

64-F-254M

SYDENHAM RIVER

TUPPERVILLE

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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PROJECT SITE No. 13-29

REPORT

TO

LAUGHLIN, WYLLIE & UFNAL

ON

SITE INVESTIGATION

PROPOSED SYDENHAM RIVER BRIDGE

COUNTY OF KENT

TUPPERVILLE

ONTARIO

64-F-254M

Distribution:

**6 copies - Laughlin, Wyllie & Ufnal
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ABSTRACT

The results of an investigation carried out to determine the soil and groundwater conditions at the site of a proposed bridge crossing over the Sydenham River at Tupperville in south-western Ontario are reported. Recommendations are made to ensure the stability of the river banks at the proposed crossing, and for the foundation design of the proposed structure.

The principal stratum at the site is a firm to soft sensitive silty clay some 21 to 37 ft. in thickness. The silty clay is overlain on the east side of the river by about 12 to 13 ft. of stratified organic sandy and clayey silts laid down in the floodplain of the river, and is underlain by a dense silty and sandy till. Shale bedrock underlies the till.

To ensure the stability of the river banks at the proposed crossing, it is recommended that the banks be trimmed to overall slopes of 4 horizontal to 1 vertical with berms at elevation 577 as recommended in the report. Scour or erosion of the river banks should be prevented.

The large diameter caisson to be located in mid-stream should be founded below the upper weathered zone in the shale bedrock. The piers and abutments for the structure should be founded on steel H-piles end bearing in the till or on bedrock.

Special precautions will have to be taken to ensure the safe installation of the large diameter caisson and the pier foundations, as discussed in the report.

INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained by Laughlin, Wyllie & Ufnal to carry out an investigation at the site of a proposed bridge crossing over the Sydenham river at Tupperville, Ontario. The purpose of the investigation was to determine the soil and groundwater conditions at the site and, based on this information, to make recommendations to ensure the stability of the river banks, and for the foundation design of the proposed structure.

PROCEDURE

The field work for the investigation was carried out between February 5 and February 15, 1964. During this time, 6 boreholes were put down by means of a standard machine drilling supplied and operated by the F. E. Johnston Drilling Co. Ltd. Details of these borings, which are located as shown on Figure 1, are given on the Records of Boreholes. A section of the inferred soil stratigraphy along the centreline of the proposed crossing is given on Figure 2.

The soil samples obtained during the investigation were brought to our laboratory for examination and testing. The results of the tests are given on the Records of Boreholes and on the figures.

The borehole elevations given in the report were supplied by the Kent County Engineer's department to Geodetic datum. The borehole locations were referred to the centreline of the proposed bridge and approach embankments as staked by others in the field.

SITE & TOPOGRAPHY

The Hamlet of Tuppersville is located in Gore of Chatham Township, Kent County, Ontario and lies approximately on the boundary between the physiographic regions known as the St. Clair Clay Plains and the Bothwell Sand Plain (Chapman & Putnam, 1951)*. The terrain is level except for the shallow incision of the Sydenham river and its floodplain.

The overburden in this area consists of a shallow sand covering over deep clay deposits laid down during, and following, glacial lakes Whittlesey and Warren. Glacial till underlies the clay and overlies bedrock which is the Huron shale of Devonian Age.

SOIL AND GROUNDWATER CONDITIONS

The floodplain deposit encountered at surface in boreholes 3, 4 and 6 was some 12 to 13 ft. in total thickness

*Reference: Chapman, L. J. and Putnam, D. F., "The Physiography of Southern Ontario", Toronto, 1951.

and consisted of stratified sandy and clayey silts containing some roots, twigs, and pieces of blackened wood and white shells. The lower few feet of the deposit was generally more sandy than the upper portion. Three grading curves for the material are shown on Figure 3. The portion of the deposit above ground-water level generally formed a desiccated crust; however, below water level the material was soft and compressible.

A stratum of firm to stiff mottled brown clayey silt was encountered below about 6 to 10 ins. of topsoil in boreholes 1 and 2 put down on the west bank of the river. This stratum, which was 8 to 10 ft. in thickness, contained some roots in the upper 5 ft. approximately and some sandy pockets throughout. The lower 6 to 9 ins. of the stratum changed to a brown silty sand.

Underlying the floodplain deposits on the east side of the river and the mottled brown clayey silt on the west side was a stratum of generally firm to soft sensitive grey or grey-brown silty clay varying between about 21 and 37 feet in thickness. The silty clay contained some layers of clayey silt, typically 1 to 1½ ins. in thickness, and some small pockets of grey silt. The upper 2 to 4 ft. of the clay in boreholes 1 and 2 at the west side of the river was weathered and desiccated.

The properties of the silty clay are summarized on Figure 6 in a plot of Atterberg limits, natural moisture contents and undrained shear strengths versus elevation. Based on this figure, and on the results of one consolidation test carried out on a sample of the silty clay (see Figure 7), it is inferred that the stratum is normally consolidated or lightly over-consolidated.

The sensitivity of the silty clay, as measured by in situ vane shear tests, varied between about 3 and 11, indicating moderate sensitivity.

A thin stratum of dark grey silty sand and gravel, some 2 to 5 ft. in thickness, was encountered below the silty clay. This material, which is a glacial till, contained numerous dark grey shale fragments and occasional grey sand lenses or layers. Two grading curves for this till are shown as Figure 3. It is inferred from the differences in these curves that the stratum is heterogeneous. Standard penetration tests carried out in the till indicated that except for the upper few inches, where some softening has probably occurred, the till is dense to very dense.

Dark grey shale bedrock was found to underlie the

till in all boreholes except borehole 6 where the boring was stopped after penetrating 1.5 ft. into the till. Based on the results of coring in AXT size (approximately 1-1/8 ins. diameter core) in the rock, it is estimated that the upper 1 to 3 ft. approximately of the shale is weathered in all boreholes except borehole 2 where the depth of weathering is possibly about 5 ft.

Casagrande type porous pot piezometers were sealed in shale bedrock in boreholes 1 and 3 to measure the piezometric water level in the rock. Details of the installations are given on the Records of Boreholes. The water levels in both tubes were at about elevation 568 on February 14, 1964 or some 4 to 5 ft. below ice level in the river at that time. Readings taken in open boreholes 2 and 4 gave water levels at approximately elevations 572 and 574, respectively. This corresponded closely to ice level in the river.

DISCUSSION

River Bank Stability

A visual examination of the natural banks of the Sydenham River in the vicinity of the proposed crossing indicates that although there is some surficial instability of the banks due possibly to river scour and erosion, there is no sign

of recent deep-seated movements. There has, however, been some movement of the east abutment of an existing bridge located about 270 ft. upstream of the proposed crossing. This movement, which was estimated to be about 10 ins. horizontally, has jammed the concrete abutment against the bridge span.

Total stress ($\phi = 0$) stability analyses were carried out for the high west bank of the river and the results of these analyses are summarized on Figure 8. This figure plots computed factor of safety, F , against mobilized undrained shear strength for various slope angles. From the figure, since the existing west bank at the proposed crossing, which has an average overall slope of about 3.5:1, has not failed ($F = 1.0+$) the average undrained shear strength mobilized around a failure arc must be at least of the order of 450 lb/sq.ft. This value agrees with the general pattern of measured undrained shear strength in the silty clay (see Figure 6).

Incidentally, an analysis of the moved east abutment of the existing bridge structure gives a computed safety factor, F , of about 0.8 assuming an undrained shear strength of 450 lb/sq.ft. and neglecting the stabilizing effects of the abutment and the jammed bridge span. Even if we include for these effects, it is unlikely that F would be greater than unity.

This would explain the appreciable movement noted in the abutment.

Computations have been carried out to determine stable river bank sections at the proposed crossing, assuming an average undrained shear strength of 450 lb/sq.ft. Based on these computations, it is recommended that the banks be trimmed to an overall slope of 4 horizontal to 1 vertical drawn tangent to the river bottom and that a berm be provided at elevation 577. Recommended sections for the east and west banks are shown on Figures 10 and 11. The computed safety factor for these sections is of the order of 1.2, or some 10 to 20 percent safer than the slopes now existing at the site. Some slight movement of the bridge abutments may take place, particularly at the east abutment which will be underlain by recent floodplain deposits; however, this movement should not exceed about 1 to 2 ins. and it is recommended that allowance should be made in design to accommodate possible abutment movements of this order. It is not anticipated that the piers will move, provided they are founded as recommended below.

The maximum recorded flood level for the river is about elevation 588, a rise of some 15 ft. above normal water level at elevation 573. A rise of this magnitude could cause considerable scour in fine grained soils in the river banks.

Thus, it is essential that the portions of the river banks below high water level be protected against scour and from erosion due to surface drainage in the vicinity of the bridge structure.

Embankment Stability

The approach earthfill embankment on the east side of the river will be a maximum height of about 8 ft. above present floodplain level. It is considered that, since the upper 7 to 8 ft. thick crust of the stratified sandy and clayey silts in the floodplain is generally compact to loose ('N' values of 7 to 17 blows/ft.), the overall stability of an earth-fill embankment up to the proposed height and having side slopes not steeper than 2 horizontal to 1 vertical should be adequate, provided there are no local soft deposits between the borehole locations.

The embankment will settle due to consolidation of the floodplain deposits and the silty clay under the additional load of the embankment. Consolidation of the upper floodplain deposits should occur largely in the soft compressible organic sandy silt zone of these deposits which was some 3 to 4 ft. in thickness in boreholes 3 and 4 and about 1.5 ft. in thickness in borehole 6. It is estimated that the total settlement in the floodplain deposits should not exceed about 6 to 9 ins.

under the maximum height of fill. The consolidation settlement of the silty clay should be less than 1 in.

The major portion of the settlement should occur by 3 to 6 months after construction of the embankment; however, some minor settlements will probably take place for several years after construction.

Foundations

In view of the low bearing capacity of the upper floodplain deposits and the silty clay stratum, it is recommended that the foundations for the structure be carried down to the shale bedrock encountered in the borings at about elevation 537, some 36 ft. below normal river level.

It is understood that the central pier, which will support the swing spans of the structure, is to be a steel or concrete shell caisson some 20 to 25 ft. in diameter filled with concrete. This caisson should be founded 1 to 2 ft. below the upper weathered zone in the shale. An examination of the rock core recovered from borehole 5, put down at station 8+00 near the proposed caisson location, indicated that the upper 1 ft., approximately, of the rock is weathered at this location. However, since it is difficult to estimate accurately the depth of weathering in a shale by coring it

in AXT size, it is recommended that the above estimate be used only as a guide for design and that the final founding elevation for the caisson be established following a careful examination of the rock during the installation of the caisson. The net bearing pressure at the base of a caisson founded as recommended should be limited to 10 tons/sq.ft.

There are several important construction problems to be overcome in installation of the caisson and these are discussed under "Construction Procedures".

The remaining piers and abutments for the proposed structure should be founded on piles or piers end bearing on rock. In view of the sensitive nature of the clay, non-displacement type piles such as steel H-piles should be used for support of the structure. The design load for 12 BP 53 steel H-section piles driven to a final set of about 15 blows/in. in the shale with a hammer delivering at least 20,000 to 22,000 ft.lb. of energy per blow may be taken as 70 tons per pile.

Settlement of the piers and abutments of the structure, if founded as recommended above, will be negligible.

Construction Procedures

It is anticipated that the two main construction problems will be:

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- i) Installation of the large diameter caisson to support the swing spans, and
- ii) The excavations for the foundations of the piers at either side of the river.

Caisson Installation. It is understood that it is proposed to install the caisson, which will be some 20 to 25 ft. in diameter, and of either steel or concrete shell construction, by excavating out inside the caisson until it is resting at the recommended founding depth below the weathered zone in the shale, then filling it with concrete.

Computations indicate that if a caisson about 20 to 25 ft. diameter is excavated in the dry through the soft silty clay stratum there will be heave of the clay at the bottom of the excavation. To prevent this heave, and consequent loss of ground around the caisson, the caisson should be kept full of water as it is excavated. Once the lower edge of the caisson is in the till, or in the upper portion of bedrock, it may then be possible to dewater the caisson since the till, in view of its gradation and density, should be relatively impervious; however, water may flow into the excavation through sand seams in the till and/or fractures in the shale rock. It is difficult to estimate the quantity of this water, but if it proves to be excessive it may be necessary to carry out the excavation completely under water sealing the base with tremie concrete.

Pier Excavations. It is understood that it is planned to put down two braced and sheeted excavations for the pier foundations on either side of the river. These excavations will be carried down to about 10 to 12 ft. below normal river level which is at elevation 573. There are two principal problems associated with the excavations; the overall stability of the river banks, and bottom heave in the excavations themselves.

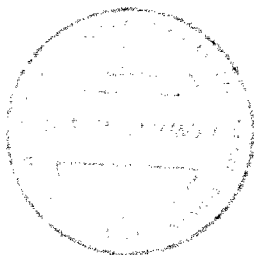
Since the ground surface is presently level adjacent to the proposed location for the pier on the east side of the river, the overall stability of the bank should not be a problem in this excavation, provided the river is not widened and no earthfill for the embankment is placed at the abutment location prior to the excavation being backfilled. The excavation for the pier on the west bank will adversely affect the stability of this bank and it should be trimmed to the section recommended on Figure 12 prior to excavation to provide a reasonable factor of safety against a rotational failure of the bank during the construction period.

The excavations could fail by heave of the soft clay at the base of the excavations. The results of computations carried out to check the factor of safety of a strutted and sheeted excavation of the approximate dimensions shown on

Figure 12 against this type of failure are given on this figure (Bjerrum and Eide, 1956)*.

Considering that during construction, piles will be driven at the base of the excavations, thus partially remoulding the clay at the base of the excavations, it is suggested that the excavation depth be limited to 10 to 12 feet. It is further suggested that the sheeting be left in place following construction.

*Bjerrum, L., and Eide, O., "Stability of Strutted Excavations in Clay", Norwegian Geotechnical Institute Publication No. 19, Oslo, 1956.



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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

<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

Soils

<i>C</i>	<i>c_m 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 *Q* or *R*.

LIST OF SYMBOLS

I. GENERAL

- $\pi = 3.1416$
 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 $= c_v / d^2$ (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
- $\left. \begin{array}{l} \text{intercept} \\ \text{effective angle of shearing resistance, or friction} \end{array} \right\} \text{in terms of effective stress}$
 $\tau_f = c' + \sigma' \tan \phi'$
- $\left. \begin{array}{l} \text{apparent cohesion*} \\ \text{apparent angle of shearing resistance, or friction} \end{array} \right\} \text{in terms of total stress}$
 $\tau_f = c_u + \sigma \tan \phi_u$

*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 1

LOCATION STA 6+22.14 LEFT See Figure 1
 BORING DATE FEB 5, 1964
 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING
 BOREHOLE DIAMETER NX 3 EX 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. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		SHEAR STRENGTH C _u , LB./SQ. FT.					WATER CONTENT, PERCENT					
						+ VANE + REM. V.					W _p W W _L					
						250	500	750	1000	1250						
588.0	TOPSOIL															
588.0	GROUND LEVEL															
580.0	8" BROWN SILTY SAND LAYER ABOVE EL. 580		1	DO	14											
576.0	STIFF MEDIUM-FIRM SILTY CLAY (WEATHERED)		2	DO	16											
576.0			3	DO	17											
576.0			4	DO	20											
576.0			5	DO	21											
576.0			6	DO	21											
576.0			7	DO	21											
576.0			8	DO	21											
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576.0			101	DO	21											
576.0			102	DO	21											
576.0			103	DO	21											
576.0			104	DO	21											
576.0			105	DO	21											
576.0			106	DO	21											
576.0			107	DO	21											
576.0			108	DO	21											
576.0			109	DO	21											
576.0			110	DO	21											
576																

RECORD OF BOREHOLE 2

LOCATION STA 17+05 - 28 RIGHT
See Figure 1

BORING DATE FEB. 7 - 8, 1964

DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX 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 R, CM./SEC					ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV./ DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		SHEAR STRENGTH C, LB./SQ. FT. 1 VANE @ REM. V. 250 500 750 1000 1250					WATER CONTENT, PERCENT Wp — W — Wl						
582.2	CLAYEY SILT		1	DO													
582.2	CLAYEY SILT		2	DO													
582.2	CLAYEY SILT		3	DO													
582.2	CLAYEY SILT		4	DO													
582.2	CLAYEY SILT		5	DO													
582.2	CLAYEY SILT		6	DO													
582.2	CLAYEY SILT		7	DO													
582.2	CLAYEY SILT		8	DO													
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BOREHOLE CAVED
@ EL. 572.3 - FEB. 11,
1964.

15-25 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *M. G.*
CHECKED *J. B.*

RECORD OF BOREHOLE 3

STA. 8431 - 15 (LEFT)

LOCATION See Figure 1

BORING DATE FEB. 14, 1964

DATUM CCELEST

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER 1 1/2 IN. EX. CASING

SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 1 LB DROP 18 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					
							+ VANE + KEM V					W _p W W _L					
						250	500	750	1000	1250							
573.0 0.0	GROUND LEVEL																
	COMPACT TO MEDIUM STRAINED SANDS & SILTS WITH SOME ROCKS AND OCCASIONAL SHELLS		1	DO	13												
571.5 7.7			2	CS	1												
			3	CS	1												
566.8 12.2	SOFT CLUMPY GRAY ORGANIC SANDY SILT CONTAINING SOME WHITE SHELLS AND PEBBLE WOOD		4	DO	5												
			5	TO	PH												
564.5 14.5			6	CS	1												
	LOOSE GRAY MEDIUM AND COARSE SAND TRACED FINE SAND AND SILT AND CONTAINING WHITE SHELLS		7	DO	1												
			8	TO	PH												
			9	DO	1												
			10	DO	1												
	SOFT GRAY BROWN SILTY CLAY BECOMING SANDY BELOW BL 540.5 AND CHANGING TO VERY DENSE DARK GRAY LAYERED SILT, TRACE CLAY BELOW BL 540		11	TO	PH												
			12	DO	1												
			13	DO	PH												
538.0 41.0			14	DO	82												
43.0	DARK GRAY SHALE BEDROCK WITH NUMEROUS FRACTURES UPPER 2' WEATHERED		15	ART RC	-												
528.7 50.3	END OF HOLE		16	-	-												

Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *M.L.*
CHECKED *M.L.*

RECORD OF BOREHOLE 6

STA. 12+55 - CENTRELINE

LOCATION

See Figure 1

BORING DATE FEB. 14 1964

DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX 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.					WATER CONTENT, PERCENT				
							+ - VANE + REM. V.					W _p W W _L				
							250	500	750	1000	1250					
585.5	GROUND LEVEL															
580.0	STIFF TO FIRM MOTTLED BROWN AND GREY CLAYEY SILT. SOME ROOTS. EST. WATER 1-2 FEET		1	2	17											
			3	3	8											
576.1	VERY FINE SAND		4	1	7											
570.5	COMPACT TO VERY CLOSE BROWN SAND WITH FINE LAYERS OF SILT. CLAY SAND. WITH SOME ROOTS. EST. WATER 0.5-1.0 FEET		5	9	9											
			6	10	10											
			7	5	570											
			8	10												
			9	10												
			10	10												
			11	WH												
			12	10												
			13	10												
			14	10												
538.5	FIRM TO VERY FIRM BROWN SILTY CLAY SOME GREY CLAYEY SILT LAYERS ESPECIALLY BETWEEN 541.0 AND 548.5															
537.0	END OF HOLE															
46.5	VERY DENSE DARK GREY SILTY SAND AND GRAVEL (MOSTLY SHALE FRAGMENT) (TILL) FEW SAND LENSES															

15-10 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *M.W.*
CHECKED *J.F.B.*

LOT 27
CONCESSION 1
TOWNSHIP OF CHATHAM GORE

LOT 28
CONCESSION 1

VICTORIA
AVE.

LOT 27
CON. 2

LOT 28

TUPPERVILLE

BANK STREET

CATT STREET

SYDENHAM RIVER

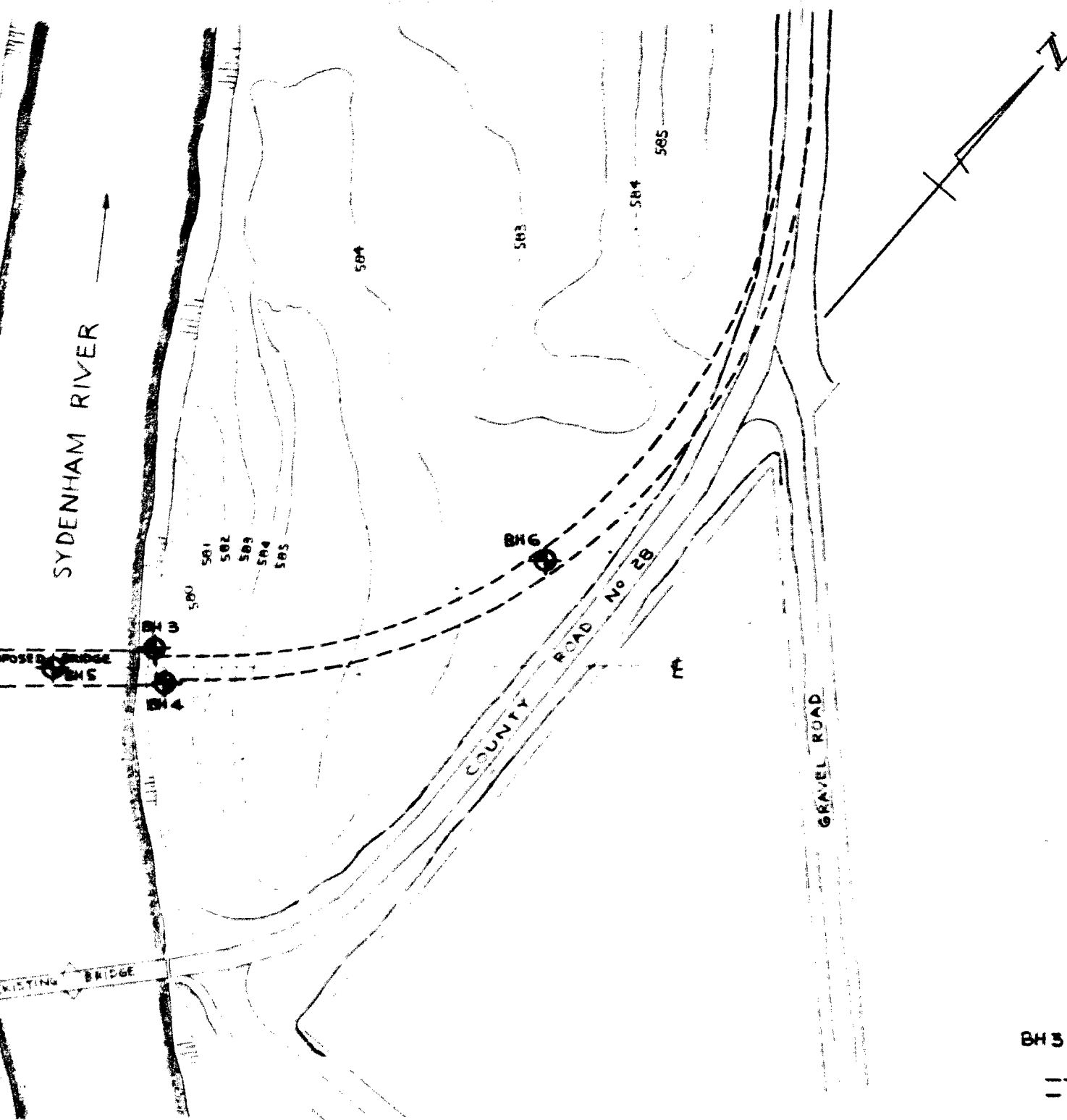
PROPOSED

EXISTING

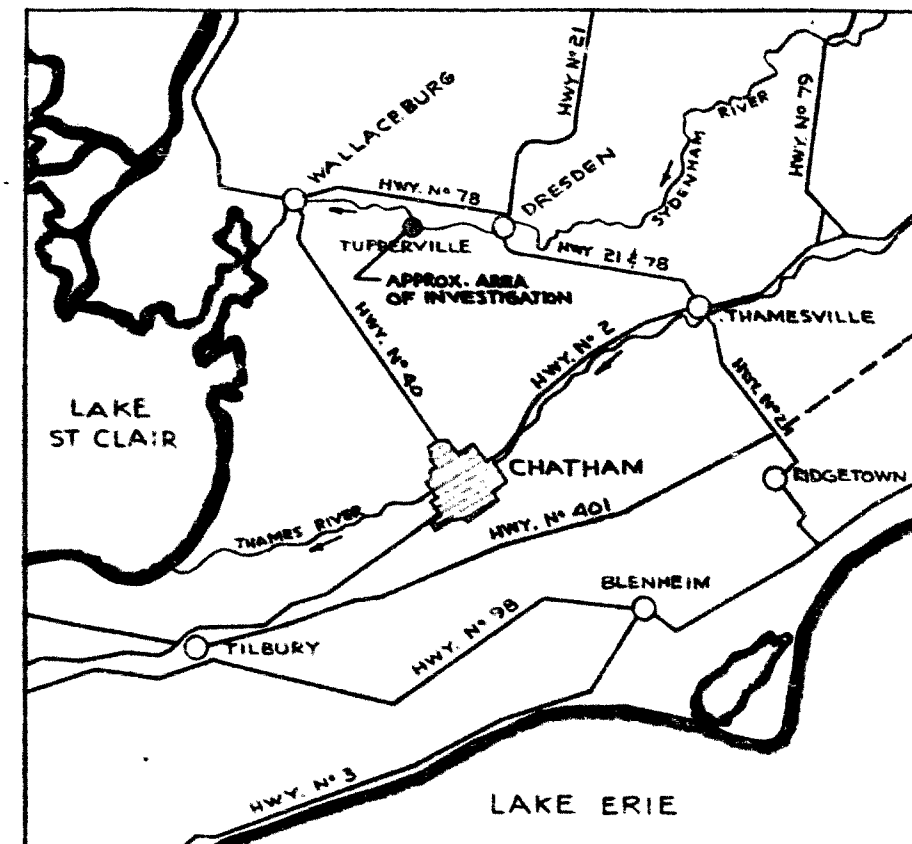
GRAVEL RD.

COUNTY ROAD No 28

C & O RAILWAY



SCALE: 1" TO 100'-0"



KEY PLAN

SCALE: 1" TO 8 MILES (APPROX.)

REFERENCE

PLAN SHOWING PROPOSED BRIDGE CROSSING OVER SYDENHAM RIVER AT HAMLET OF TUPPERVILLE, LOT 28, CONCESSION 2, SORE OF CHATHAM TOWNSHIP - COUNTY OF KENT, DATED DEC. 14, 1963, SUPPLIED BY LAUGHLIN, WYLLIE & UFNAL.

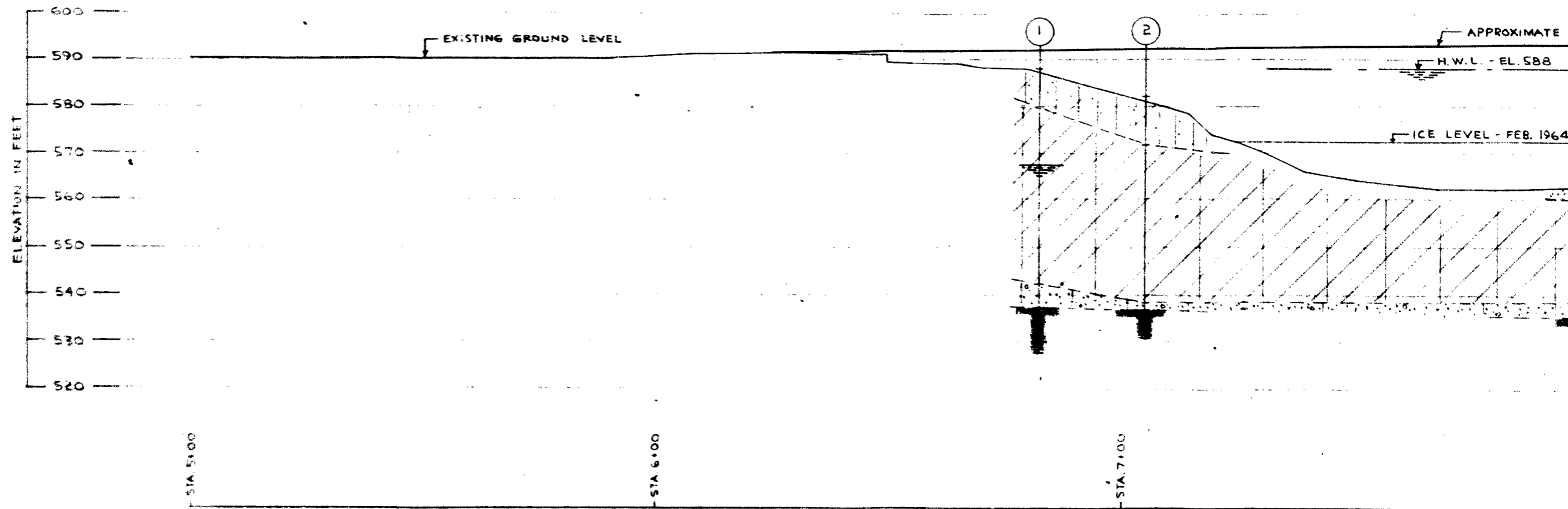
LEGEND

BH3  BOREHOLE IN PLAN

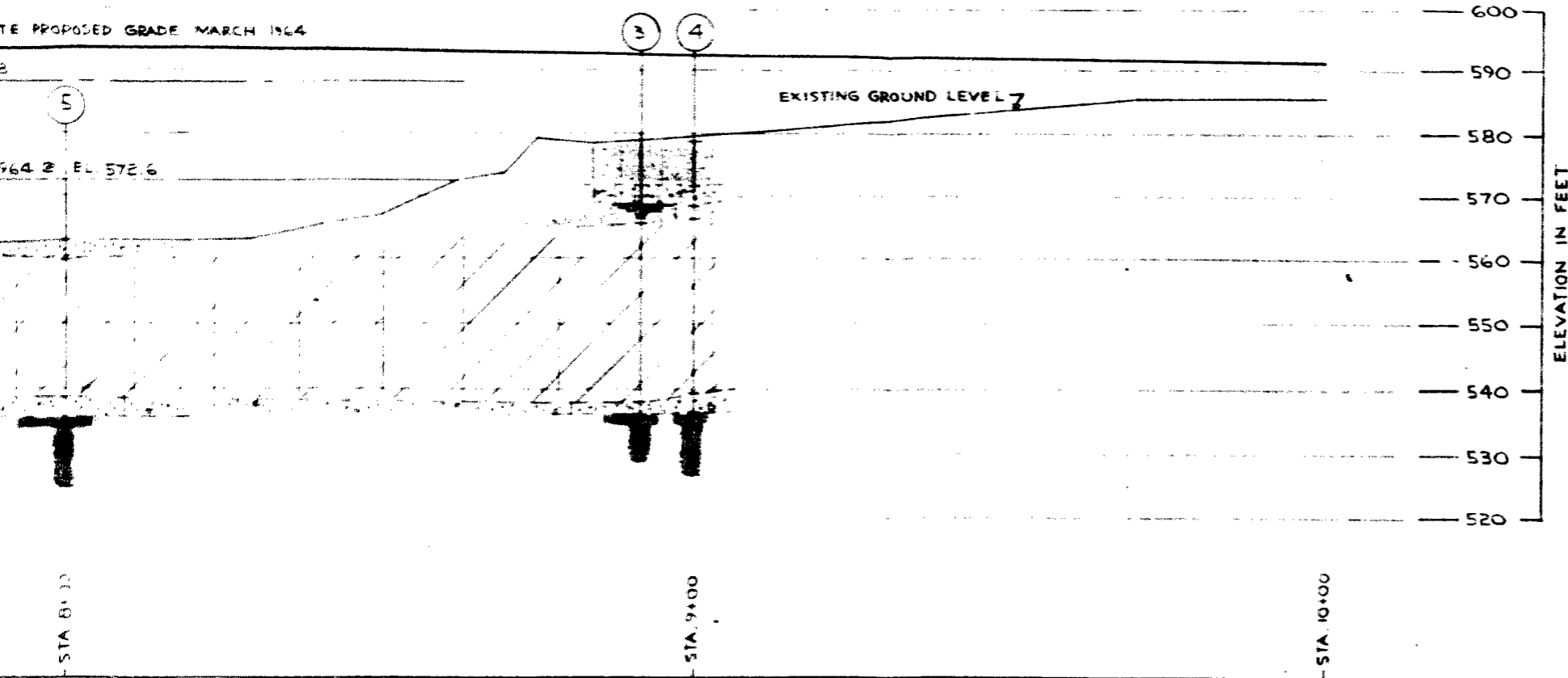
--- PROPOSED REALIGNMENT, COUNTY ROAD NO 28

GOLDER & ASSOCIATES








Made - J.A.
 Chkd. - J.A.
 Appd. - J.A.



SCHEMATIC SECTION ALONG CENTRELINE OF PROPOSED
SCALE: 1" TO 20'-0"



STRATIGRAPHY

-  COMPACT TO LOOSE MOTTLED BROWN STRATIFIED SANDY AND CLAYEY SILTS WITH SOME ROOTS.
-  SOFT COMPRESSIBLE ORGANIC SANDY SILT WITH SOME WHITE SHELLS.
-  VERY LOOSE TO LOOSE GREY SILTY SAND WITH SOME SHELLS AND PIECES OF BLACKENED WOOD.
-  FIRM TO STIFF MOTTLED BROWN CLAYEY SILT WITH SOME SANDY POCKETS.
-  GENERALLY FIRM TO SOFT GREY-BROWN SENSITIVE SILTY CLAY WITH A FEW LAYERS OF GREY CLAYEY SILT.
-  DENSE TO VERY DENSE DARK GREY SILTY SAND AND GRAVEL (TILL)
-  DARK GREY SHALE BEDROCK, UPPER PORTION, (GENERALLY 1 TO 3 FT.) IS WEATHERED. ROCK CONTAINS SOME FRACTURES.

FLOODPLAIN DEPOSITS

LEGEND



BOREHOLE IN ELEVATION



WATER LEVEL IN BOREHOLE - FEB. 1964

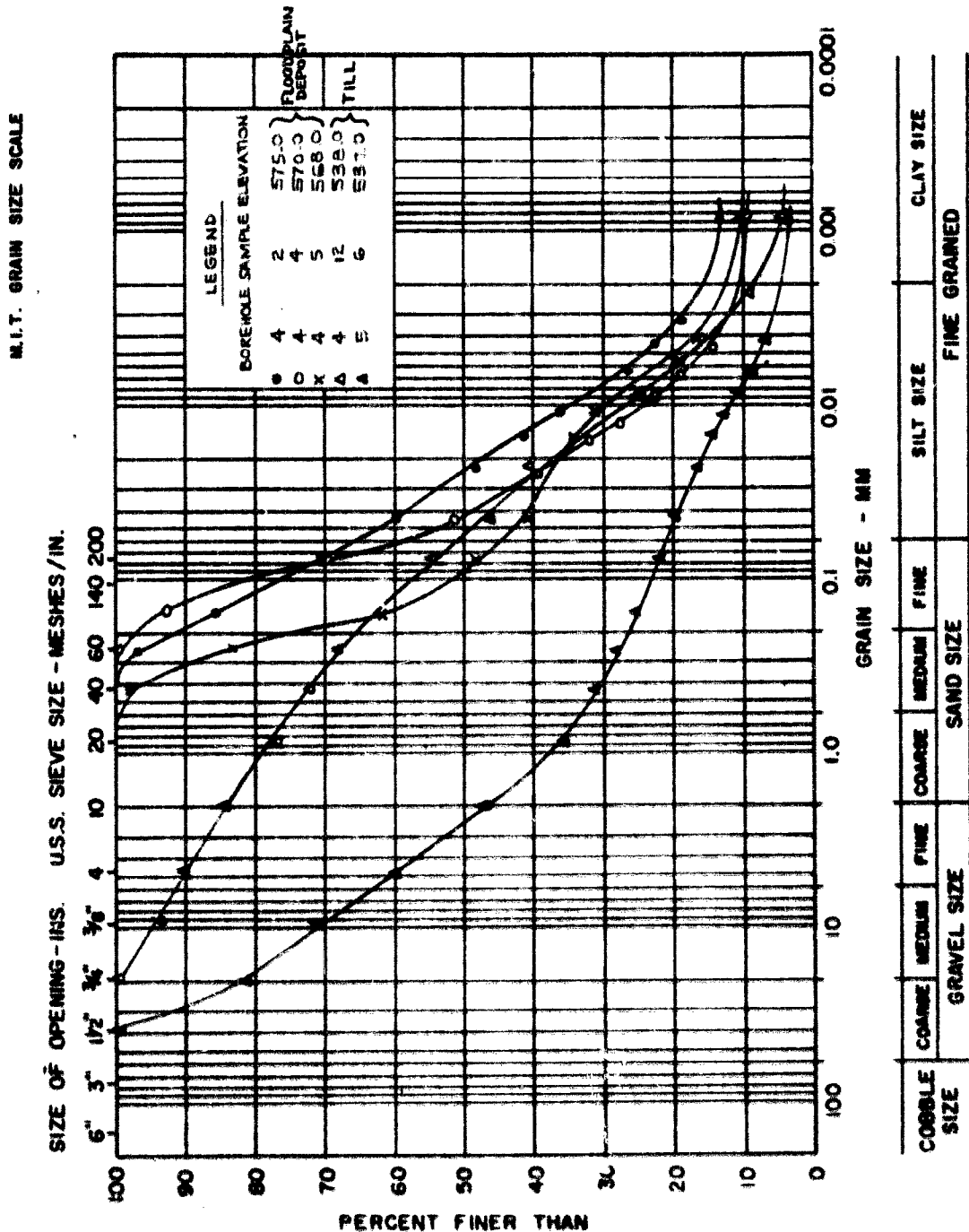
D REALIGNMENT - COUNTY ROAD N° 28

GOLDER & ASSOCIATES

Made J.A.
 Chkd. J.A.
 App. J.A.

GRAIN SIZE DISTRIBUTION

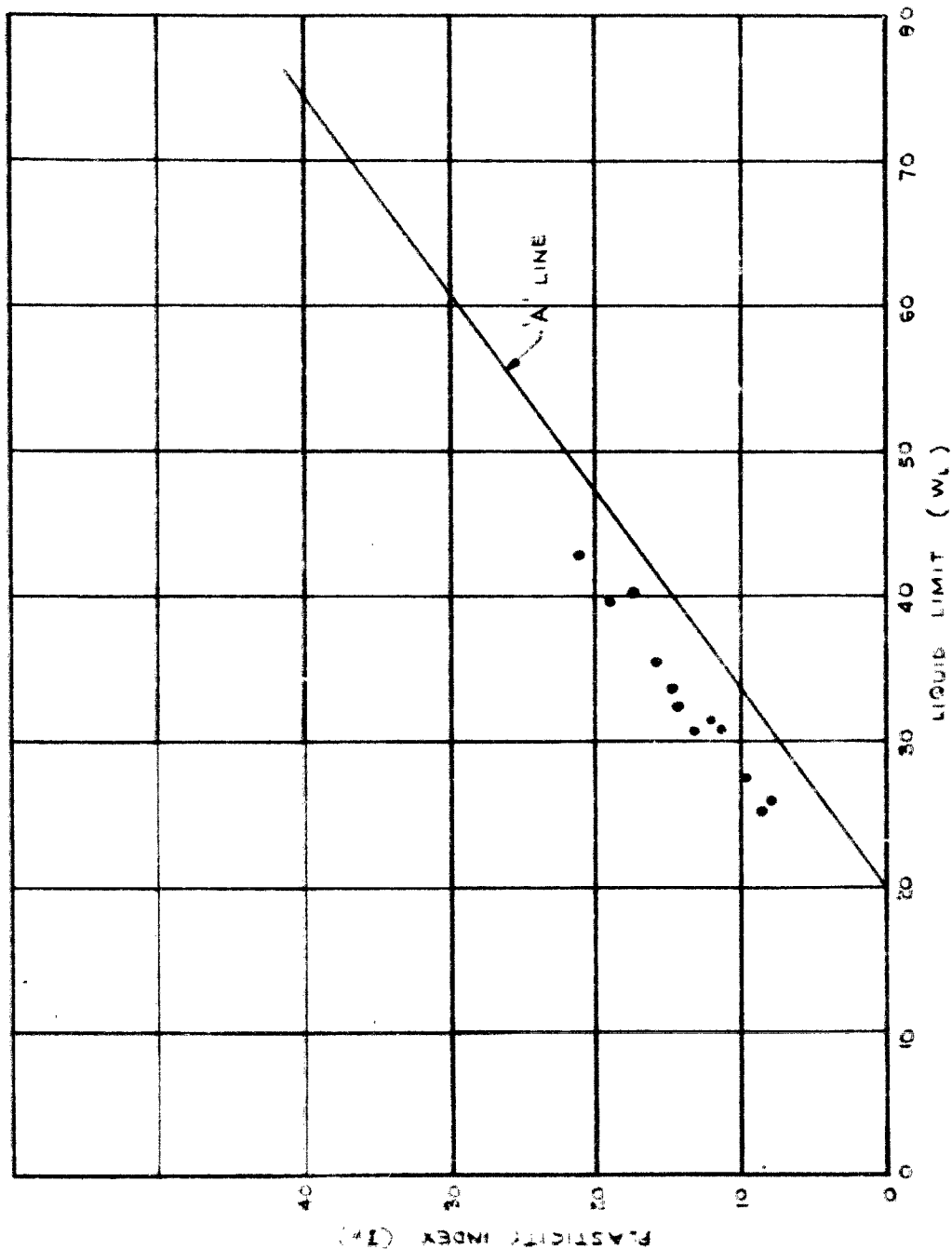
FIGURE 3



PLASTICITY CHART

SOFT SILTY CLAY

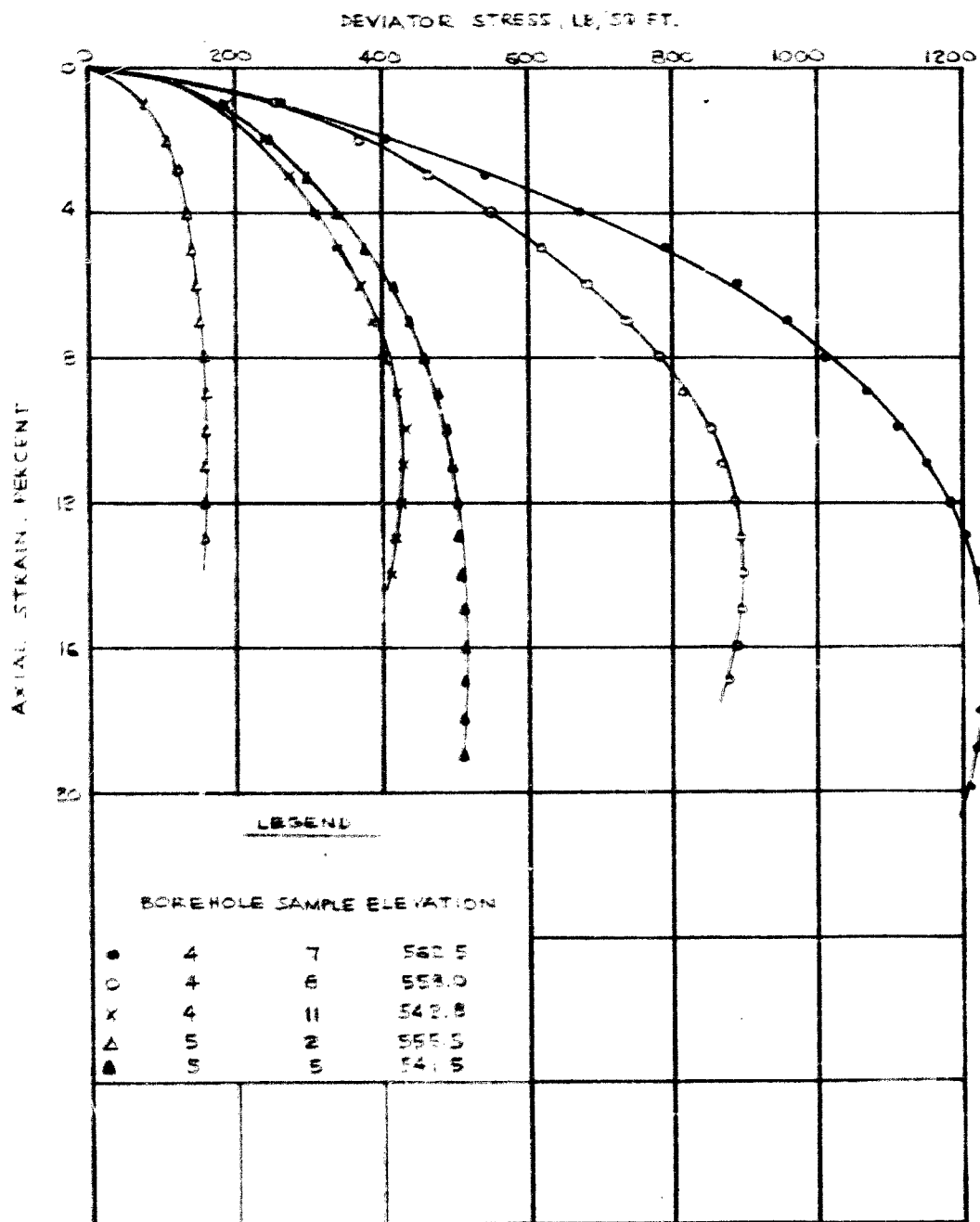
FIGURE 4

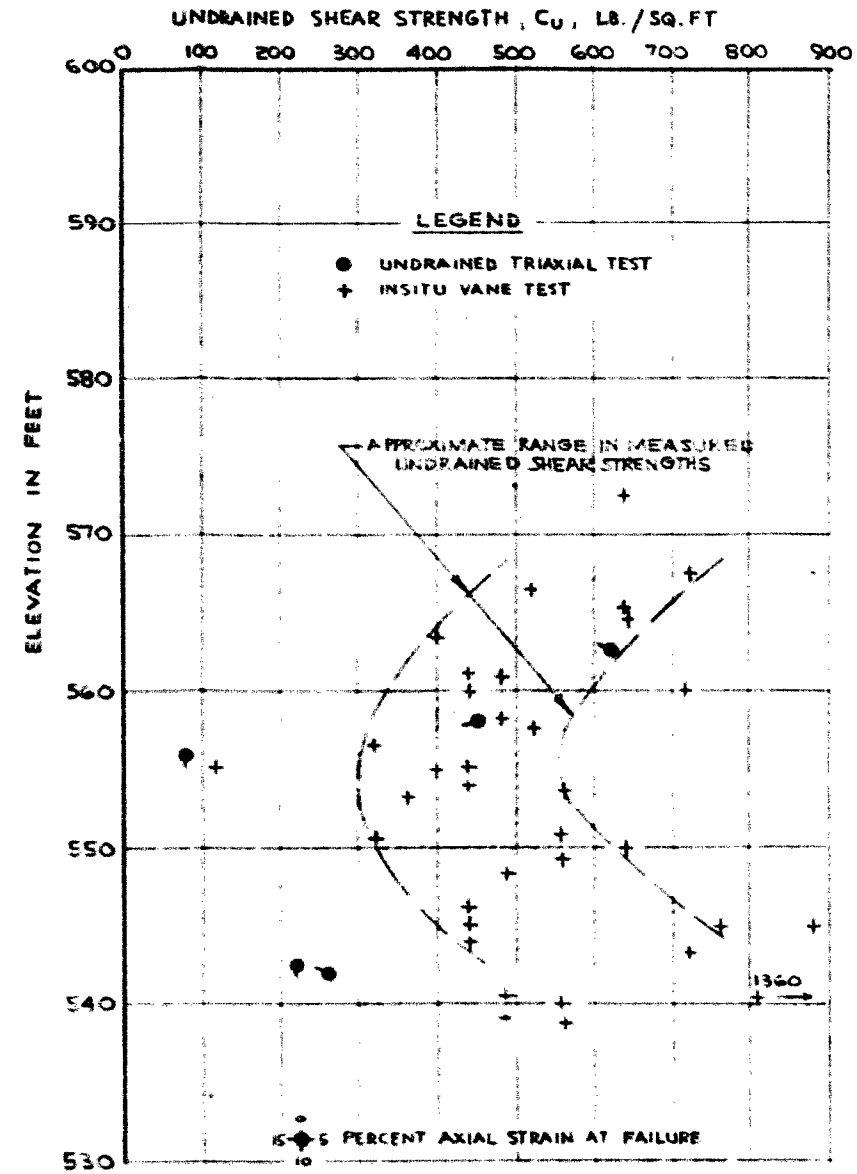
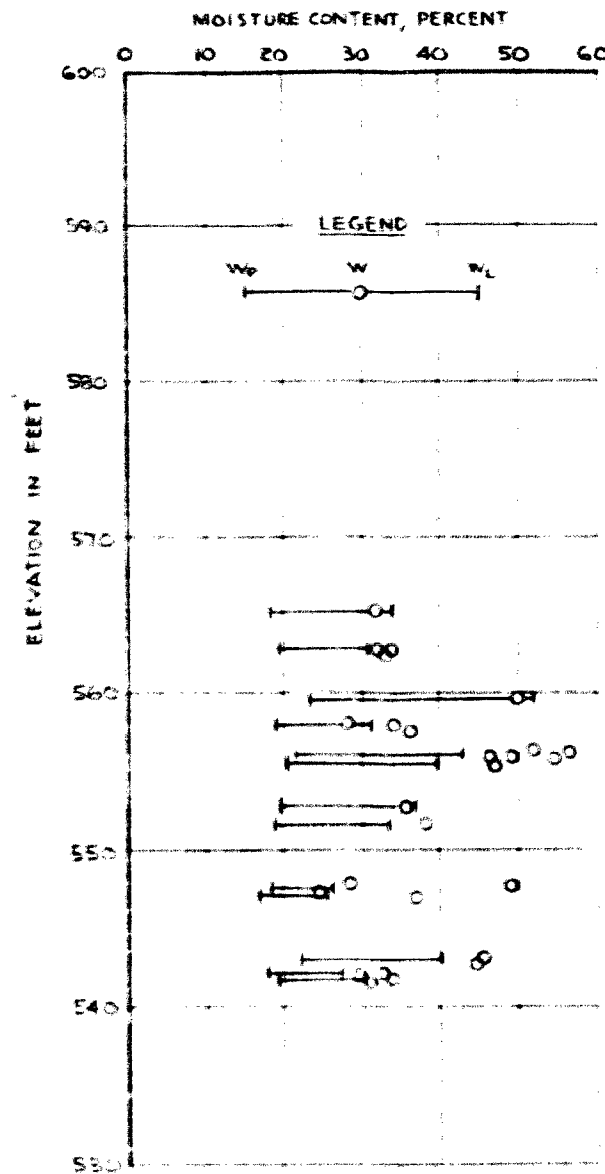


GOLDER & ASSOCIATES

UNDRAINED TRIAXIAL TESTS STRESS-STRAIN CURVES

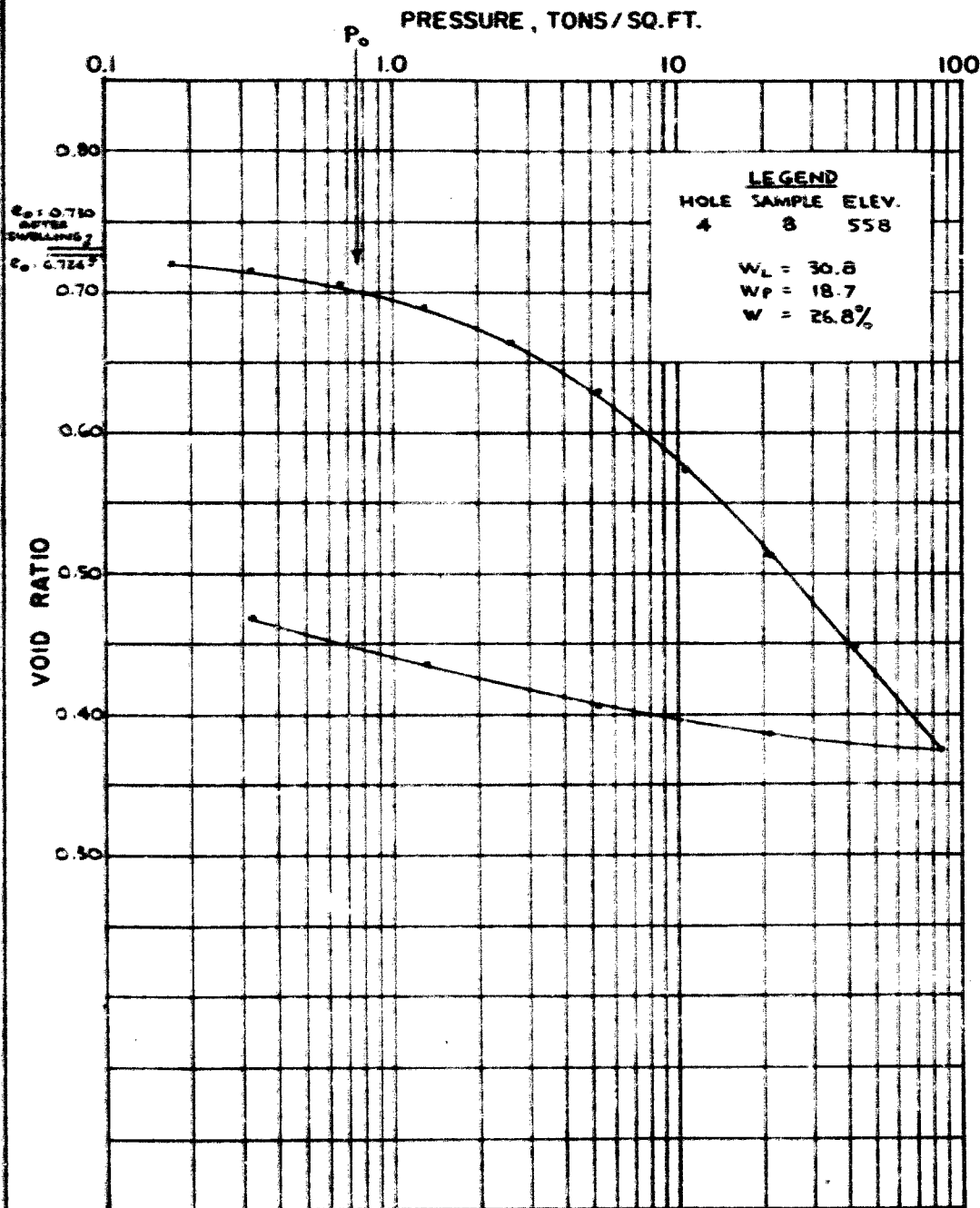
FIGURE 5





VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

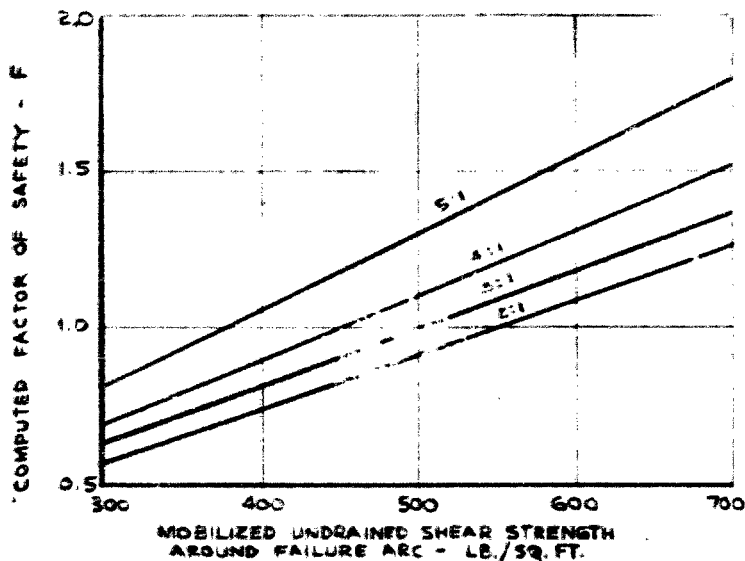
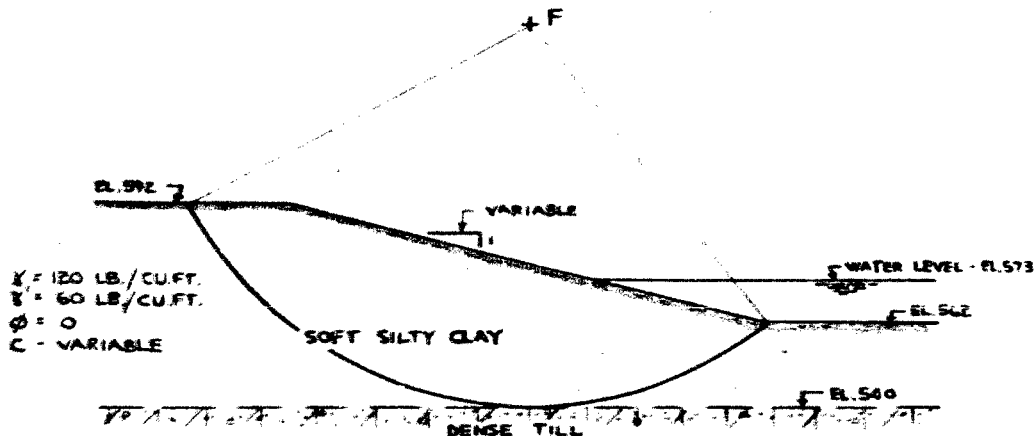
FIGURE 7



GOLDER & ASSOCIATES

$\phi = 0$ CIRCULAR ARC STABILITY ANALYSES SUMMARY OF RESULTS

FIGURE 8

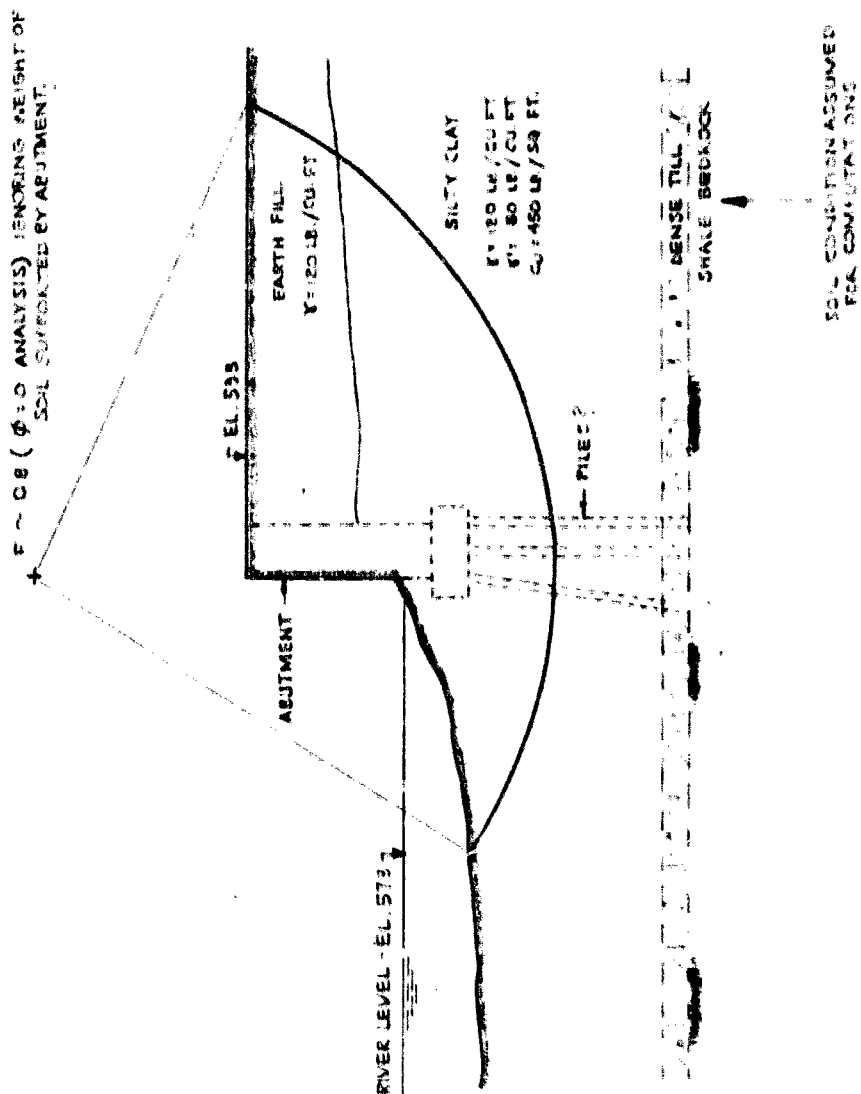


GOLDER & ASSOCIATES

Made... J.A.
Chkd... *sls*
Appd... *sls*

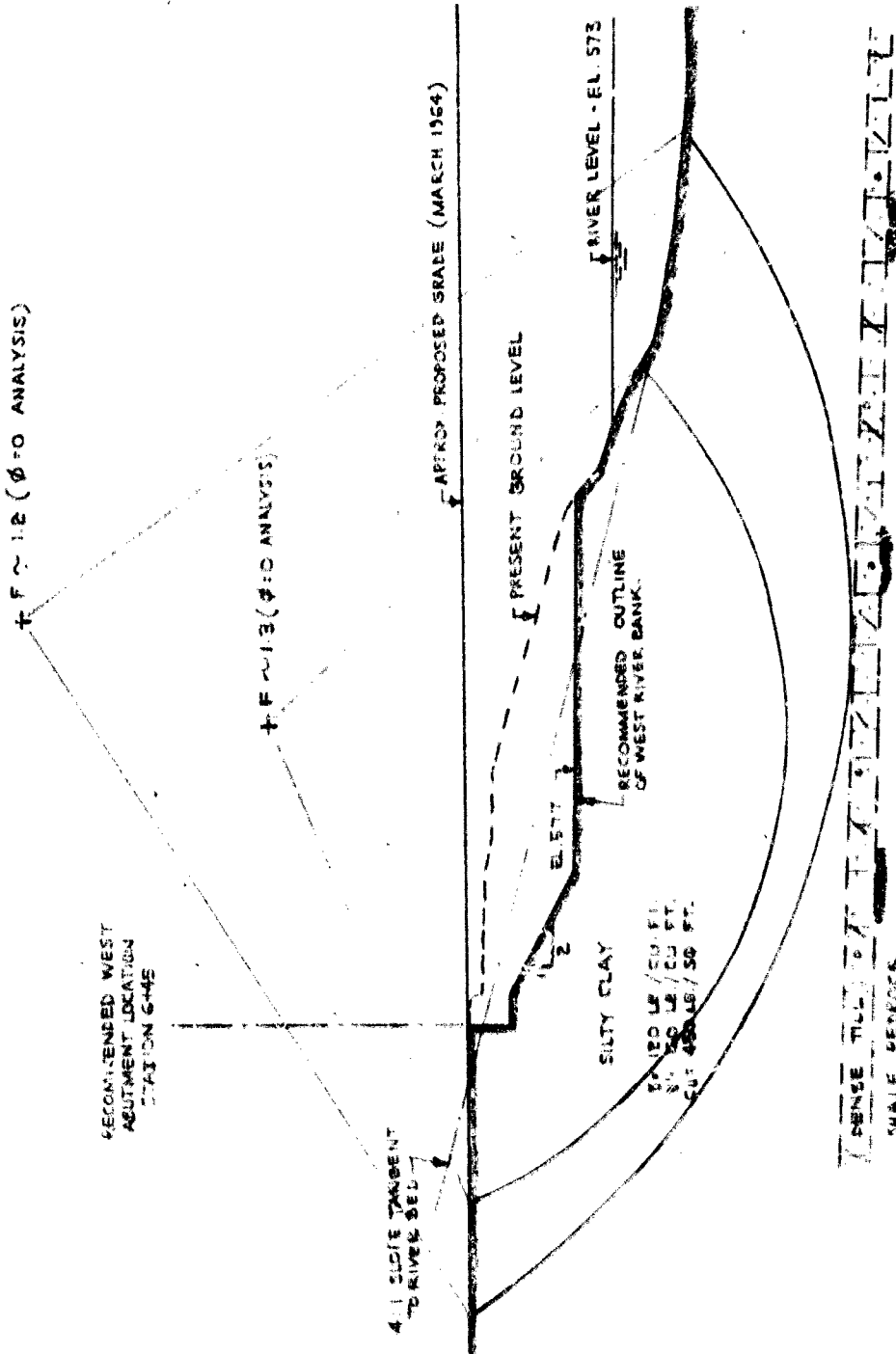
EAST ABUTMENT OF EXISTING BRIDGE STABILITY ANALYSIS

FIGURE 9



GOLDER & ASSOCIATES

Made 11-2-00
Chkd 11-2-00
Appd 11-2-00

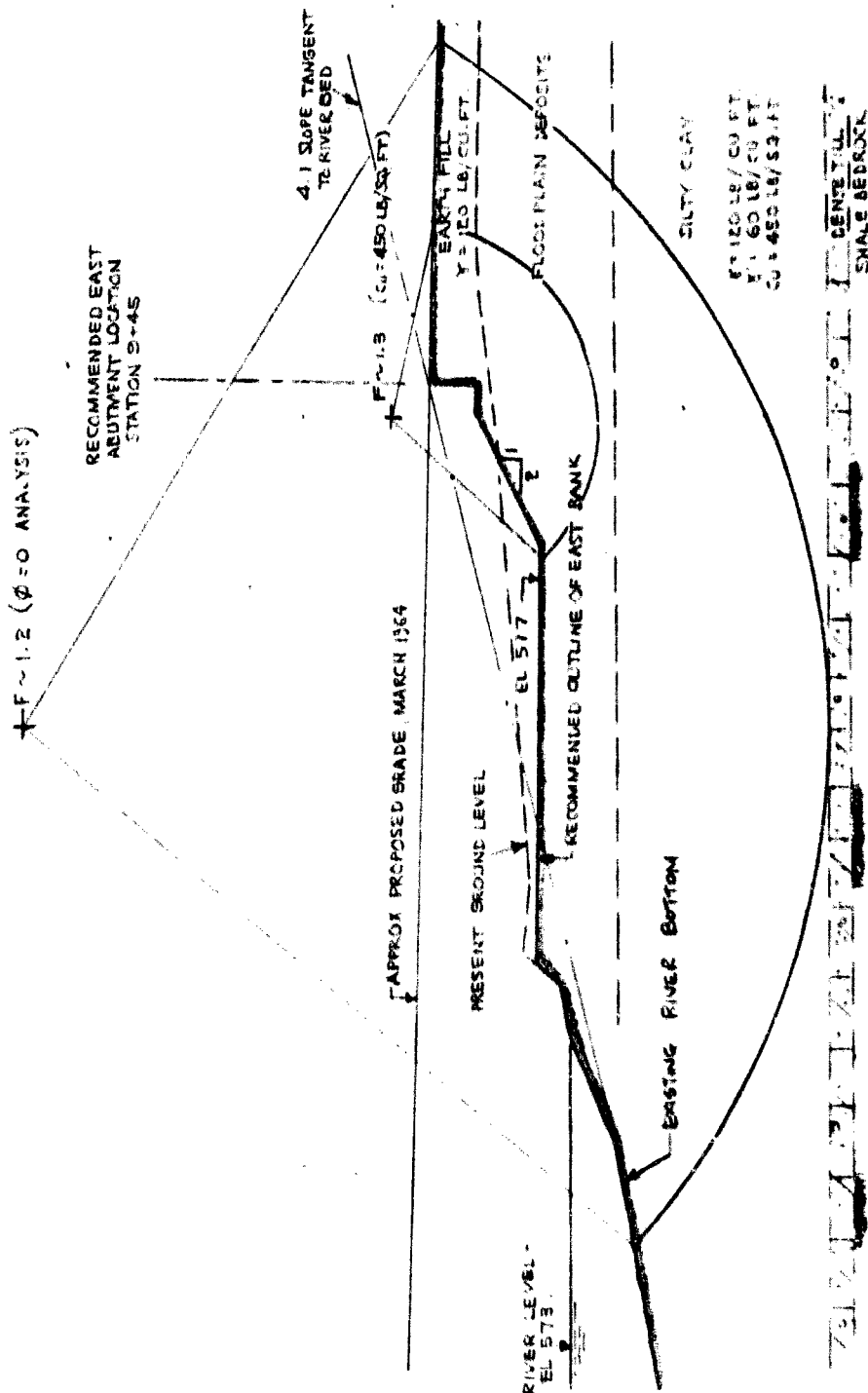


GOLDER & ASSOCIATES

Made *[Signature]*
 Chkd *[Signature]*
 Appd *[Signature]*

RECOMMENDED SECTION - EAST BANK OF RIVER

FIGURE 11



GOLDER & ASSOCIATES

Made: *[Signature]*
 Check: *[Signature]*
 Appd: *[Signature]*

FIGURE 12

**GOLDER & ASSOCIATES**

Modo 240
Chkd 1-10
Appd 1-10