existing bridge site where the creek flows through a relatively narrow channel.

From available geological information, it is known that the region is generally covered by a thin mantle of glacial till, overlain in some areas by sands and gravels. Because of this thin overburden covering, the surface topography generally reflects bedrock relief. The bedrock is metamorphic, principally granite gneiss of Pre-Cambrian age.

SOIL CONDITIONS

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

The route is covered by a deposit of soft fibrous peat ranging in depth from about 2 feet at the northern and southern extremities of the site to about 12 feet in parts of the creek bed. The deposit of peat on the north bank is fairly extensive in area, stretching from the bank of the creek to some 500 feet north of the creek and some 200 feet on either side of the proposed centreline. On the south bank the peat deposit is not present beyond a distance of about 60 feet of the creek edge where the ground surface rises steeply. The peat is dark brown, fibrous, very soft and has a high water content. Laboratory water content values for samples of the peat range from 150 to 600 percent by dry weight.

The peat along the route investigated is underlain by a deposit of sand some 10 to 30 feet thick. The sand deposit is generally well graded and has the appearance of alluvium. Grading curves for samples of the sand are shown on Figure 2. The sand deposit appears to contain more gravel sizes on the south side of the site than on the north side where grain size curves show the

material in boreholes 7 and 8 to be essentially a medium to coarse sand.

Standard and dynamic penetration tests indicate that the density of the sand varies considerably from boring to boring. In boreholes 1 and 3, "N" values of between about 20 and 40 blows/ft. were obtained indicating a compact to dense condition, while in boreholes 5 and 7, "N" values as low as 3 blows/ft. were obtained indicating a very loose to loose condition.

In-situ falling head permeability tests were carried out in boreholes 3 and 5. The results of these tests indicate the average coefficient of permeability of the sand to be in the order of 1×10^{-3} cm/sec.

A dense to very dense sandy glacial till directly overlies the bedrock beneath the sand deposit in most of the boreholes. In boreholes 1, 8 and 11 numerous cobbles and boulders were encountered within the till. Generally the cobbles and boulders are more predominant in the lower part of the stratum, close to or at the till bedrock contact. The thickness of the till stratum varies from about 30 feet at the southern extremity of the site (borehole 11) to about 5 feet in borehole 8 at the northern extremity. In borehole 7 the till was not encountered. The results of grading tests on samples of the till obtained using $1\frac{1}{2}$ inch I.D. sampling

equipment are shown on Figure 3.

Standard penetration test values were generally found to be in excess of 50 blows/ft. indicating the very dense nature of the till deposit.

Bedrock was encountered and cored in AXT size in all boreholes. From visual examination of the rock core and from the high core recovery obtained, the bedrock is considered to be sound. Bedrock is a hard, sound, dark grey to pink granite gneiss. The surface elevation of the bedrock appears to dip gently to the south-west. The bedrock surface in the north-east corner of the site is at about elevation 1,250 while at the south-western extremity of the site bedrock was encountered slightly above elevation 1,240.

GROUNDWATER CONDITIONS

Piezometers were installed within the till deposit in boreholes 1 and 5 following their completion. During two weeks of daily reading of the piezometer in borehole 1 and one week of daily reading in borehole 5, it was found that the piezometric groundwater level closely followed that of the creek. The creek water level at the site varied between elevation 1,286.3 and 1,287.2 during the period of the investigation. No artesian pressure was encountered at the site.

DISCUSSION

General

It is understood that the Papineau Creek bridge is to be a five span structure, with the central and end spans being about 40 feet long and the remaining two spans each about 60 feet in length. It is not known at this time whether the abutments, which will be located slightly back from the creek edge, will be spill-through or of the retaining type. The proposed highway grade is to be at an elevation of about 1,297, that is some 11 feet above present creek level, necessitating roadway approach embankments about 10 feet high above the peat at existing ground surface or as much as 20 feet above the underlying sand stratum.

Foundations

Due to the variable and generally loose density of the sand deposit, which extends from within a few feet of the creek bed to as much as a 30 foot depth, the sand is not considered a suitable bearing stratum for the support of the bridge abutments and piers on spread footings. It is therefore recommended that a piled foundation be employed at this site due to the more favourable subsurface conditions at a relatively shallow depth. It is considered that a driven end bearing pile would be the most suitable pile type at this site. The pile type and length required

is largely governed by the glacial till stratum encountered immediately below the granular deposit and directly overlying the bedrock. At the north abutment and north piers, where the till stratum is thin or non-existent, pile lengths below existing ground surface would be as much as 40 feet. At the south abutment where the till is closer to ground surface it is estimated that piles in the order of 20 feet would be sufficient.

Steel piles, either pipe or H section, would be suitable at this site. For an "H" steel pile driven to a final set of about 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 piles should, however, be provided with a specially reinforced drive shoe tip to prevent damage during driving to practical refusal through the boulders and cobbles in the till stratum. It is estimated that "H" piles would encounter practical refusal within the lower portion of the till stratum in the southern end of the site and on bedrock in the northern portion of the site.

The disadvantage of steel "H" piles is that they may experience considerable deflection and be subjected to large bending stresses during driving due to the presence of cobbles and boulders within the till. An alternative to an "H" pile section is a pipe pile which allows inspection of the shaft after driving.

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 also taken as 70 tons/pile. It is estimated that pipe piles would meet practical refusal within some 10 feet below the surface of the glacial till deposit. To facilitate driving, the base of pipe piles should be provided with a conical shaped tip.

The above piles will provide a relatively high capacity per pile. If lower capacity piles have an economic advantage for the proposed bridge structure, timber piles driven to practical refusal may also be employed at this site. Considering a timber pile, with a 12 inch butt and 8 inch tip diameter, driven to a final set of about 5 blows to the inch with a hammer having a rated energy of about 12,000 ft.lb/blow, the design load may be taken as 20 tons per pile. It is estimated that the timber piles would meet the above penetration resistance at about the surface of the dense till stratum. The length of timber piles required below existing ground surface would vary between about 15 feet at borehole 11 to about 40 feet at penetration test 6 location. As the water level upstream from the existing bridge is relatively constant throughout the year at about elevation 1,286, due to the spillway affect of the rock outcrop at the existing bridge site, timber piles could be cut-off below creek low water level and

therefore should not be subject to rotting.

With provision of either a steel or timber piled foundation, as discussed above, the settlement of the bridge structure should be negligible.

If retaining type abutments are 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 6 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, $K_0 = 0.4$, 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 north approach it is known that the peat deposit is some 3 to 5 feet thick at the proposed abutment location. The thickness of the peat deposit was investigated for some 60 feet

beyond this abutment location, as shown on Figure 1. It is understood that the District Soils Section of the Department of Highways, Ontario will investigate the extent of the peat to determine the treatment required to the north of the north abutment location for the proposed roadway. Within the limits of the proposed bridge abutments (i.e. at least 60 feet behind the abutments, the peat should be completely removed down to the underlying sand. The approach embankments directly behind the abutments should then be constructed on the sand deposit underlying the peat using 2 horizontal to 1 vertical side slopes. Provided that suitable fill properly compacted in place is used in the embankment construction, there should be no overall stability problem. As no organic or peat layers were encountered in the sand deposit any settlement of the approach embankments resting on the sand would be small and would 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 gullyng.

FJH:hdg
66080
June 28, 1966.



F. J. Heffernan
F. J. Heffernan, P.Eng.

F. J. H.
for J. L. Seychuk, P.Eng.

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

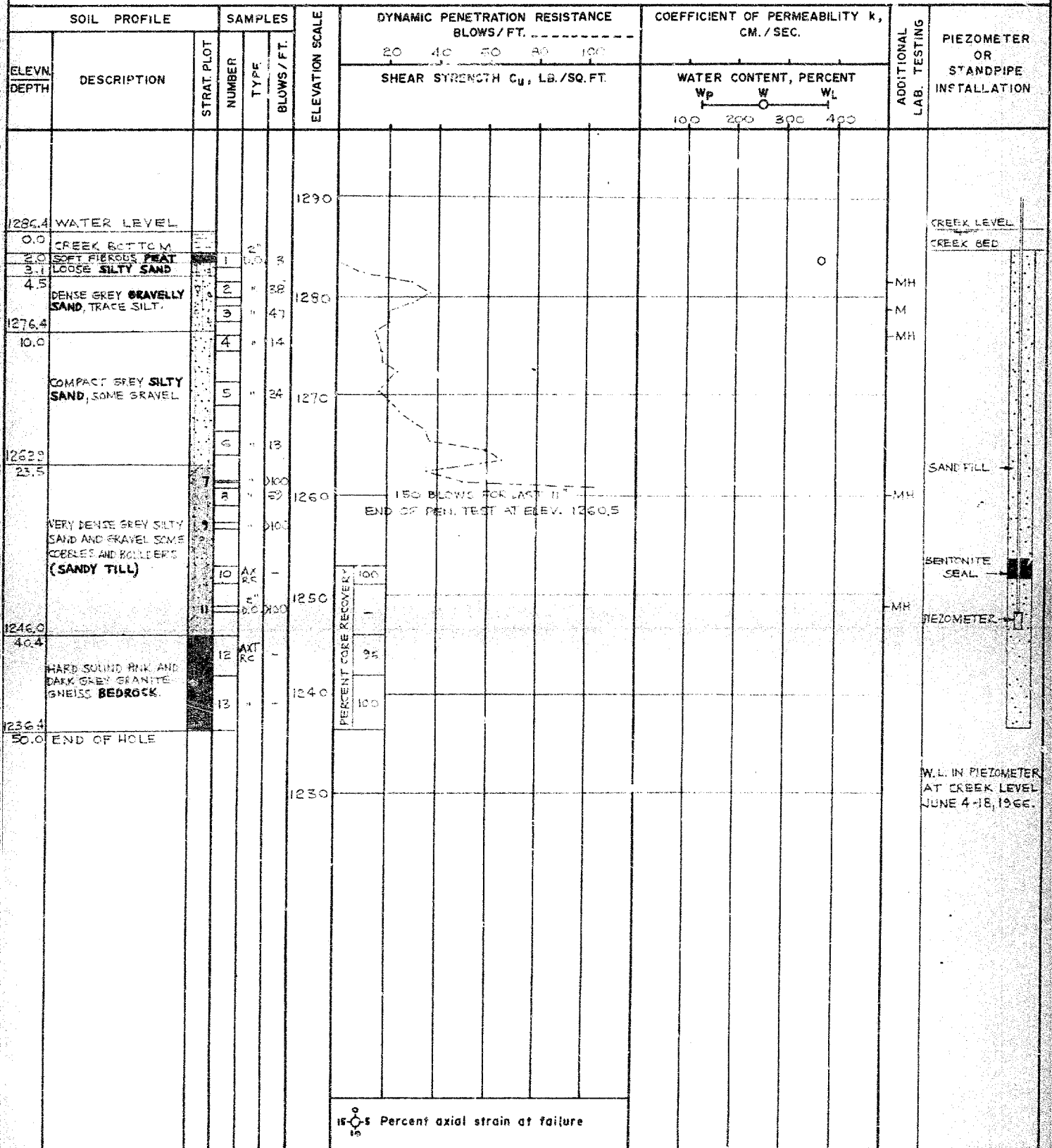
(e) Shear strength

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_f	sensitivity

*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 1

LOCATION See Figure 1 BORING DATE JUNE 1-3, 1966 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 8X, 1X, 1X 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 *[Signature]*
 CHECKED *[Signature]*

RECORD OF BOREHOLE 3

LOCATION See Figure 1 BORING DATE JUNE 3-8, 1955 DATUM GEOLETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER
 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
ELEVATION DEPTH	DESCRIPTION	STRAT. PLOT NUMBER	TYPE		BLOWS/FT.	BLOWS/FT.	WATER CONTENT, PERCENT	WATER CONTENT, PERCENT		
1286.4	WATER LEVEL									
0.0	CREEK BOTTOM									
2.0	SOFT DARK BROWN FIBROUS PEAT	1	D.C. WH							
1278.5		2	" WH							
7.8	LOOSE SILTY SAND	3	" 9							
9.5		4	" 6							
		5	" 9							
		6	WS -							
	LOOSE TO COMPACT GREY SAND SOME GRAVEL AND SILT	7	D.C. 13							
		8	WS -							
		9	D.C. 21							
1255.4		10	" 40							
31.0		11	AX RC -							
	DENSE TO VERY DENSE GREY SILTY SAND, SOME GRAVEL (SANDY TILL)	12	D.C. 100							
1241.9		13	AX RC -							
44.5		14	" -							
	HARD SOUND PINK AND DARK GREY GRANITE GNEISS BEDROCK	15	" -							
1233.1		16	" -							
53.3	END OF HOLE									

1290

1280

1270

1260

1250

1240

1230

WH

WH

9

6

9

-

13

-

21

40

-

100

-

-

-

END OF PEN. TEST AT EL. 1244.4

153

% CORE RECOVERY

25

100

100

100

15-5 Percent axial strain at failure

ORGANIC CONTENT 24.2

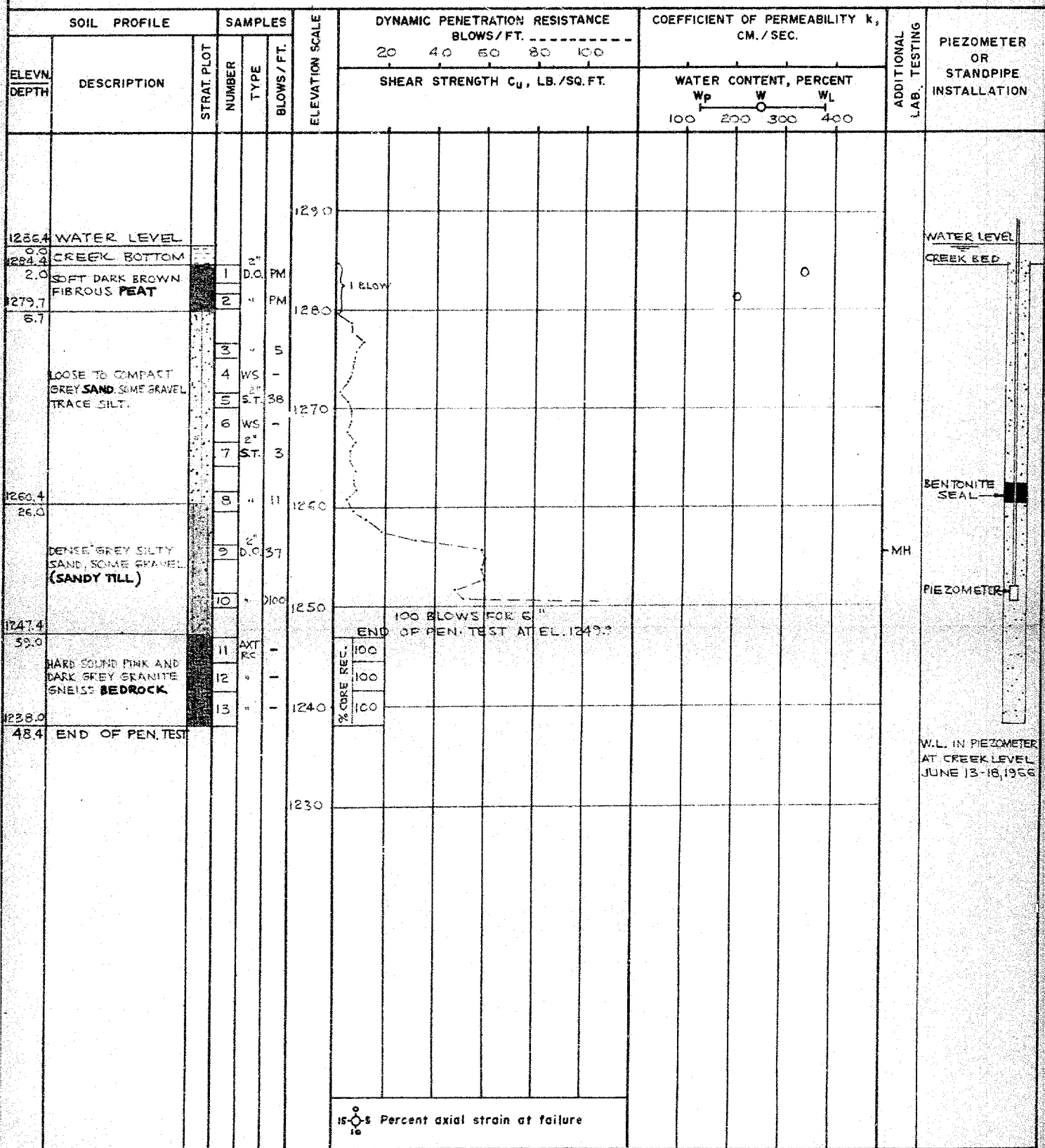
VERTICAL SCALE
1 INCH TO 10' 0"

GOLDER & ASSOCIATES

DRAWN *m.w.*
CHECKED *3/2*

RECORD OF BOREHOLE 5

LOCATION See Figure 1 BORING DATE JUNE 8-10, 1966 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX, AX 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 *mw*
CHECKED *gja*

RECORD OF PENETRATION TEST 6

LOCATION See Figure 1

BORING DATE JUNE 13, 1966

DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

[illegible]

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN _____
CHECKED F. J.

RECORD OF BOREHOLE 7

LOCATION See Figure 1

BORING DATE JUNE 13-14, 1966

DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER 8x CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. -----	COEFFICIENT OF PERMEABILITY K, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
ELEV'N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER		TYPE	SHEAR STRENGTH C _u , LB./SQ.FT.	WATER CONTENT, PERCENT W _p W W _L 100 200 300 400					
	CREEK BED				1290							
1286.4	WATER LEVEL											
0.6			1	2"	D.C. PM							
	DARK BROWN FIBROUS PEAT		2	"	PM							
1274.5			3	"	PM							
11.9			4	"	20							
	LOOSE TO COMPACT GREY MEDIUM AND COARSE SAND, SOME GRAVEL.		5	"	0							
			6	"	0							
			7	"	0							
			8	"	23							
1247.9					1250							
38.5			9	AST RC	1							
	HARD SOUND PINK AND DARK GREY GRANITE GNEISS BEDROCK		10	"	1							
			11	"	1							
1236.7			12	"	1							
49.7	END OF HOLE											
					1230							

15-10-5 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *m.w.*
CHECKED *3/2*

RECORD OF BOREHOLE S 9 & 10

LOCATION

See Figure 1

BORING DATE

JUNE 15 1966

DATUM

GEORGETIC

BOREHOLE TYPE

WASH BORING:

BOREHOLE DIAMETER

255

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT --- LB. DROP --- INCHES

SOIL PROFILE			SAMPLES		ELEVATION SCALE	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 <div>W_p W W_L 100 200 300 400</div>				
1286.4	GROUND LEVEL					10						
0.0	SOFT DARK BROWN FIBROUS PEAT		1	D.O.	PM							
1282.4	COARSE SAND		2	N	PM							
4.5	END OF HOLE											
1286.4	GROUND LEVEL					10						
0.0	SOFT DARK BROWN FIBROUS PEAT		1	D.O.	PM							
1282.4	COARSE SAND		2	N	PM							
3.5	END OF HOLE											

<

VERTICAL SCALE

1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN me

CHECKED 4 12

RECORD OF BOREHOLE II

LOCATION See Figure 1

BORING DATE JUNE 17-18, 1965

DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX BX AX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT -- LB. DROP -- INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. -----	COEFFICIENT OF PERMEABILITY k_v , CM./SEC.		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVATION DEPTH	DESCRIPTION	STRAT PLOT NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH C_u , LB./SQ. FT.	WATER CONTENT, PERCENT <div><div><div>W_p</div><div>W</div><div>W_L</div></div><div>100200300400</div></div>		
1286.4	GROUND LEVEL								
1283.1	SOFT DARK BROWN FIBROUS PEAT	1	D.O.	PM					
1273.4	LOOSE TO COMPACT GREY SAND, SOME GRAVEL.	2	"	21	1280				
1273.4		3	"	15					
1273.4		4	"	59	1270				
1260.0		5	"	100					
1260.0	DENSE TO VERY DENSE GREY GRAVELLY SILTY SAND, NUMEROUS COBBLES AND BOULDERS BELOW ELEV. 1258. (SANDY TILL)	6	"	76	1260				MH
1250.0		7	AXT RC	-					
1250.0		8	D.O.	100	1250				
1240.4		9	AXT RC	-					
1235.9	HARD SOUND PINK AND DARK GREY GRANITE GNEISS BEDROCK.	10	"	-	1240				
1235.9	END OF HOLE				1230				

1510

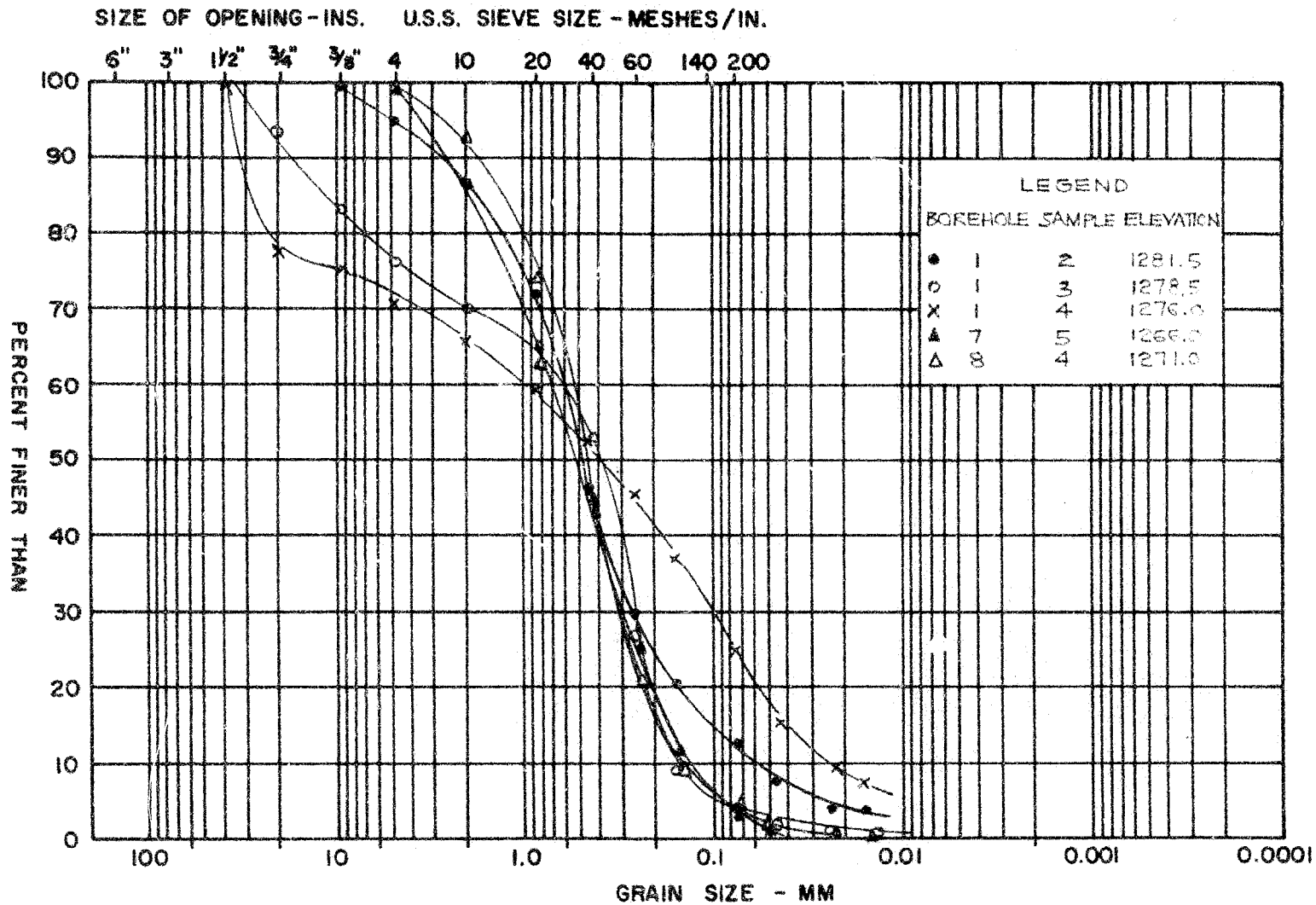
Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN me
CHECKED J. J.

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

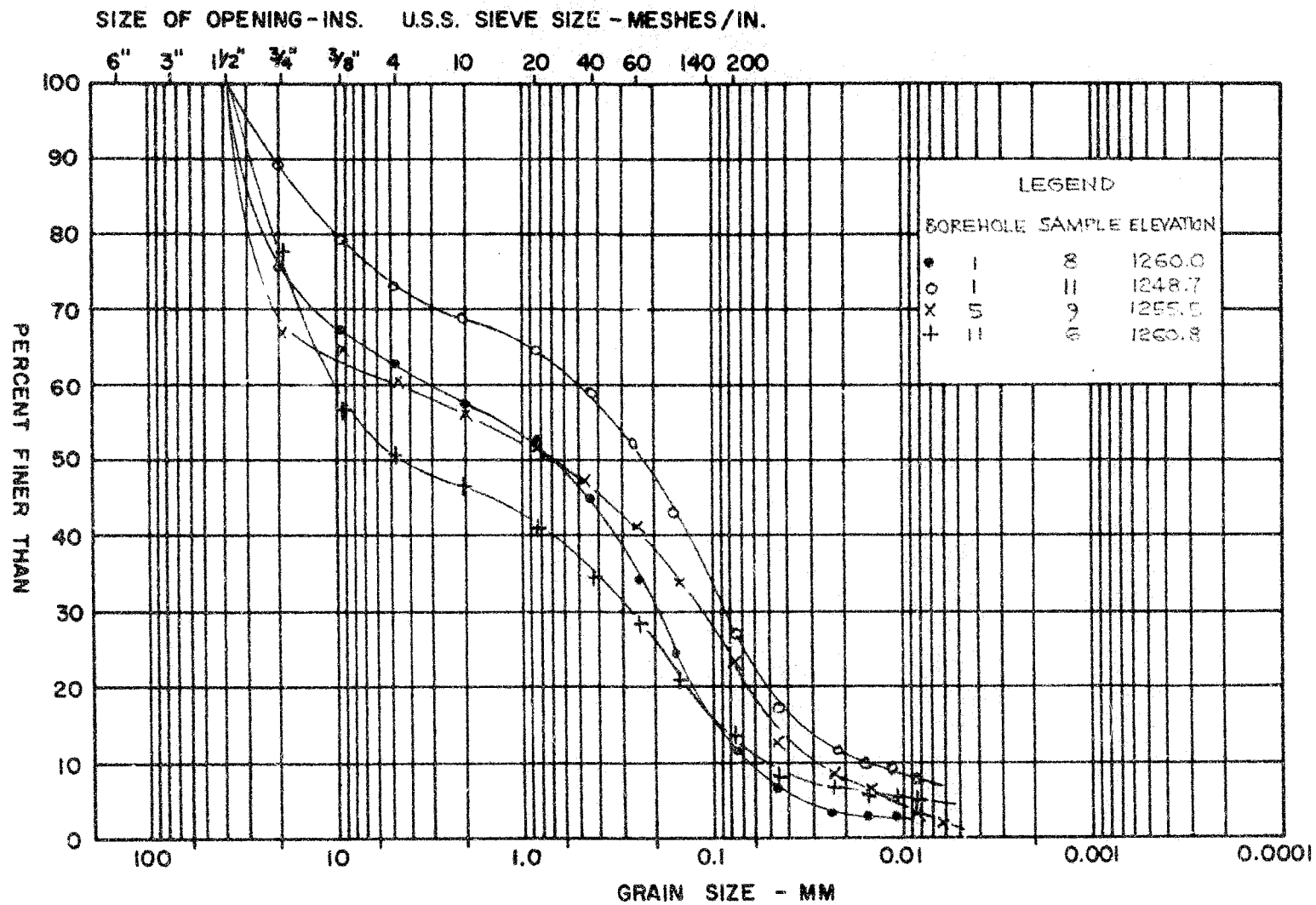
GRAIN SIZE DISTRIBUTION
(SAND)

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
(SANDY TILL)

FIGURE 5

cc: Mr. E. Szymanski

Eng. 401 & Leslie S.
Brampton, Ontario

May 30, 1966

Materials and Testing Division

M. J. Galter and Associates Ltd.,
2444 Elcor Street West,
Toronto, Ontario.

Attention: Mr. J. L. Szychak

- Re: Foundation Investigations - Letter of Authority -
- (1) S.P. 191-64 - The Battershall Falls Bridge,
Eng. No. 35, Proposed Highway, Line 'G', Dist. #11.
 - (2) S.P. 178-64 - The Scotch River Bridge,
Eng. No. 35, Dist. #11.
 - (3) S.P. 66-64 - Site #11-3, ✓
Papineau Creek, Eng. No. 127, Dist. #10.

Dear Sir:

This is to authorize you to carry out the foundation investigations at the above mentioned sites. The plans and all the necessary information pertaining to the jobs were given to your Mr. J. L. Szychak on May 27, 1966. The names and telephone numbers of personnel to be contacted in connection with survey information and/or assistance, were also given to Mr. Szychak.

The urgency of these investigations was discussed, and it was arranged that two of the investigations will be started on Monday, May 30, 1966, and the third one, immediately upon completion of the investigation closest to it.

You are requested to contact our office as soon as enough information becomes available and a meeting can be held with the designer. The final reports (10 copies of each project) will follow at a later stage; however, every effort should be made to have them delivered to our office as soon as possible.

E. C. Golder and Assoc. Ltd.
Attn: Mr. J. L. Seyamuk

- 2 -

May 30, 1966

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

Charges for the work performed will be in accordance with your Schedule of Rates, dated October 1, 1965, and invoices to be addressed to the attention of the undersigned.

We are attaching the following purchase orders:

J 34810 - A.R. 191-64 (Castroville Mills Bridge),

J 34811 - A.R. 190-64 (Sanak River Bridge),

J 34812 - A.R. 191-64 (Explosion Area site 41-5).

Regarding the purchase of any new material required for this work, in order that you may use these as a basis for exemption from the Federal Tax for such purchase. The Exemption Certificate is printed thereon.

Yours very truly,

400/1008
Attn:

A. E. Egan,
ARTIFICIAL & TESTING ENGINEER

cc: Messrs. E. C. Golder
A. E. Egan
W. H. Miller
J. L. Jones
J. L. Williams
J. L. Brown
J. L. Gruesley
re. J. L. Gruesley
W. H. Miller
J. L. Jones
J. L. Williams (2)
Foundations Office
San. 110 (2)

DEFECTS IN NEGATIVE COPY
CONDITION OF ORIGINAL DOCUMENT

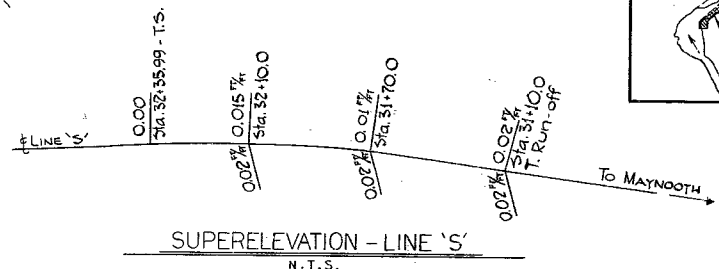
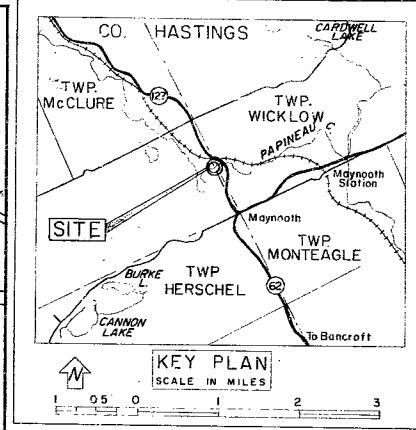
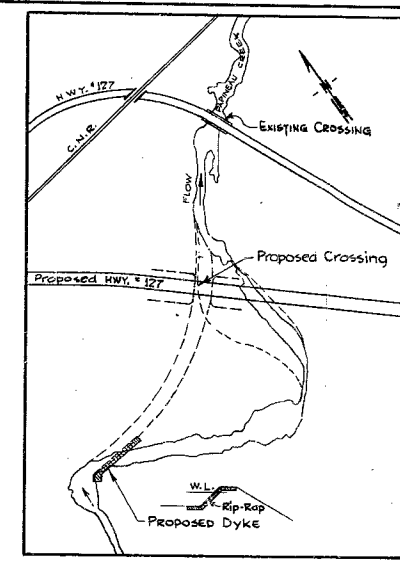
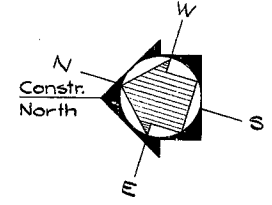
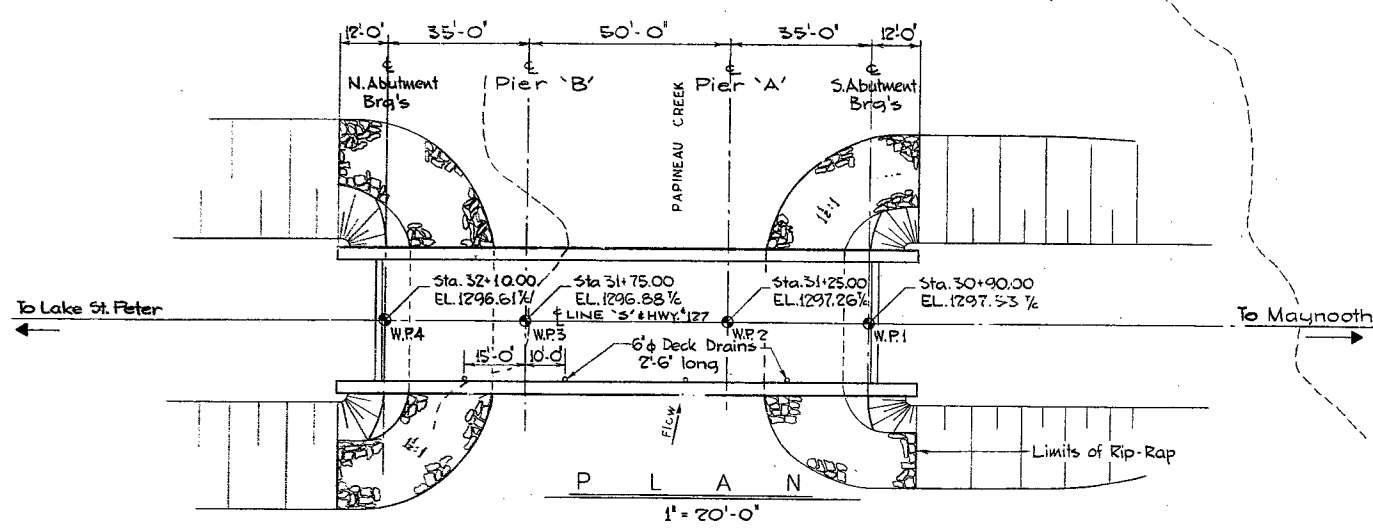
66-F-221-C

W.P. # 66-64

HWY. # 127

PROP. PAPINEAU

CREEK CROSSING

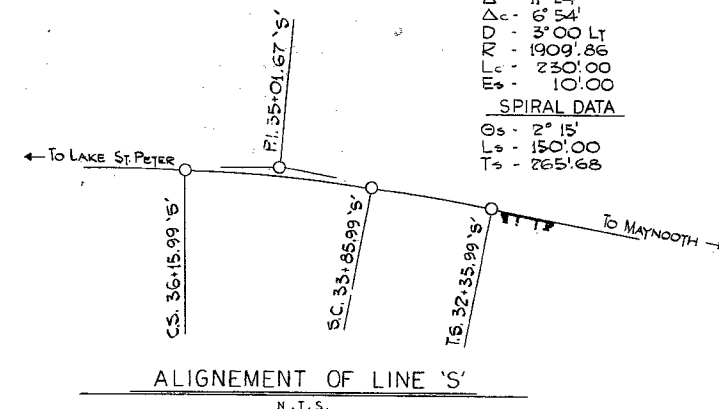
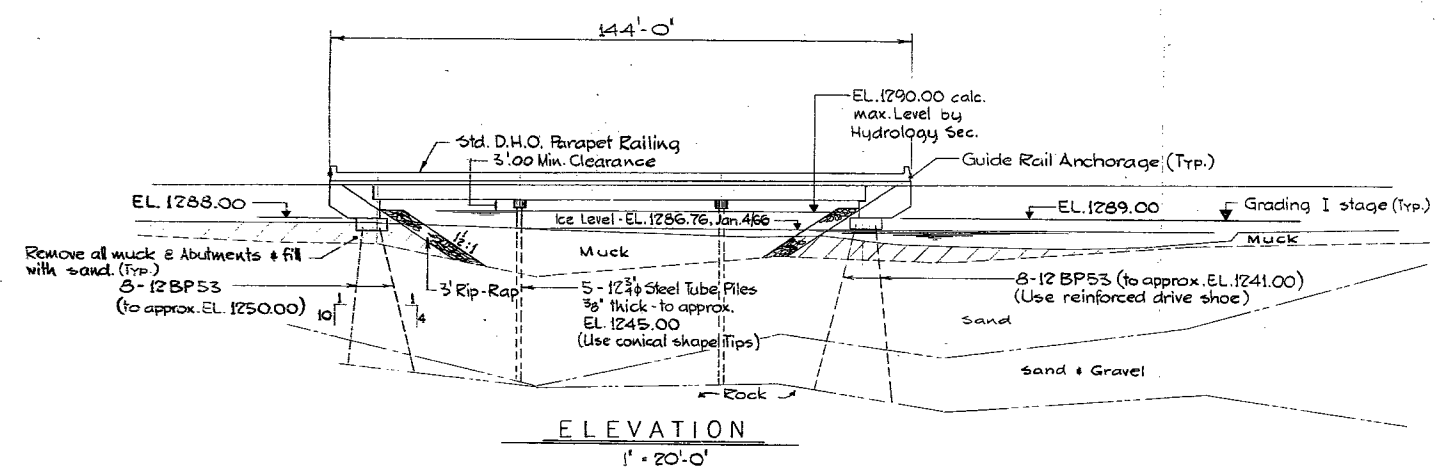


LINE 'S' CURVE DATA

Δ	11° 24'
Δc	6° 54'
ΔD	3° 00' LT
ΔE	1909.86
ΔF	2301.00
ΔG	101.00

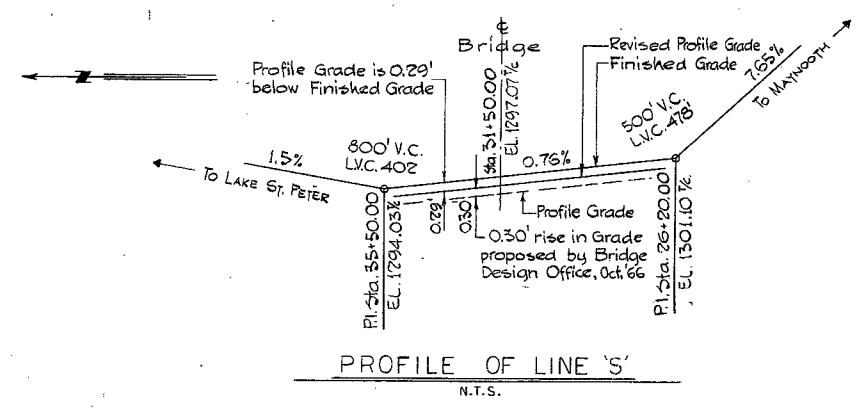
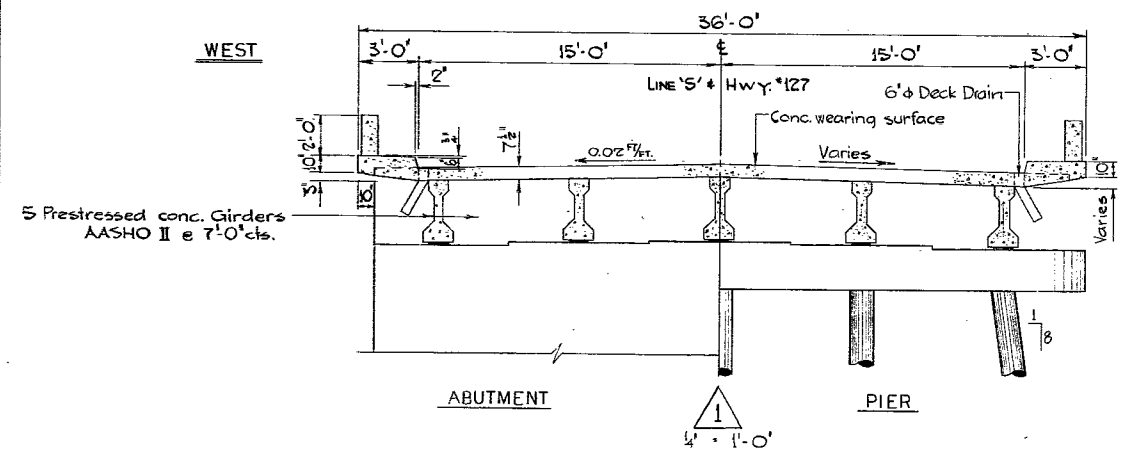
SPIRAL DATA

ΔS	2° 15'
ΔT	1501.00
ΔU	2651.68



G.B.M. N° 342-G, EL. 1297.30
Conc. box culvert under C.N. Ry., 700' North of Station. North end of West Face, 10 inches below coping. Bolt set horizontally.
PUBLICATION 19, MAYNOUTH

REVISIONS	DATE	BY	DESCRIPTION



DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

PAPINEAU CREEK
SOUTH BRIDGE
(0.5 MILES NORTH OF HWY# 62)
KING'S HIGHWAY No. 127 DIST. No. 10
CO. OF HASTINGS
TWP. OF WICKLOW LOT 5 CON. E. H. R.

PPELIMINARY

APPROVED
DESIGN 76
DRAWING T.T.B.
DATE Oct. 66

BRIDGE ENGINEER
CHECK
LOADING HS20-44

CONTRACT No.
DRAWING No.
D-6021-P1

FILE No. 11-5
W.P. No. 66-64

