

68 - F - 231 M

CTY. RD # 23

RIVIERE AU BAUDET

BRIDGE

DALHOUSIE MILLS

BA. 2981
Site 31 - 131

H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS
HEAD OFFICE - TORONTO, ONTARIO

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OTTAWA 4, ONTARIO
235-9698

F. J. HEFFERNAN (OTTAWA)

August 20, 1968.

68-F-231M

United Counties of Stormont, Dundas and Glengarry,
County Court House,
Cornwall, Ontario.

Attention: Mr. A. R. Ferguson, P. Eng.
County Engineer.

RE: Soil Investigation,
Proposed Riviere au Baudet Bridge,
County Road #23,
Dalhousie Mills, Ontario.

Dear Sirs:

This letter reports the results of a soil investigation carried out at the bridge crossing of Riviere au Baudet by County Road #23, near the Quebec border and 10 miles north of Lancaster, Ontario. 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 of a proposed bridge replacement structure.

PROCEDURE

The field work for this investigation was carried out between July 9 and July 12, 1968. A borehole was put down

through the south approach embankment and a clay stratum was encountered at and below river bottom elevation, followed by glacial till, then bedrock. A borehole was put down near the river's edge on the north bank of the river to obtain further information on the shear strength of the clay stratum and at the same time to determine the depth to the till and bedrock on that side of the river. The boreholes were put down using a machine drill rig supplied and operated by the F. E. Johnston Drilling Co. Ltd. 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 Figure 2. A plot of shear strength versus elevation is given on Figure 3.

The elevations given in this report are referred to a bench mark located on the west side of the north abutment. The elevation of this bench mark was assumed by us as elevation 100.0

as referred to a local datum.

SITE AND GEOLOGY

The bridge site is located about 10 miles north of Lancaster, Ontario. The topography of the area is relatively flat at and east of the site and gently rolling west of the site.

From available geological information, the Riviere au Baudet enters a clay plain just west of the bridge site. The bedrock is limestone of the St. Martin formation, Ordovician Age.

SUBSURFACE 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.

Embankment Fill

Borehole 1, which was put down through the south approach embankment, encountered 10 feet of stiff clayey fill, containing some sand and gravel material and an occasional boulder.

Firm Silty Clay

The embankment and the river bank area is underlain by a stratum of grey silty clay some 18 feet thick. The upper 5 feet of this clay stratum has been weathered to a stiff, grey brown crust. Atterberg limit test carried out on a sample of

the clay gave a liquid limit value of 59 and a plasticity index of 39. The corresponding water content was 8 percent above the liquid limit value, typical for a relatively soft Leda clay deposit. Test results on a clay sample containing some organic material gave a liquid limit of 82 and a plasticity index of 54.

The shear strengths obtained by vane testing in the boreholes are plotted versus elevation on Figure 3. Based on the shear strength profile, the clay deposit appears to be preconsolidated by about 0.5 tons/sq.ft.

Glacial Till

The clay is underlain at the borehole locations by 6 to 8 feet of very dense granular till. The till consists mainly of sand and gravel sizes, with some silt sizes and some cobbles and boulders. The results of a grading test on a sample of the till are shown on Figure 2.

Limestone Bedrock

Limestone was encountered at elevation 68 in borehole 1 and near full recovery was obtained in a two foot run. The casing came to refusal at elevation 65 in borehole 2 on what is considered to be the bedrock.

A piezometer was sealed into the bedrock and till in borehole 1. The water level in the piezometer on July 12, 1968 was at elevation 95, or 6 feet above the river level. On

reaching the till in borehole 2, the water level in the casing rose 6 feet above ground surface, that is, to elevation 96.

PROPOSED BRIDGE STRUCTURE

a) General

The existing bridge, built about 1910, consists of a steel truss of 87 foot span on concrete abutments. Wood planks form the present decking on the bridge. It is understood that it is planned to replace this bridge structure with a wider structure of about the same span and at the same location. The proposed roadway grade at the bridge has not been finalized at the time of this report though it may probably be raised by about 3 feet over the existing grade.

b) Foundations

The clay stratum at and below river bottom elevation has a shear strength of about 600 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 glacial till or 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. 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 or on a boulder within the granular till 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. Should it be decided that the approach embankments be raised considerably (5 feet), significant settlement would result from consolidation of the clay deposit under the increased embankment loading. Based on the experiences of underpass structures on Highway 401 in the Lancaster area (Stermac, Devata, Selby, 1967) it is recommended that for a grade raise of 5 feet, 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 embankment.

The closed end abutments and wing walls should be backfilled for a distance of at least 5 feet horizontally with

STERMAC, A. G., DEVATA, M., and SELBY, K. G. (1967). "A study of abutment movements of a number of Highway 401 underpass structures between Cornwall (Ont) and the Quebec border. 20th Canadian Soil Mechanics Conference, Quebec, Canada.

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_o , = 0.4 should be used for the compacted granular backfill in design of rigid abutments. If some movement of the top of the abutment wall is possible, an active earth pressure coefficient K_a , = 0.3 may be used.

c) Approach Embankments

It is understood that the grade of the approach embankments will probably be raised by about 3 feet above present roadway grade and some 19 feet above the river bottom elevation. Based on the undrained shear strengths measured in the silty clay stratum at the site, the factor of safety against instability of the front slope and the side slopes would be in excess of 1.5, which is considered adequate.

From the shear strength profile of the clay it is believed that the present embankment loading approaches the preconsolidation load for the clay. In widening the embankment and raising the embankment grade by 3 feet, it is considered that several inches of settlement will occur over a period of a few years. Raising the grade by more than 5 feet will result in an embankment loading well in excess of the preconsolidation load of the clay with subsequent major settlement of this compressible clay deposit. For this reason an embankment grade raise in excess of 5 feet is not recommended at this bridge site.

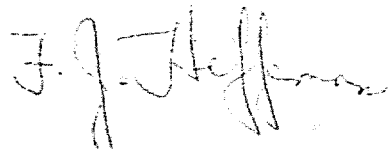
The protection of the embankment slopes against erosion should be provided to some 2 feet above the maximum flood level. In this case, boulders graded from about 1 foot to 6 inch diameter and dumped on the side slopes should provide adequate protection.

We trust that this report contains sufficient information for your design purposes. If we can be of any

further service to you on this project, please call us.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.



F. J. Heffernan, P. Eng.



FJH/ml

68768

August, 1968.

Distribution:

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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

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

WH sampler advanced by static weight—weight, hammer
 PH sampler advanced by pressure—pressure, hydraulic
 PM 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

C consolidation test
 H hydrometer analysis
 M sieve analysis
 MH combined analysis, sieve and hydrometer¹
 Q undrained triaxial²
 R consolidated undrained triaxial²
 S drained triaxial
 U unconfined compression
 V 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 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_i	sensitivity

$\left. \begin{array}{l} \text{in terms of effective} \\ \text{stress} \end{array} \right\} \tau_f = c' + \sigma' \tan \phi'$

$\left. \begin{array}{l} \text{in terms of total stress} \end{array} \right\} \tau_f = c_u + \sigma \tan \phi_u$

*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 1

LOCATION See Figure 1 BORING DATE JUL 1 9 11 45 AM DATUM
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BY CALIPERS
 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. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH C_u , LB./SQ. FT.	WATER CONTENT, PERCENT W _P W W _L			
102.8 G.D.	GROUND LEVEL										
	DARK BROWN SILTY CLAY SOME SAND, GRAVEL AND BOULDER (EMBANKMENT FILL)										WATER TABLE PIEZOMETER SEAL
102.0 G.D.											
101.5 G.D.	STIFF GREY BROWN SILTY CLAY										PIEZOMETER TUBING
101.0 G.D.											
100.5 G.D.											
100.0 G.D.	FIRM GREY SILTY CLAY										PIEZOMETER SEAL
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98.0 G.D.	DENSE BROWN TO RED SILTY SANDY CLAY WITH GRAVEL AND BOULDER (SANDY TILL)										
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GOLDER & ASSOCIATES

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CONDITION OF ORIGINAL DOCUMENT

RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE JULY 12, 1968 DATUM
 BOREHOLE TYPE WASH BOREHOLE BOREHOLE DIAMETER BY 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
ELEVATION DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH C _u , LB./SQ. FT. VALUES + 1000 + 1500 + 2000					WATER CONTENT, PERCENT W _p W W _L					
							500	1000	1500	2000	10	20	30	40			
89.8 0.0	GROUND LEVEL					90											
84.3 5.5	FIRM GREY BROWN SILTY CLAY		1	10	10	85	⊕		+								
			2	10	10	80	⊕		+								
			3	10	10	75	⊕	+									
	SOFT TO FIRM GREY SILTY CLAY		4	10	10	70	⊕	+									
			5	10	10	65	⊕	+									
70.2 19.6	COMPACT TO DENSE GREY SILTY SAND AND GRAVEL (SANDY TILL)		6	10	10	60	⊕	+									
65.4 24.0	END OF BORE REFUSAL TO CASING, PROBABLY BEDROCK					55											
						50											
						45											
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Percent axial strain at failure

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ELEV. 92.7
JULY 12, 1968
(ARTESIAN
PRESSURE)

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15-5 Percent axial strain at failure

VERTICAL SCALE

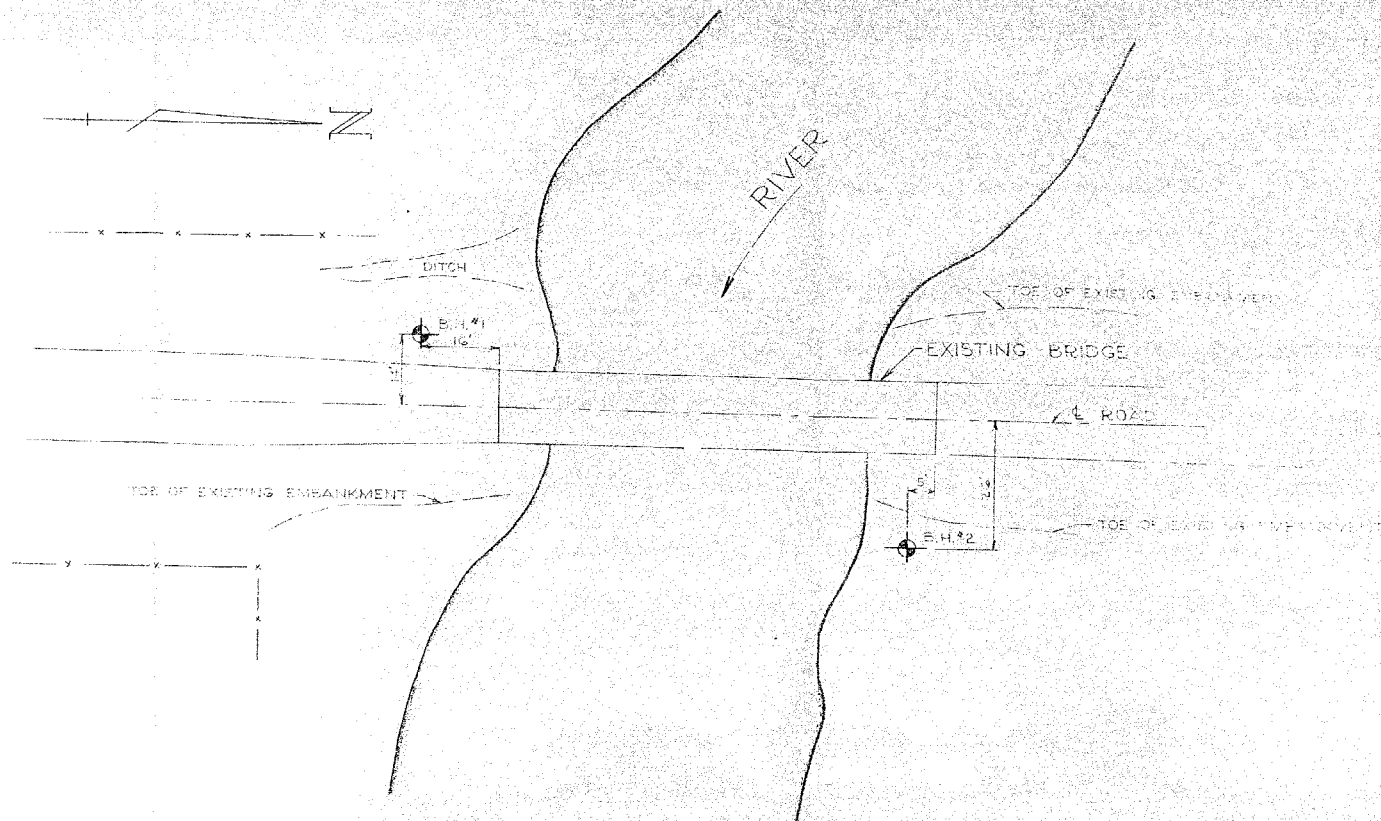
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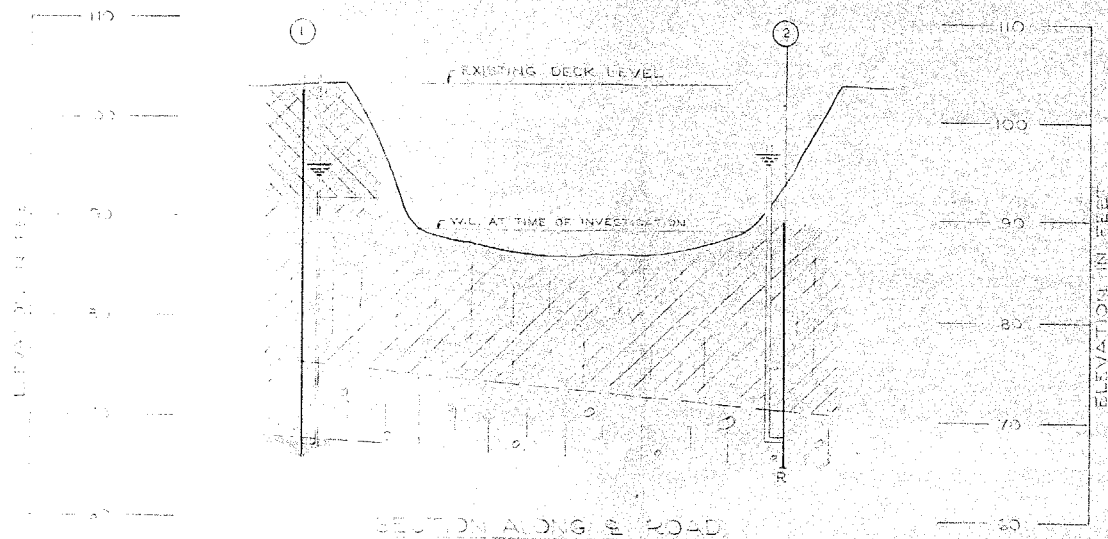
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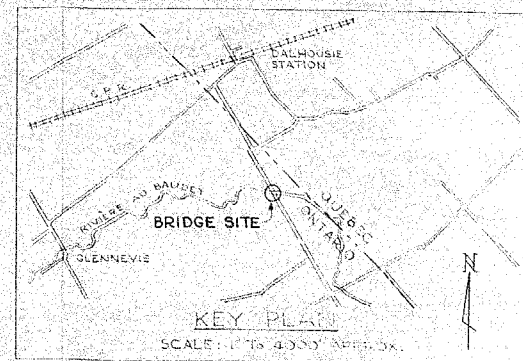
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HORIZ. 1" = 20'
 SCALE: VERT. 1" = 10'

LEGEND

- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- WATER LEVEL IN ELEVATION



STRATIGRAPHY

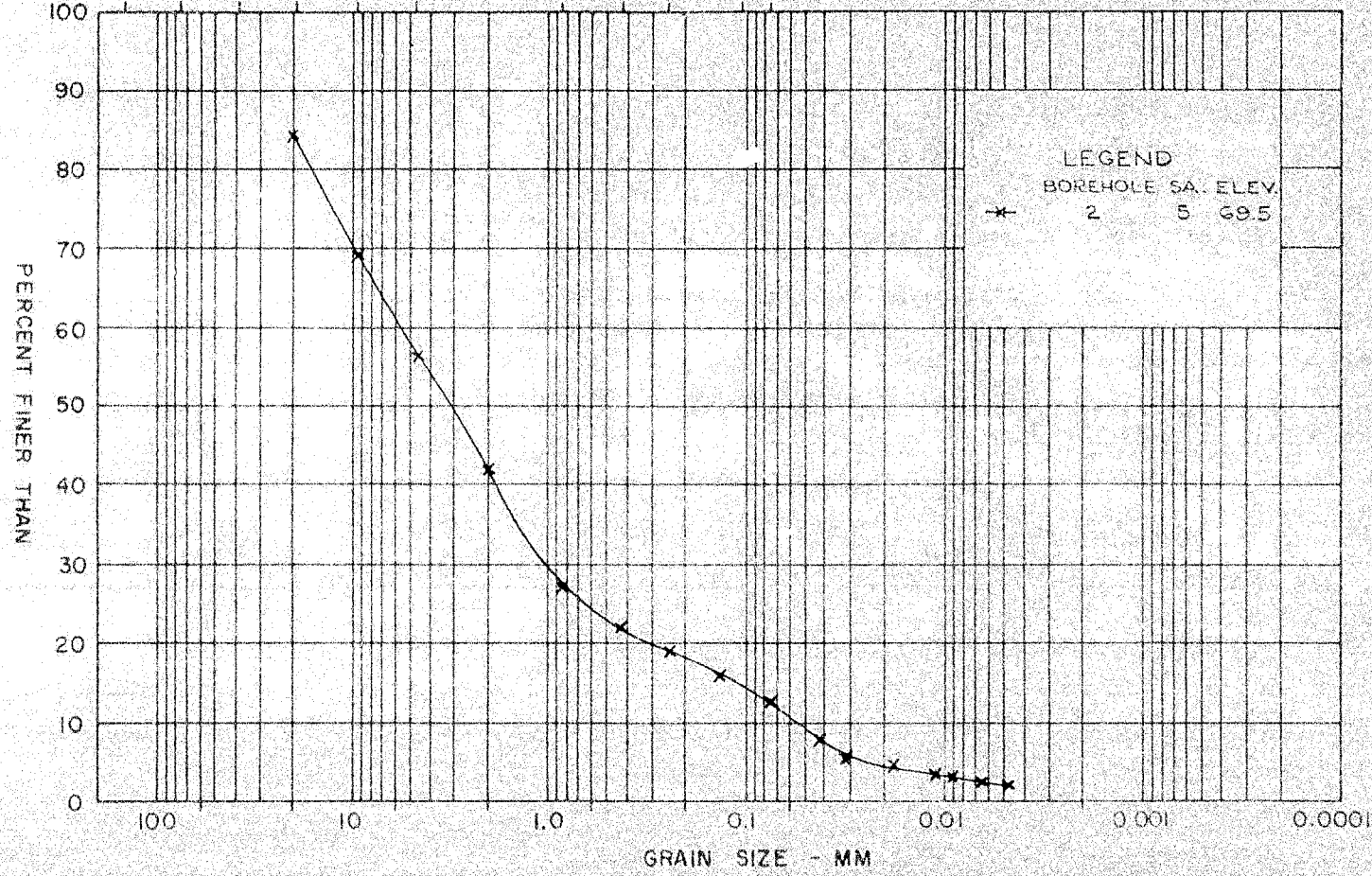
- DARK BROWN SILTY CLAY, SOME SAND, GRAVEL AND BOULDERS (EMBANKMENT FILL)
- FIRM TO STIFF, GREY BROWN CHANGING WITH DEPTH TO SOFT TO FIRM, GREY SILTY CLAY
- COMPACT TO DENSE, BROWN TO GREY SILTY SAND AND GRAVEL, SOME BOULDERS (SANDY TILL)
- SOUND GREY LIMESTONE BEDROCK
- R REFUSAL TO CASING, PROBABLY BEDROCK

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

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 60 140 200



SANDY TILL

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT
GRAIN SIZE DISTRIBUTION

FIGURE 2

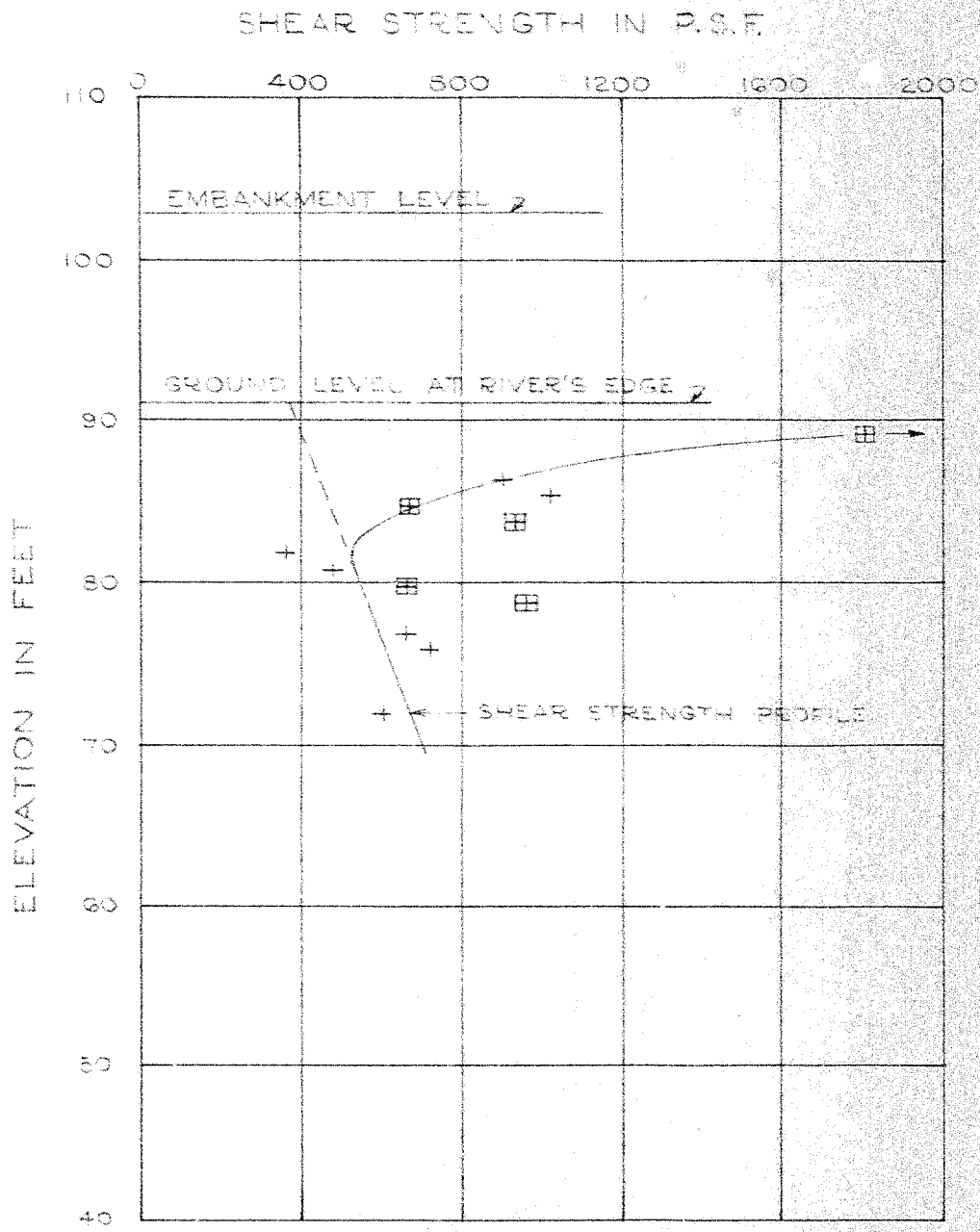
GOLDER & ASSOCIATES

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

PROJECT No. 5875

SHEAR STRENGTH VERSUS ELEVATION

FIGURE 3



LEGEND

- ▣ VANE TEST IN BOREHOLE #1
- + VANE TEST IN BOREHOLE #2

GOLDER & ASSOCIATES

Made D.N.
 Chkd J.
 Appd C.