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GEOCRES No. 31G5-118

W.P. No. _____

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W. O. No. _____

STR. SITE No. 3-279

HWY. No. _____

LOCATION CEDARVIEW Rd. &
JOCK RIVER

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. NONE

REMARKS: _____

Site 3-279

3165-118

GEOCRES No.

REPORT

TO

C. C. PARKER AND ASSOCIATES LIMITED

ON

PROPOSED CEDARVIEW ROAD BRIDGE

JOCK RIVER

TWP. OF NEPEAN

ONTARIO.

Lot 13, Con. III & IV

Distribution:

4 copies - C. C. Parker and Associates Limited,
Ottawa, Ontario.

2 copies - H. Q. Golder & Associates Ltd.,
Ottawa, Ontario.

September, 1968.

GOLDER & ASSOCIATES

68772

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ABSTRACT

The results of an investigation to determine the subsoil conditions at the proposed Cedarview Road crossing of the Jock River in the Township of Nepean, Ontario, are reported and recommendations are made for foundation design and construction of the proposed bridge structure and approach embankments.

It was found that the river banks are underlain by 5 to 10 feet of surficial alluvial deposits consisting of very loose to loose silty sand and organic silt. These alluvial deposits are underlain by some 55 feet of silty clay, the upper 40 feet of which is soft to firm in consistency and sensitive to disturbance in nature. The clay is underlain by sound limestone bedrock. On reaching the bedrock, water rose in the casing to some 9 feet above the river water level.

It is recommended that the piers and abutments for this structure be founded on piles driven to end bearing on the limestone bedrock. In order to limit the disturbance of the sensitive clay during pile driving it is recommended that steel H piles be employed. In the piling for the abutments, it is recommended that batter piles be installed in a direction away from the structure, as well as in the conventional direction, to resist any movement of the abutments towards the center of settlement below the approach embankments.

The stability of the approach embankments both on the front slope and the side slopes is considered adequate. It is estimated that as much as 1 foot of settlement will occur below the approach embankments. The time period between embankment and bridge construction should be maximized to allow a good portion of this settlement to take place.

INTRODUCTION

H. Q. Golder & Associates., have been retained by C. C. Parker and Associates Limited, Consulting Engineers to the Township of Nepean, to carry out a soil investigation for a bridge crossing of the Jock River along the Cedarview Road alignment. The purpose of this investigation was to determine the subsoil conditions at the site and, based on this information, to make recommendations for the foundation design and construction of the proposed structure and to assess the stability of the approach embankments.

PROCEDURE

The field work for this investigation was carried out between July 29 and August 1, 1968. Two boreholes were put down at the abutment locations using a machine drill rig supplied and operated by the F. E. Johnston Drilling Co. Ltd., of Ottawa. These boreholes encountered some 55 feet of relatively soft clay, then limestone bedrock. In order to check the uniformity of the depth to bedrock, a dynamic penetration test will be put down this winter at each pier location when the river has frozen sufficiently to support a drill rig. The bedrock was cored in AXT size in the boreholes. The field work was supervised by a member of our engineering staff.

The location of the borings, together with a soil stratigraphy section, are shown on Figure 1. Detailed logs for

the borings are given on the Record of Borehole sheets following the text of this report.

The samples obtained during the investigation were brought to our laboratory for examination and testing. The results of the tests are shown on the Record of Borehole sheets and on Figures 2 and 3. A plot of shear strength versus elevation is given on Figure 4.

The elevations given in this report are referred to a bench mark located on the north root of a 2.5 foot elm tree, 125 feet left of Station 106+00. The elevation of this bench mark was given to us as 300.56 as referred to Geodetic datum.

SITE AND GEOLOGY

The bridge crossing is to be located at the intersection of the Jock River with the extension of the Cedarview Road alignment, about $1\frac{1}{2}$ miles east of the Moody Drive bridge. The land adjoining the river is relatively flat and the river has cut a channel some 10 feet into the plain area.

From available geological information it is known that the site is underlain by a considerable thickness of overburden, then by limestone bedrock of the Ottawa formation.

SOIL CONDITIONS

The detailed soil stratigraphy encountered in the boreholes is given on the Record of Borehole sheets. Following

is a summarized account of the soil conditions.

Surficial Deposits

The river banks are underlain by 5 feet of loose brown silty sand, a flood plain or alluvium deposit. This deposit is underlain in borehole 2 by 6 feet of very loose organic silt, which is probably also a recent river deposit. The results of a grain size distribution test on a sample of the organic silt are shown on Figure 2.

Clay Strata

The surficial deposits are underlain by extensive strata of clay. To a depth of about 50 feet below ground surface, the site is underlain by a very sensitive silty clay which contains some organic material. Based on the ratio of the undisturbed vane tests to the remoulded vane tests, the sensitivity of this clay is about 20. Atterberg limits carried out in this sensitive clay gave liquid limit values of 33 to 44 and plasticity indices of 13 to 25. The water content of the clay is generally about 15 percent above the liquid limit and the liquidity index generally about 1.7, typical for a very sensitive clay in the Ottawa area. The shear strength of the clay is plotted versus elevation on Figure 4. The in situ vane shear values range from about 400 lb/sq.ft. to about 800 lb/sq.ft. with depth. A consolidation test was carried out on a sample of this clay and

the results are shown on Figure 3. The results indicate that the clay is near normally consolidated; the preconsolidation pressure of this sample from 30 foot depth is considered to be about 1.2 tons/sq.ft., which is about 0.3 tons/sq.ft. above the present overburden pressure for this sample. The compression index, C_c , of this sample is 0.8.

This sensitive silty clay is underlain at a depth of 50 feet by about 15 feet of another clay stratum, less sensitive in nature. The shear strength of this lower silty clay is about 1000 to 1500 lb/sq.ft.

Limestone Bedrock

At a depth of about 65 feet, the river banks are underlain by limestone bedrock. The limestone bedrock contains some dark grey layers of shale. Full core recovery was obtained in drilling the bedrock and based on this, together with visual examination of the core, it is considered that the bedrock is sound.

GROUNDWATER CONDITIONS

On reaching the bedrock in borehole 1, the water level rose in the casing to 3 feet above ground level, that is, to elevation 302. A piezometer was placed in the clay in borehole 1 but the bottom bentonite seal formed in the artesian flow is faulty and the piezometer is measuring the artesian pressure

from the bedrock. On reaching the bedrock in borehole 2, the water level again rose in the casing to elevation 302, which is some 9 feet above the river water level at the time of the investigation.

PROPOSED BRIDGE STRUCTURE

a) General

It is understood that a three span bridge, 160 feet in length and about 30 feet in width, is planned for the river crossing. The roadway grade will be raised to elevation 307, which is some 7 feet above the general ground surface in the area.

b) Foundations

The clay stratum at and below river bottom elevation has a shear strength of about 500 lb/sq.ft., which is too low to support the bridge foundations. It is therefore recommended that the foundation loads be transferred to the underlying limestone bedrock by the use of end bearing piles. The clay deposit is sensitive to disturbance and significant loss of strength in the clay stratum due to pile driving may endanger the slope stability of the approach embankments. In order to minimize disturbance of the sensitive clay deposit, it is recommended that steel H piles be used. For design purposes, the allowable load on a 12 inch steel H pile, driven to a set of 20 blows per inch on the bedrock with a hammer developing in excess of 20,000 ft.lb. of energy per

blow, may be taken as 70 tons per pile. The clay stratum will give little horizontal support to the piles and the lateral loads on the abutments should be taken by steel H piles driven on a batter. A significant amount of settlement will result from consolidation of the clay stratum under the approach embankment loading. It is recommended that batter piles be installed in a direction away from the structure as well, to resist any movement of the abutments towards the center of settlement below the approach embankments.

The closed end abutments and wing walls should be backfilled for a distance of at least 5 feet horizontally with a free-draining and non-frost-susceptible granular material. In this case, to improve the embankment stability in the area of the abutment, it is recommended that "one-size" limestone fill, e.g. 3/4 inch size only, be used in backfilling behind the abutments. The unit weight of this "one-size" limestone fill is about 95 lb/cu.ft. and as such it is effectively a light weight fill. Provision should be made for drainage from this backfill to prevent hydrostatic or ice pressure build up behind the walls. With full effective drainage of the backfill, a coefficient of lateral earth pressure at rest, K_0 , = 0.4 should be used for the compacted granular backfill in design of rigid abutments. If some movement of the top of the abutments wall is possible, an active earth

pressure coefficient, K_a , = 0.3 may be used.

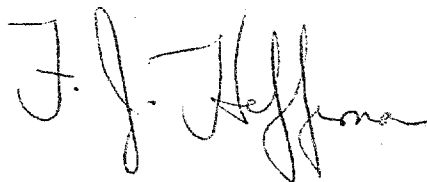
c) Approach Embankments

The grade of the approach embankments will be raised to elevation 307, some 7 feet above existing ground level and some 17 feet above the river bottom elevation. Based on the undrained shear strengths measured in the sensitive clay stratum, the factor of safety against instability of the front slope would be about 1.5, which is considered adequate. The side slopes which are of much less overall height would have a higher factor of safety against instability.

From the results of the consolidation test together with the shear strength profile of the clay, it is believed that the proposed embankment loading approaches or will slightly exceed the preconsolidation load for the clay. Based on the settlement results of other embankments in the Ottawa area loaded to near the preconsolidation load, the settlement of the clay stratum could reach a value of 2 percent of its thickness over a period of about 5 years. At this site, where the relatively soft clay is some 40 feet thick, this would entail a maximum settlement of some 9 to 12 inches at a point some 75 to 100 feet behind the abutments. The settlement of the embankment front edge adjacent to the abutments would be somewhat less, estimated to be about 6 to 9 inches. This would require periodic

maintenance in adjusting the approach grade adjacent to the pile supported abutment. It is recommended that the embankments be built well in advance of bridge construction to allow a good portion of this settlement, due to embankment loading, to take place. The embankment settlement at the abutment could be reduced by the generous use of light weight "one-size" limestone fill to some 40 or 50 feet behind the abutment.

The protection of the embankment side slopes against erosion should be provided to some 2 feet above maximum flood level. In this case, boulders graded from about 1 foot to 6 inch diameter and dumped on the side slopes should provide adequate protection.



F. J. Heffernan, P. Eng.

FJH/ml

68772

September 30, 1968.



GOLDER & ASSOCIATES

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_s	coefficient of consolidation
T_v	time factor = $c_s t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion intercept
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

*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

LOCATION

See Figure

BORING DATE JULY 29 & 30, 1968 DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER BX \neq AX

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST	HAMMER WEIGHT	LB.	DROP	INCHES
1	10	10	10	10
2	10	10	10	10
3	10	10	10	10
4	10	10	10	10
5	10	10	10	10
6	10	10	10	10
7	10	10	10	10
8	10	10	10	10
9	10	10	10	10
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85	10	10	10	10
86	10	10	10	10

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT.				COEFFICIENT OF PERMEABILITY K, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
LEV. N EPTH	DESCRIPTION	STRAT. PLCT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH C_u , LB./SQ. FT. VANE: + NAT. & REM.				WATER CONTENT, PERCENT					
							500	1000	1500	2000	W _p	W	W _L	20		
	TOPSOIL															
198.9	GROUND LEVEL					200										
0.3	LOOSE BROWN SILTY SAND, TRACE OF OF CLAY (ALLUVIUM)		1	SC	4											
193.9			2	"	2											
5.0	STIFF TO FIRM BROWN SILTY CLAY		3	"	PM	290	+									
287.9			4	"	PM		+									
11.0			5	"	PM	230	+									
	SOFT TO FIRM GREY SENSITIVE SILTY CLAY TRACE TO SOME ORGANIC MATERIAL		6	"	PM		+									
			7	"	PM		+									
			8	"	PM		+									
			9	"	PM		+									
251.9			10	"	PM	250		+								
43.0			11	"	PM	240		+								
	FIRM TO GREY SILTY CLAY 2' BOULDER AT 35' DEPTH		12	"	PM											
233.2			13	"	PM											
65.7	SOUND GREY LIMESTONE BEDROCK, SOME DARK GREY SHALES LAYERS		14	"	PM											
			15	"	PM											
222.0																
76.9	END OF HOLE															

GROUND SURFACE

SURFACE SEAL

SAND FILL

POLY TUBING

BENTONITE SEAL

PIEZOMETER

BENTONITE SEAL

SAND FILL

W.C. II CASING AT ELEV. 302 JULY 30, 1962 ARTESIAN FLOW BROKE THROUGH LOWER BENTONITE SEAL, WATER OVERFLOWING FROM TUBING

EST. 95 DRILL WATER RETURN
% CORE RECOVERY

15-5 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'

GOLDER & ASSOCIATES

DRAWN D.N.
CHECKED F.J.H.

RECORD OF BOREHOLE 2

See Figure

BORING DATE JULY 31 & AUG 1, 1968

DATUM

WASH. BORING

BOREHOLE DIAMETER N_X & B_X

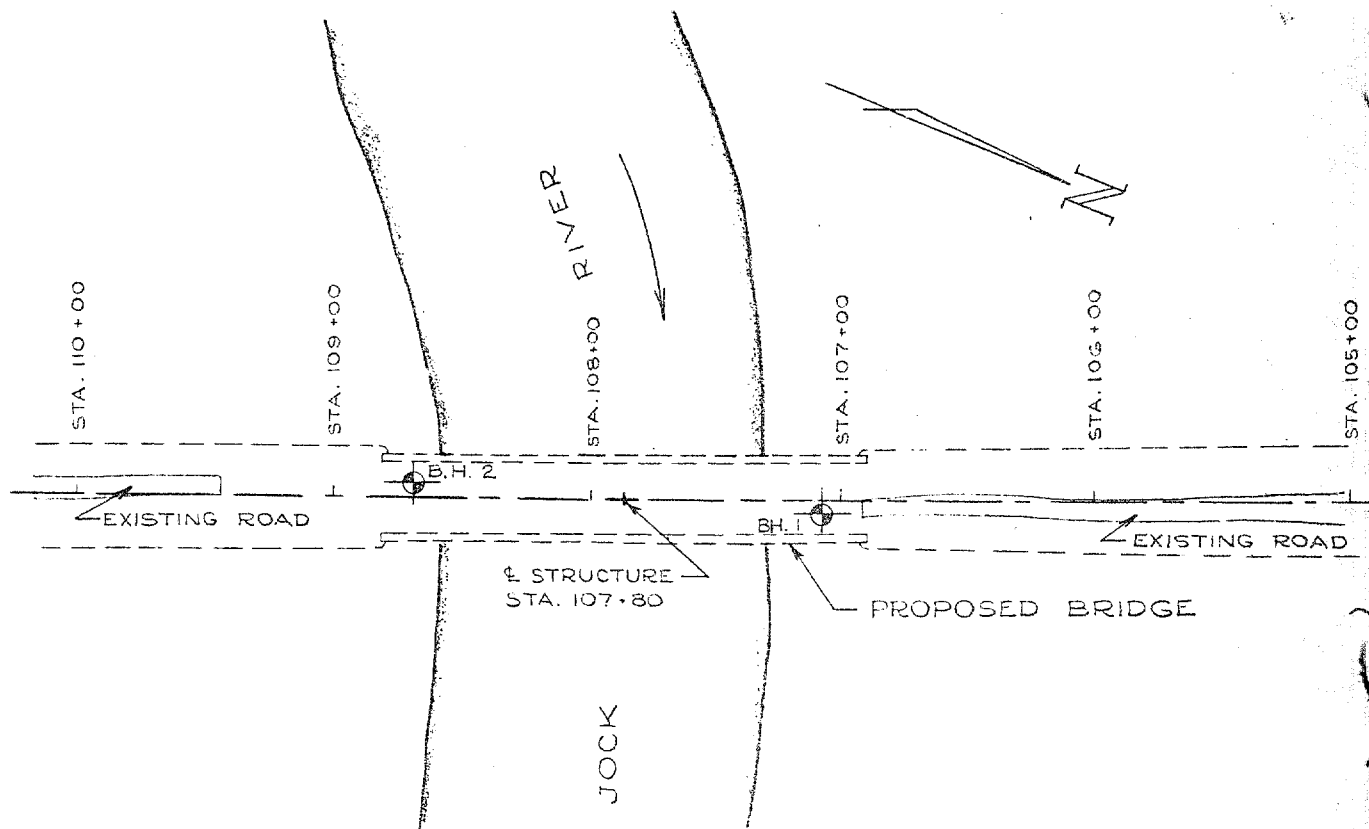
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST	HAMMER WEIGHT	LB.	DROP	INCHES
1	10	10	12	1.5
2	10	10	12	1.5
3	10	10	12	1.5
4	10	10	12	1.5
5	10	10	12	1.5
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8	10	10	12	1.5
9	10	10	12	1.5
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11	10	10	12	1.5
12	10	10	12	1.5
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74	10	10	12	1.5
75	10	10	12	1.5
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85				

[illegible]

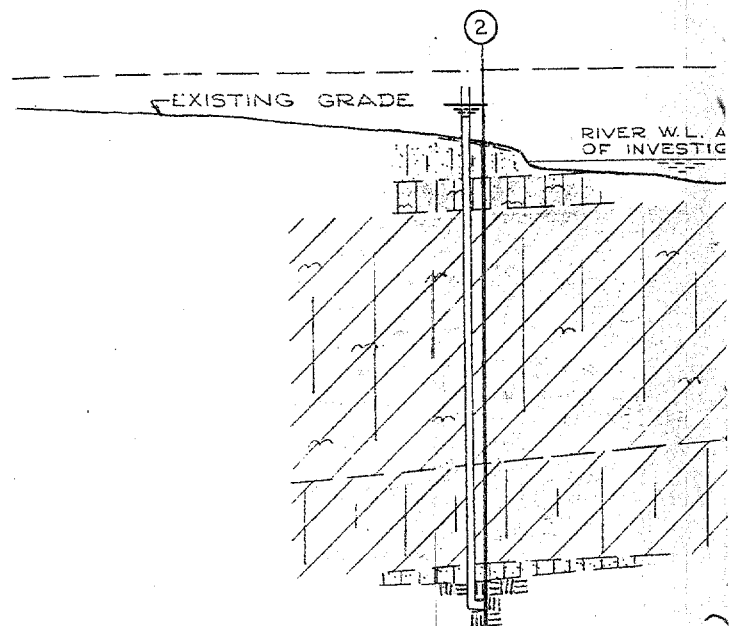
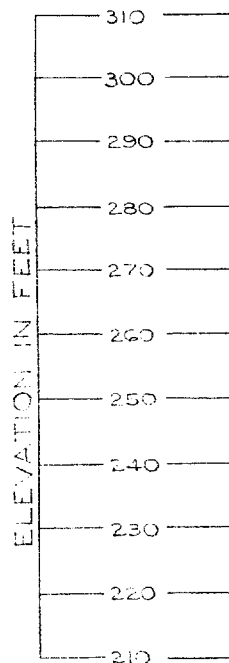
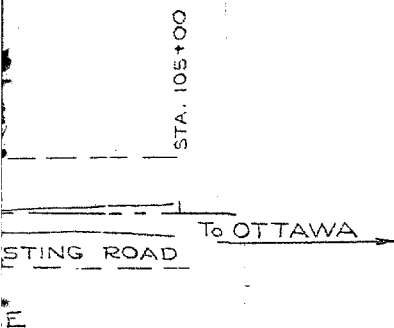
GOLDER & ASSOCIATES

DRAWN P.M.
CHECKED T.T.



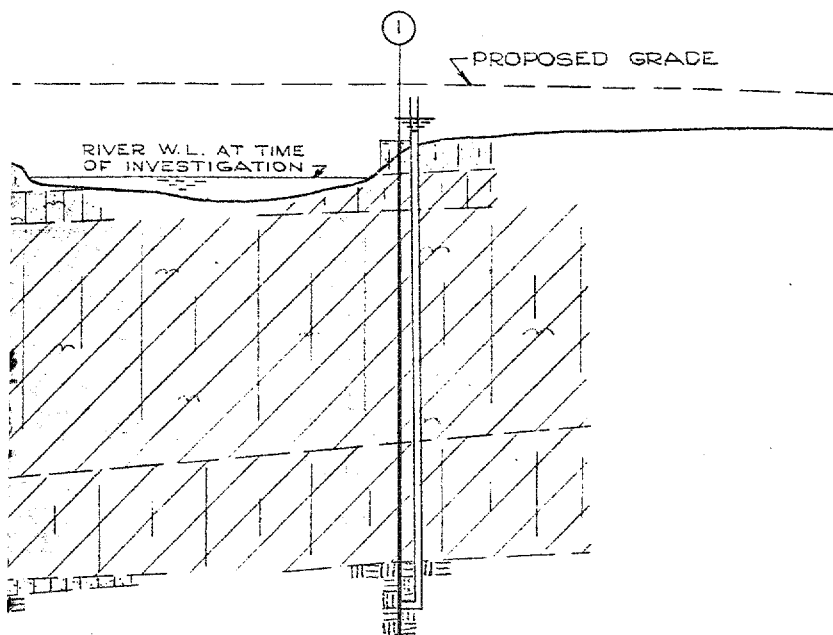
PLAN

SCALE 1" TO 50'



SECTION ALONG

SCALE: HOR.
VER.



SECTION ALONG PROPOSED \perp BRIDGE

SCALE: HOR. 1" TO 50'
VER. 1" TO 20'

STRATIGRAPHY



LOOSE BROWN
(ALLUVIUM)



VERY LOOSE OR
OCCASIONAL FINE



STIFF TO FIRM CLAY



SOFT TO FIRM CLAY
TRACE TO SOME SAND



FIRM TO STIFF CLAY



VERY LOOSE SAND
AND GRAVEL



SOUND GREY CLAY
DARK GREY SAND

LEGEND



BOREHOLE 1



BOREHOLE 2



WATER LEVEL

REFERENCE: PRELIMINARY
DATED SEPT. 1968
SUPPLIED BY
C.C. PARKER

STRATIGRAPHY

LOOSE BROWN SILTY SAND, TRACE OF CLAY
(ALLUVIUM)



VERY LOOSE **ORGANIC** SANDY SILT, SOME WOOD,
OCCASIONAL FINE SAND SEAM.



STIFF TO FIRM BROWN **SILTY CLAY**



SOFT TO FIRM GREY SENSITIVE SILTY CLAY,
TRACE TO SOME ORGANIC MATERIAL



FIRM TO STIFF GREY **SILTY CLAY**



VERY LOOSE GREY **SANDY SILT**, SOME CLAY
AND GRAVEL



SOUND GREY LIMESTONE BEDROCK, SOME
DARK GREY SHALE LAYERS

LEGEND

BOREHOLE IN PLAN



BOREHOLE IN ELEVATION



WATER LEVEL IN ELEVATION

REFERENCE:

PRELIMINARY DRAWING,

DATED SEPT. 16, 1968,

SUPPLIED BY

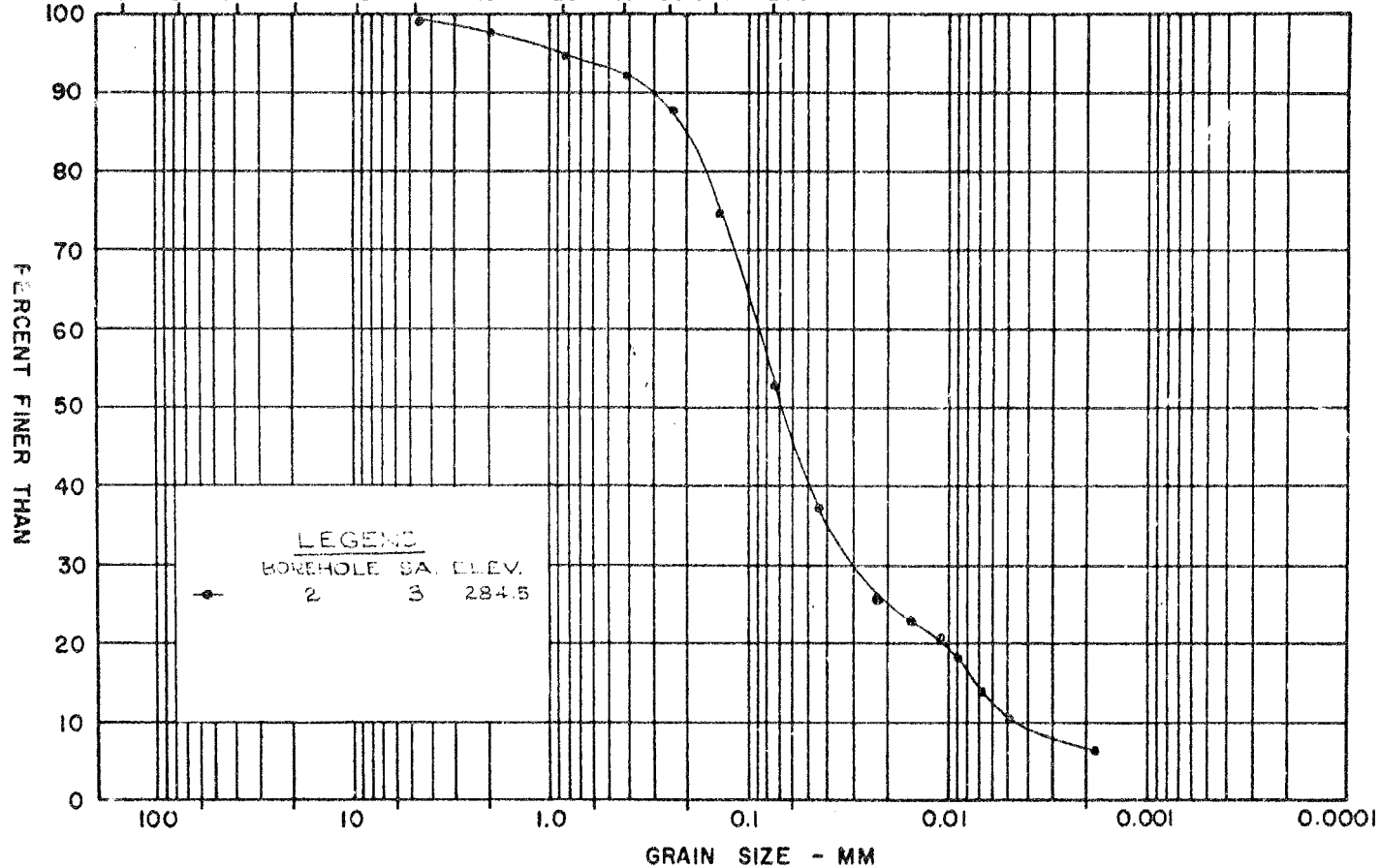
G.C. PARKER AND ASSOCIATES LTD.

3165-118
GOLDER & ASSOCIATES

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



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	

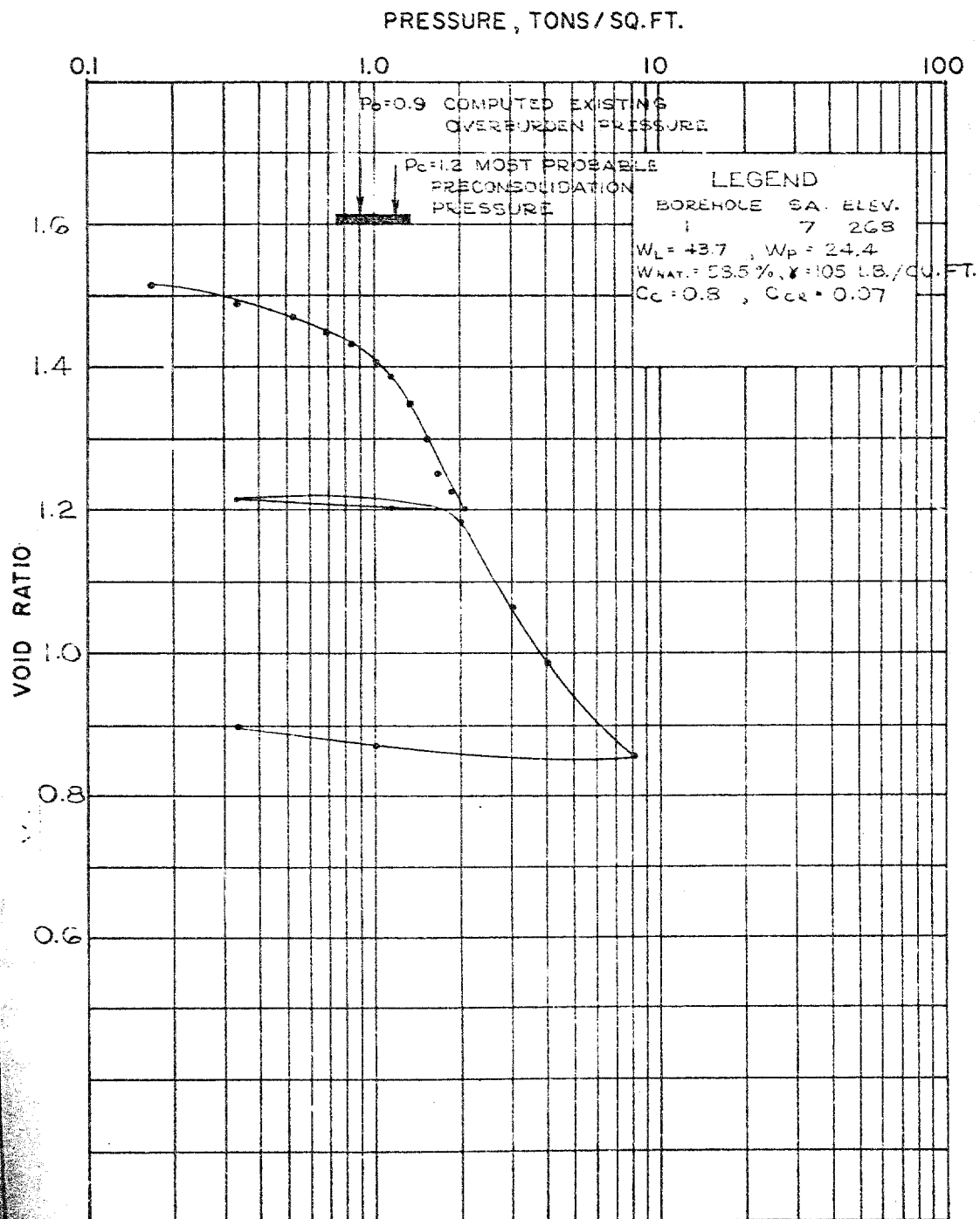
GRAIN SIZE DISTRIBUTION

ORGANIC SILT

FIGURE 2

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 3



SHEAR STRENGTH VERSUS ELEVATION

FIGURE 4

SHEAR STRENGTH IN P.S.F.

