

Mr. A. M. Toye,
Bridge Engineer.

Materials & Research Section.

October 4, 1960.

RE: PILE TESTING -

by D.H.O.

Attention: Mr. S. McCombie.

Re: Proposed Thames River Bridge,
Wardsville, Ontario, District #2.

Four piles driven for the support of the North pier at the above proposed structure, were tested, and the following was observed:-

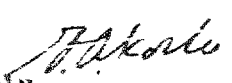
At the depth 35 - 40 ft. below the cut-off (149 ft.) elevation of the pile, the blows with the D-12 hammer, averaged 75 blows per foot. The observed rebound value 'C' was 0.5. From these values the calculated ultimate load per pile from the Hile formula chart, is about 185 tons.

It is believed that this value has enough safety factor for the indicated 40 tons per pile design load. As such, it will be sufficient to drive the piles down to elevation 109 ft. and if the blows for the last five feet average at 70 - 75 (with a rebound value 'C' - 0.5), then further driving down of the pile will be stopped.

YV/4deF

cc: Messrs. L. A. Tregaskes
L. G. Ramsay
A. Gater
W. L. Fraser
J. Roy
Foundations Office.
Gen. Files.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.
Per:


(V. Korlu,
PROJECT FOUNDATIONS ENGR.)

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

April 8, 1960.

Attention: Mr. A. McGeehan.

Re Report No. S7035A by Gecon, Limited
to R.C. Dunn and Associates, Limited
on Soil Investigation - Proposed
Thames River Bridge, Wardsville, Ont.
District No. 2.

Attached hereto, for your information and files,
is the foundation report prepared by Gecon, Limited, to
R. C. Dunn and Associates, Limited, Consulting Engineers
for the proposed Thames River structure, Wardsville, Ontario.

for - Mr. de Looze

LGS/MdeF
Attach.

L. C. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. C. Ramsay
A. Gater
W. L. Fraser
J. Roy
A. Watt
Foundations Office
Gen. Files. ✓

S7035A

28-2

REPORT

TO

R. C. DUNN AND ASSOCIATES LTD.

ON

SOIL INVESTIGATION

PROPOSED THAMES RIVER BRIDGE

WARDSVILLE

ONTARIO

Distribution:

- 5 copies - R. C. Dunn and Associates Ltd.,
London, Ontario.
- 10 copies - Department of Highways, Ontario,
Downsview, Ontario.
- 2 copies - Geocon Ltd,
Toronto, Ontario.

March 31st, 1960

GEOCON

GEOCON LTD

HEAD OFFICE
180 VALLÉE ST., MONTREAL 19, QUEBEC
TELEPHONE UN. 6-7632

DISTRICT OFFICES

14 HAAS ROAD
REXDALE, TORONTO, ONT.
TEL. CH. 4-8641

1425 WEST PENDER ST.
VANCOUVER 5, B.C.
TEL. MU. 1-8926

Rexdale, Ontario,
March 31st, 1960.

R. C. Dunn and Associates Ltd.,
Consulting Engineers,
410 Third Street,
London Ontario.

Attention: Mr. N. M. Warner, P. Eng.

Re: Soil Investigation,
Proposed Thames River Bridge,
Wardsville, Ontario.

Dear Sirs:

This letter reports the results of the above investigation carried out in accordance with our verbal retention by the Department of Highways, Ontario on January 15th, 1960 and your written confirmation of February 2nd, 1960. The object of the investigation was to determine and interpret the subsoil conditions at the above site, as they affect the design of foundations for the proposed bridge.

PROCEDURE

A total of three boreholes, each with an adjacent dynamic penetration test, was put down at the proposed site between January 25th, 1960 and January 30th, 1960, using mobile power auger equipment. Following this two further boreholes were put down between February 10th and February 16th, 1960 using a barge-mounted machine drillrig. The locations of all the boreholes, together with the inferred soil stratigraphy, are shown on Drawing S7035A-1 at the rear of this report. The detailed log of each boring is shown on the Office Reports on Soil Exploration in Appendix I.

PROCEDURE (continued)

The testing of the soil samples was carried out in the Toronto Soil Mechanics Laboratory of Geocon Ltd and the results are plotted on the Office Reports and on the figures in Appendix II. The soil samples remaining after testing will be stored until October 1st, 1960, at which time you will be contacted regarding their disposal.

All elevations are referred to County of Elgin datum and were obtained from a bench mark located on top of the southeast corner of the curb on the west side of the existing bridge.

SITE

The proposed Thames River Bridge is to be located approximately 55 feet east of the existing bridge structure across the Thames River on the road between Wardsville and Rodney. At this location the river is the dividing line between the County of Elgin to the south and the County of Middlesex to the north.

The area north and south of the river consists of farm lands generally sloping gently towards the river.

SOIL CONDITIONS

The principal soil strata encountered by the borings are as follows:

Very Loose to Loose Brown and Grey Sandy Silt

A stratum of brown and grey sandy silt was encountered at ground surface in boreholes 1, 2, and 3. It ranged in depth from

SOIL CONDITIONS (continued)

Very Loose to Loose Brown and Grey Sandy Silt (continued)

8 feet in borehole 2 to 16 feet in borehole 1. The relative percentages of sand and silt varied throughout the stratum. The upper portion generally contains some organic matter.

One grain size distribution curve for a typical sample of the stratum is shown on Figure 1 of Appendix II. It indicates that approximately 35 percent silt sizes and 65 percent sand sizes are present.

Standard penetration tests carried out in the stratum gave "N" values varying from 2 to 9 blows per foot with an average of 5 blows per foot indicating that the relative density of the stratum is very loose to loose.

Very Loose Brown Sand

A stratum of brown sand, about 4 feet in thickness was encountered at river bottom at boreholes 4 and 4A. The upper portion of the stratum is black in colour. It is probable that this stratum extends across the river bottom.

One "N" value obtained from a standard penetration test gave a value of 1 blow per foot indicating that the relative density of the stratum is very loose.

SOIL CONDITIONS (continued)

Stiff to Hard Grey Silty Clay

Underlying the brown and grey sandy silt on the river banks and the brown sand in the river channel, a stratum of grey silty clay was encountered. The silty clay was penetrated to a maximum depth of about 70 feet in borehole 2. The stratum contains scattered pebble gravel throughout its depth. A thin layer or lense of sand was encountered at about elevation 124 in borehole 4A.

Atterberg limit determinations gave liquid limits varying between 25 and 37 with an average value of about 33 and plasticity indices generally varying between 10 and 20 with an average value of about 15.

The wet unit weights ranged from 139 to 127 pounds per cubic foot at corresponding natural moisture contents which varied generally from 14 to 29 percent.

The shear strength, as determined by undrained triaxial compression tests, ranged from about 900 to 5,000 pounds per square foot. A plot of shear strength versus elevation for this stratum is shown on Figure 3 of Appendix II. This indicates a minimum shear strength of about 1,500 pounds per square foot at about elevation 140.0. Based on these results, it is estimated that the consistency of the stratum is generally stiff to hard.

SOIL CONDITIONS (continued)

Stiff to Hard Grey Silty Clay (continued)

Two consolidation tests were carried out on samples from the silty clay and the resulting log pressure-void ratio curves are shown on Figure 2 in Appendix II. A plot of the computed coefficient of consolidation, C_v , against log pressure for one of the tests is also given in the figure. From the general shape of the consolidation curves and from the pattern of shear strength versus depth and $(c/p)_n$ relationship, it is estimated that the stratum has been subjected to a minimum past pressure of at least 4,000 pounds per square foot in excess of the existing overburden pressure. In settlement computations a value of 0.05 may be used for the rebound compression index, C_R . This value is the average of the rebound slopes and the slopes of the initial section of the laboratory curves for the two consolidation tests carried out.

Dark Grey Shale

A dark grey shale was encountered underneath the grey silty clay in borehole 4A and was cored for about 6 feet in AXT size. The top 9 inches was found to be weathered and below this it was generally sound.

WATER CONDITIONS

The water level in boreholes 1 and 2 was at about elevation 195.0 on February 16th, 1960. Borehole 3 caved in at elevation 175.0 on January 30th, 1960 indicating the probable

WATER CONDITIONS (continued)

groundwater level. The river level during the investigation varied between about elevations 154.0 and 160.0.

DISCUSSION

General:

It is understood that it is proposed to replace the existing bridge over the Thames River on the County Road between Wardsville and Rodney by a three-span, two-lane continuous structure. The proposed new structure is to be adjacent to and on the east side of the existing bridge. The proposed pier and abutment locations are shown on Drawing S7035A-1 attached to this report. The approach embankments at the north and south ends of the bridge will reach maximum heights of about 30 and 20 feet respectively and will have end slopes of 2 horizontal to 1 vertical on the river side of the spill-through type abutments.

Approach Embankments:

Computations show that the factor of safety against a circular sliding type failure of the approach embankments having end slopes of 2 horizontal to 1 vertical is adequate and equal to at least 1.3, provided that the embankments are constructed of granular material well compacted in place. The end slope of the embankments facing the river should be protected by a rip-rap cover to at least the river high water level in order to prevent erosion and scour.

DISCUSSION (continued)

Approach Embankments: (continued)

Consolidation of the silty clay will take place under the proposed approach embankments. Assuming a top width of 40 feet and side slopes of 2 horizontal to 1 vertical for the approach embankments of the height shown on Drawing S7035A-1, it is estimated that a total consolidation settlement of the silty clay of the order of 5 and 6 inches will take place at the south and north abutment locations, respectively.

The time required for 90 percent of the total consolidation settlement of the silty clay to take place is estimated to be of the order of 20 to 25 years.

From preliminary engineering considerations of various types of foundation feasible for the proposed structure, two types were chosen for further study: spread footings and displacement or bored piles.

Spread Footing Foundations:

To eliminate the possibility of scouring action by the river, spread footings at the central pier locations would have to be founded not higher than elevation 130.0. At the north and south abutments, they should be founded in the silty clay stratum at elevations 145.0 and 160.0 respectively. For spread footings at these locations, founded at or below the recommended elevations indicated above, a net bearing pressure of 3,000 pounds per square

DISCUSSION (continued)

Spread Footing Foundations: (continued)

foot may be used for design. This figure is based on a minimum average shear strength of 1,500 pounds per square foot as determined by undrained triaxial compression tests.

Assuming pier and abutment loadings of 1,000 and 500 tons respectively, it is computed that the total consolidation settlement of spread footings, about 50 feet in width, under the allowable net bearing pressure will be of the order of 1.5 inches for the abutments and 2 inches for the central piers.

However, in the case of the abutments the situation is complicated by the approach embankments which will induce a consolidation settlement in the silty clay. The effect this settlement will have on the spread footings is difficult to compute, but it is estimated that the spread footings will suffer settlement of the same order as the adjacent embankments.

Piled Foundations:

For computation purposes a displacement or bored pile approximately 12 inches in mean diameter was considered. The ultimate capacity of a friction pile was computed on the following basis:

- (a) adhesion of the clay to the pile perimeter
equal to 1,000 pounds per square foot throughout the full depth of penetration in the silty clay,

DISCUSSION (continued)

Piled Foundations: (continued)

- (b) bearing value at the pile tip equal to 9 times the average shear strength of the silty clay at this depth.

It is computed that the ultimate capacity of a friction pile of the dimensions given above for a penetration of 50 feet within the clay is about 85 tons. Allowing a factor of safety of approximately 3 on the computed ultimate capacity, it is recommended that the load on individual piles under total dead loading be not greater than 30 tons; the load on individual piles under transient loading should not exceed 40 tons.

At the south abutment, utilizing a pile group having an area in plan approximately 5 to 6 feet by 50 feet and having a toe elevation of 115.0, the total consolidation settlement is estimated to be about 1.5 inches. At the north abutment with a toe elevation of 95.0 the consolidation settlement would be of the same order. However, allowing for the negative skin friction induced by the consolidation of the silty clay stratum under the approach embankments the most probable maximum consolidation settlement of the pile groups at the abutments is estimated to be about 4 inches.

At the two central pier locations, assuming a pile group of plan area 12 feet by 50 feet and a toe elevation of 95.0 the consolidation settlement is estimated to be about 2 inches.

DISCUSSION (continued)

Conclusions:

If spread footings are used, it would be necessary to sheet the excavations and this sheeting would have to be carried down to a minimum of 5 feet below the footing elevation. Due to the depth of excavation required and the possible occurrence of water bearing sand seams or lenses in the silty clay, it is probable that spread footings would prove uneconomical in this case. It is thus suggested that piled foundations be considered for the structure. It is considered that at this site bored concrete filled cylindrical piles would provide a suitable and economical piled foundation.

As can be seen from the consolidation settlement figures given above for spread footing foundations, a differential settlement between the piers and the abutments of up to 4 inches could occur with time. For piled foundations, a differential settlement of up to 2 inches could occur between the piers and abutments. It is for this reason that it is recommended that friction piles be used throughout in order to reduce the resulting differential settlement to a minimum. It is suggested that the effect of a differential settlement of up to 2 inches between a pier and adjacent abutment be studied for the continuous structure proposed. If a differential settlement of this order is not tolerable, it is further suggested that either simply supported spans or a structure cantilevered on the piers and fixed at one end be considered. With this type of structure, which may tolerate large differential settlements, it may prove feasible to adopt spread footing foundations, as discussed above.

R. C. Dunn and Associates Ltd.,
Consulting Engineers,
March 31st, 1960,
Page 11.

DISCUSSION (continued)

Conclusions: (continued)

We would be pleased to discuss the behaviour of specific foundation types with you once the final design of the structure is determined.

We believe that this letter report, which was written by Mr. N. R. McCammon and checked by Mr. J. L. Seychuk, contains all the necessary information for the foundation design of the proposed bridge. Should you have any further questions, please do not hesitate to contact us.

Yours very truly,

GEOCON LTD

N. R. McCammon

N. R. McCammon, P. Eng.,
Soils Engineer.



NRM/dw
S7035A

GEOCON

APPENDIX I

OFFICE REPORTS ON SOIL EXPLORATION

EXPLANATION OF THE FORM "OFFICE REPORT ON SOIL EXPLORATION"

The object of this form is to enable a comprehensive study of the soil to be made by combining on one sheet all of the information obtained from the boring. An explanation of the various columns of the report follows.

ELEVATION AND DEPTH

This column gives the elevation and depth of boundaries between the various soil strata. The elevation is referred to the datum shown in the general heading.

WATER CONDITIONS

In this column the water level in the casing at the time of boring or the water table in the ground, determined by a series of observations in a piezometer or standpipe, is indicated to scale by a horizontal line with the symbol W.L. or W.T. above the line. A notation of any complicated groundwater conditions will be made in this column.

DESCRIPTION

A description of the soil, using standard terminology, is contained in this column. The consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

<u>Consistency</u>	<u>U-Strength Tons/sq. ft.</u>	<u>Relative Density</u>	<u>Standard Penetration Resistance, Blows/ft.</u>
Very soft	0.03 to 0.25	Very loose	0 to 4
Soft	0.25 to 0.5	Loose	4 to 10
Firm	0.5 to 1.0	Compact	10 to 30
Stiff	1.0 to 2.0	Dense	30 to 50
Very stiff	2.0 to 4.0	Very dense	over 50
Hard	over 4.0		

STRATIGRAPHIC PLOT

The stratigraphic plot follows the standard symbols of the National Research Council, Canada.

ELEVATION SCALE

The information in all columns is plotted to a true elevation scale which is shown in this column.

GRAPHS

The main body of the report forms a graph which is used to plot to correct elevation the important soil properties which are obtained through field and laboratory tests. The scales and symbols for the plotting are shown at the head of the column.

OTHER TESTS

In this column are shown, by symbol, the other field or laboratory tests which have been performed on the soil and for which the results have not been plotted on the above graph.

SAMPLES

The first three columns describe the condition, type and number of each sample obtained from the boring. The location and extent of each sample is plotted to scale.

In the last column is shown the penetration resistance in blows of 4200 inch-pounds required to drive one foot of the sampler into the ground. When a 2 inch Drive Sampler is used the result obtained is termed the "Standard Penetration Resistance".

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57035 A BORING # 1 DATUM COUNTY OF ELGIN CASING
BORING DATE JAN. 25, 1960 REPORT DATE FEB. 23, 1960 COMPILED BY M.W. CHECKED BY J.M.E.
SAMPLER HAMMER WT. 140 LBS. DROP 28 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. - LBS. ENERGY)

SAMPLE CONDITION


 DISTURBED
 FAIR
 GOOD
 LOST

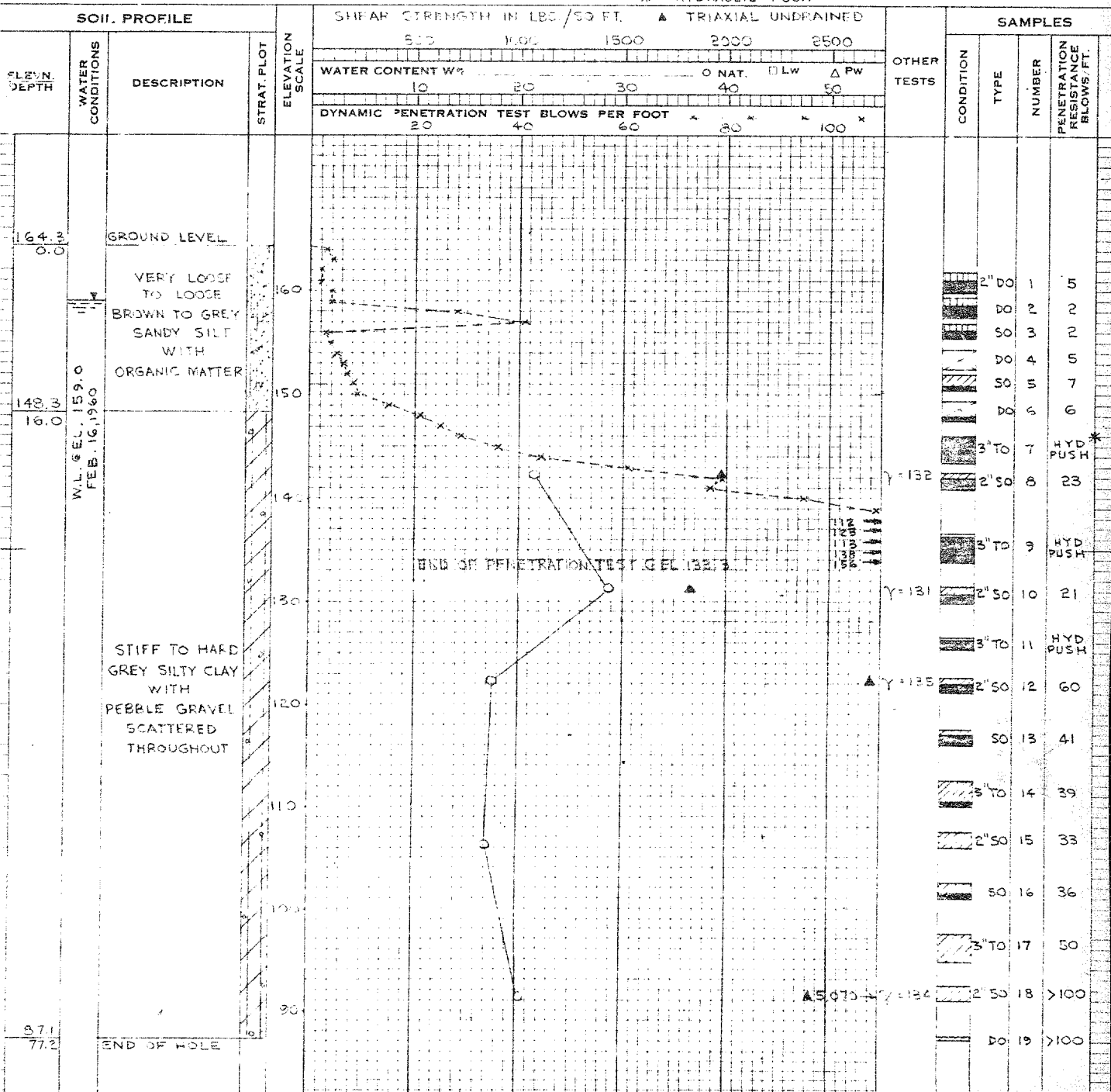
SAMPLE TYPES

A.S. - AUGER SAMPLE	F.S. - FOIL SAMPLE
S.T. - SLOTTED TUBE	S.O. - SLEEVE-OPEN
W.S. - WASHED SAMPLE	S.F. - SLEEVE-FOOT VALVE
D.O. - DRIVE-OPEN	T.O. - THIN WALLED OPEN
D.F. - DRIVE-FOOT VALVE	R.C. - ROCK CORE
C.S. - CHUNK SAMPLE	

ABBREVIATIONS

V - IN-SITU VANE TEST γ - WET UNIT WEIGHT
M - MECHANICAL ANALYSIS K - PERMEABILITY
U - UNCONFINED COMPRESSION C - CONSOLIDATION
Qc - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK WL - WATER LEVEL IN CASING
S - TRIAXIAL SLOW WT - WATER TABLE IN SOIL
* - HYDRAULIC PUSH

SOIL PROFILE



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS

	DISTURBED
	FAIR
	GOOD
	LOST

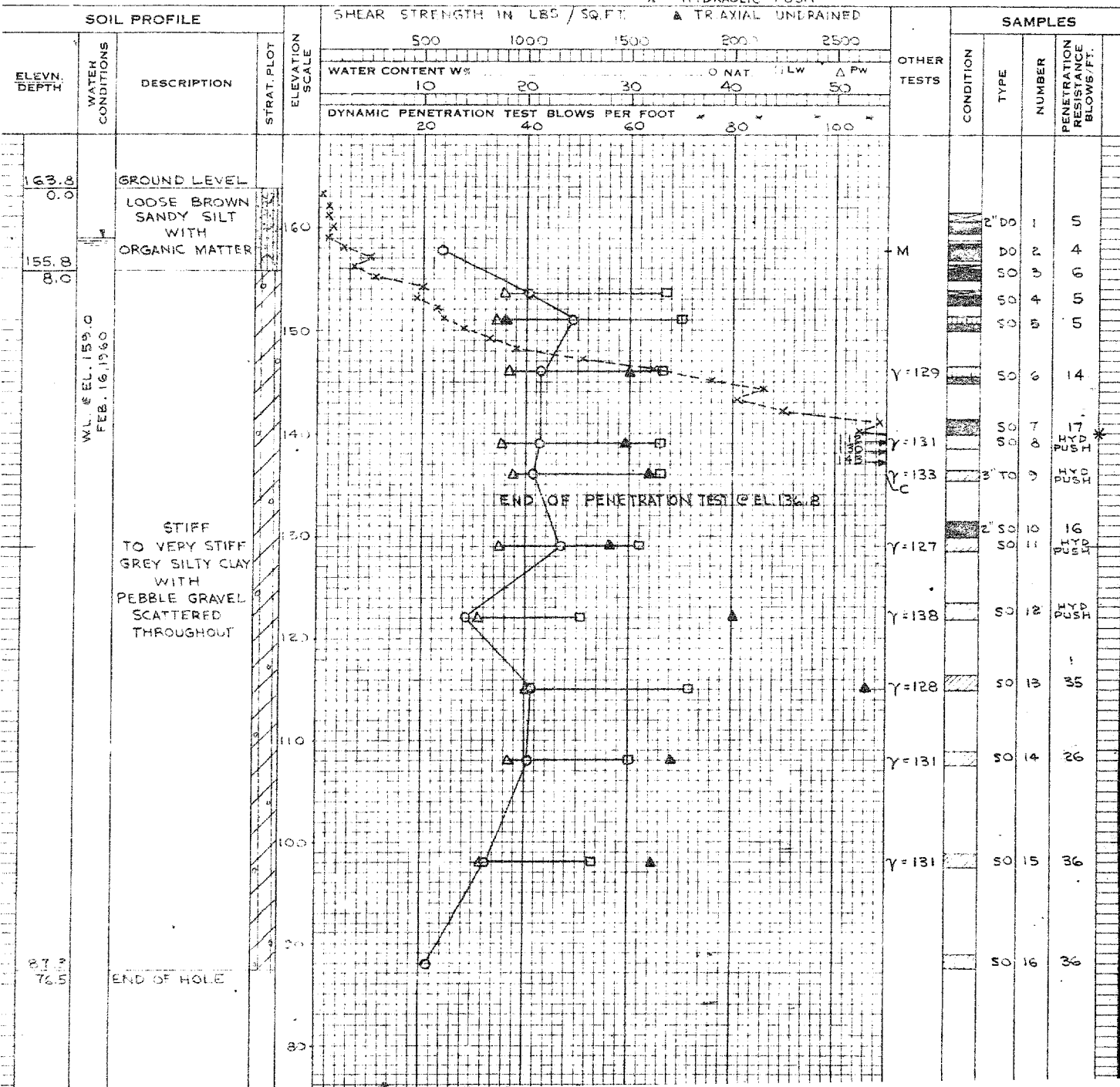
A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

- V - IN-SITU VANE TEST
- M - MECHANICAL ANALYSIS
- U - UNCONFINED COMPRESSION
- QC - TRIAXIAL CONSOLIDATED QUICK
- Q - TRIAXIAL QUICK
- S - TRIAXIAL SLOW
- % - HYDRAULIC PUSH

γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

CONTRACT 57235 A BORING # 3 DATUM COUNTY OF ELGIN CASING -
 BORING DATE JAN. 29, 1960 REPORT DATE FEB. 24, 1960 COMPILED BY M.V. CHECKED BY A.M.B.
 SAMPLER HAMMER WT. 140 LBS. DROP 23 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL SLOW
 S - TRIAXIAL SLOW
 * - HYDRAULIC PUSH
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

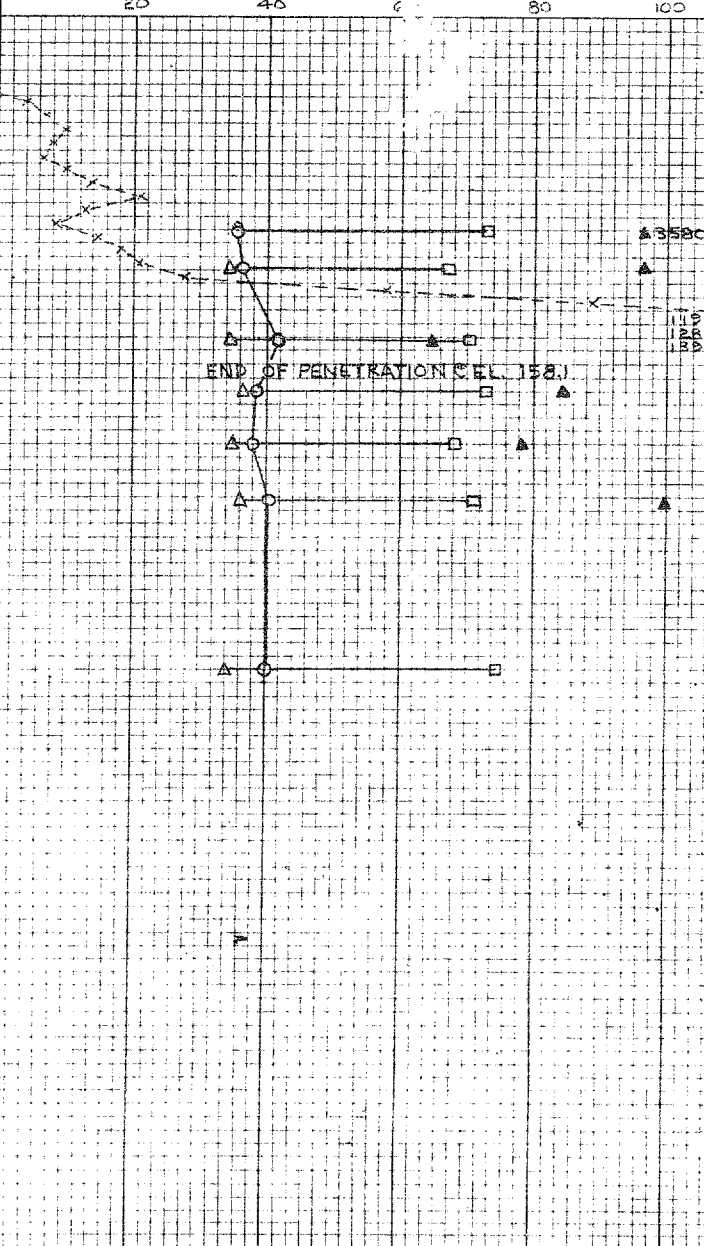
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
177.1 0.0		GROUND LEVEL		180
167.1 10.0		LOOSE BROWN AND GREY SAND AND SANDY SILT		170
	HOLE CAVED REL. 175.0 - JAN. 30, 1960	STIFF TO VERY STIFF GREY SILTY CLAY WITH PEBBLE GRAVEL SCATTERED THROUGHOUT		160
132.6 44.5		END OF HOLE		150

SHEAR STRENGTH IN LBS./SQ. FT. ▲ TRIAXIAL UNDRAINED

500 1000 1500 2000 2500

WATER CONTENT W% 10 20 30 40 50 NAT. LW PW

DYNAMIC PENETRATION TEST BLOWS PER FOOT 20 40 60 80 100



OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
2" DO	1	7	
DO	2	9	
SO	3	8	
SO	4	22	
SO	5	16	
SO	6	16	
DF	7	-	
SO	8	20	
SO	9	23	
3" TO	10	HYD PUSH	
2" SO	11	31	
SO	12	35	
SO	13	HYD PUSH	

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S-7035-A BORING # 4 DATUM COUNTY OF ELGIN CASING BX
 BORING DATE FEB. 10, 1960 REPORT DATE FEB. 24, 1960 COMPILED BY MLV CHECKED BY AMZ
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION



DISTURBED
FAIR
GOOD
LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE
F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
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S - TRIAXIAL SLOW
* - HYDRAULIC PUSH
γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE

ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE
153.4 0.0		RIVER LEVEL		150
		WATER		
147.1 6.3		RIVER BOTTOM		
142.6 10.5		VERY LOOSE BROWN SAND		140
		STIFF TO VERY STIFF GREY SILTY CLAY WITH PEBBLE GRAVEL SCATTERED THROUGHOUT		130
113.1 40.3		END OF HOLE		120
				110

SHEAR STRENGTH IN LBS./SQ. FT. ▲ TRIAXIAL UNDRAINED

500 1000 1500 2000 2500

WATER CONTENT W% 10 20 30 40 50

DYNAMIC PENETRATION TEST BLOWS PER FOOT

OTHER TESTS

SAMPLES

CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
WS	1	1	
DO	2	5	
DO	3	10	
TO	4	HYD	*
DO	5	10	
DO	6	18	
TO	7	HYD	PUSH
TO	8	44	
TO	9	HYD	PUSH

γ=131

γ=131

OFFICE REPORT ON SOIL EXPLORATION

CONTRACT S7035 A BORING # 4 A DATUM COUNTY OF ELGIN CASING 8X
 BORING DATE FEB. 12, 1960 REPORT DATE MARCH 3, 1960 COMPILED BY M.W. CHECKED BY J. H. B.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS ENERGY)

SAMPLE CONDITION



A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE

SAMPLE TYPES

F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

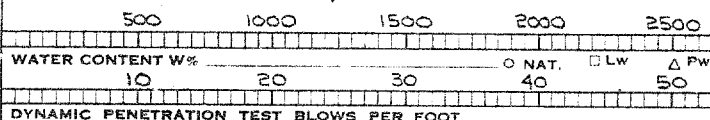
ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 OC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 * - HYDRAULIC PUSH
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE

SHEAR STRENGTH IN LBS./SQ.FT.

▲ TRIAXIAL UNDRAINED

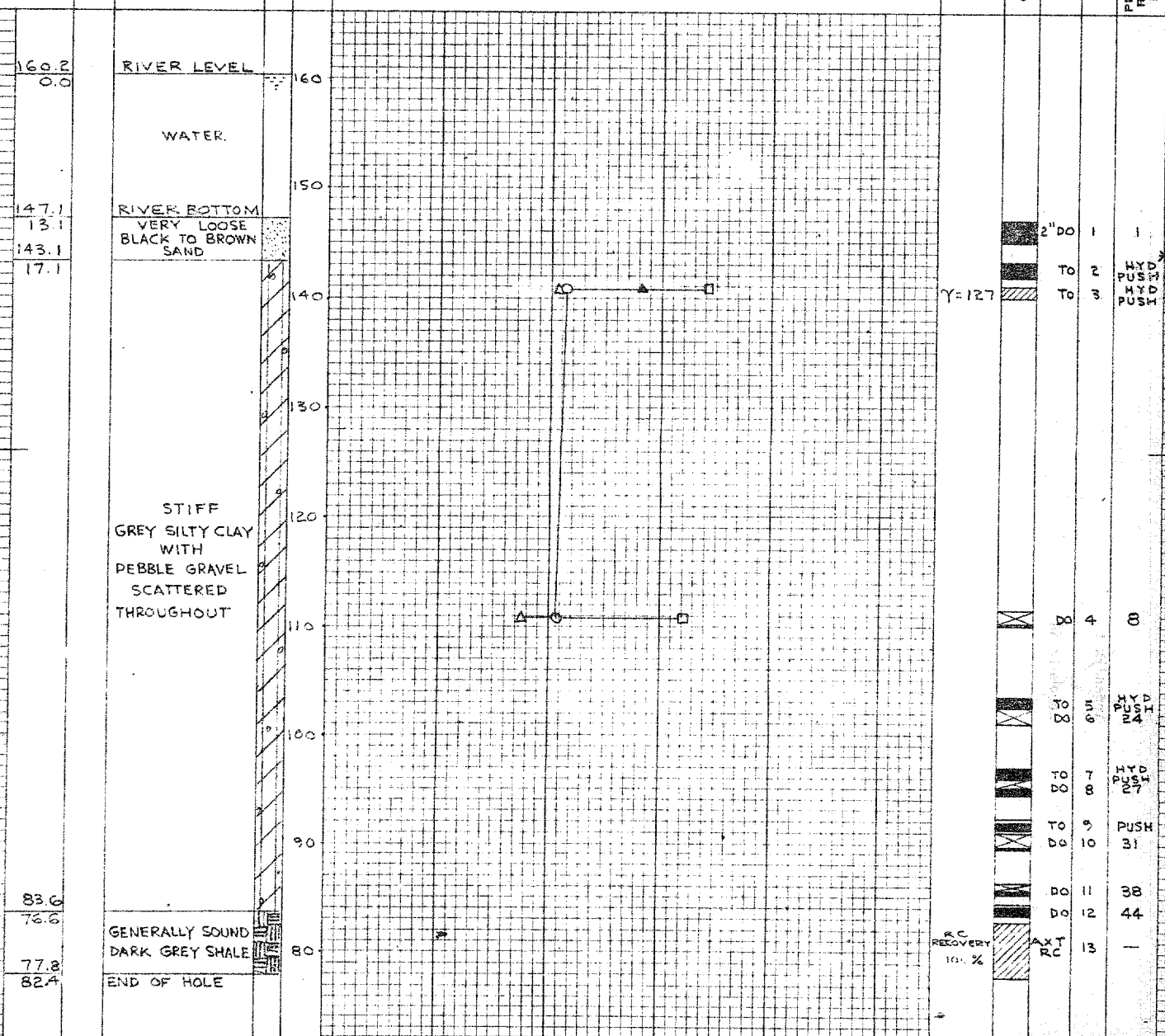


DYNAMIC PENETRATION TEST BLOWS PER FOOT

SAMPLES

CONDITION
 TYPE
 NUMBER
 PENETRATION RESISTANCE BLOWS/FT.

OTHER TESTS



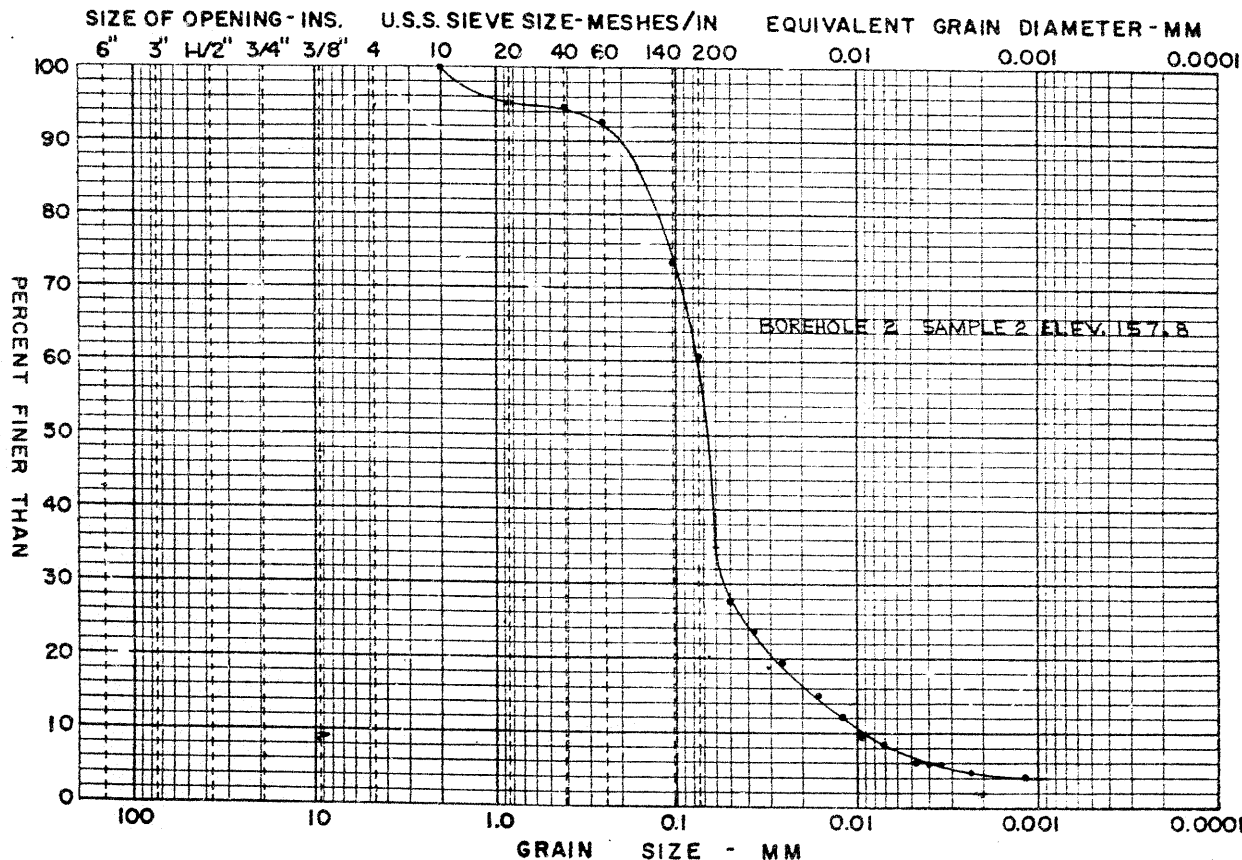
APPENDIX II

FIGURES - LABORATORY TESTING

GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 1
PROJECT S7035A

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

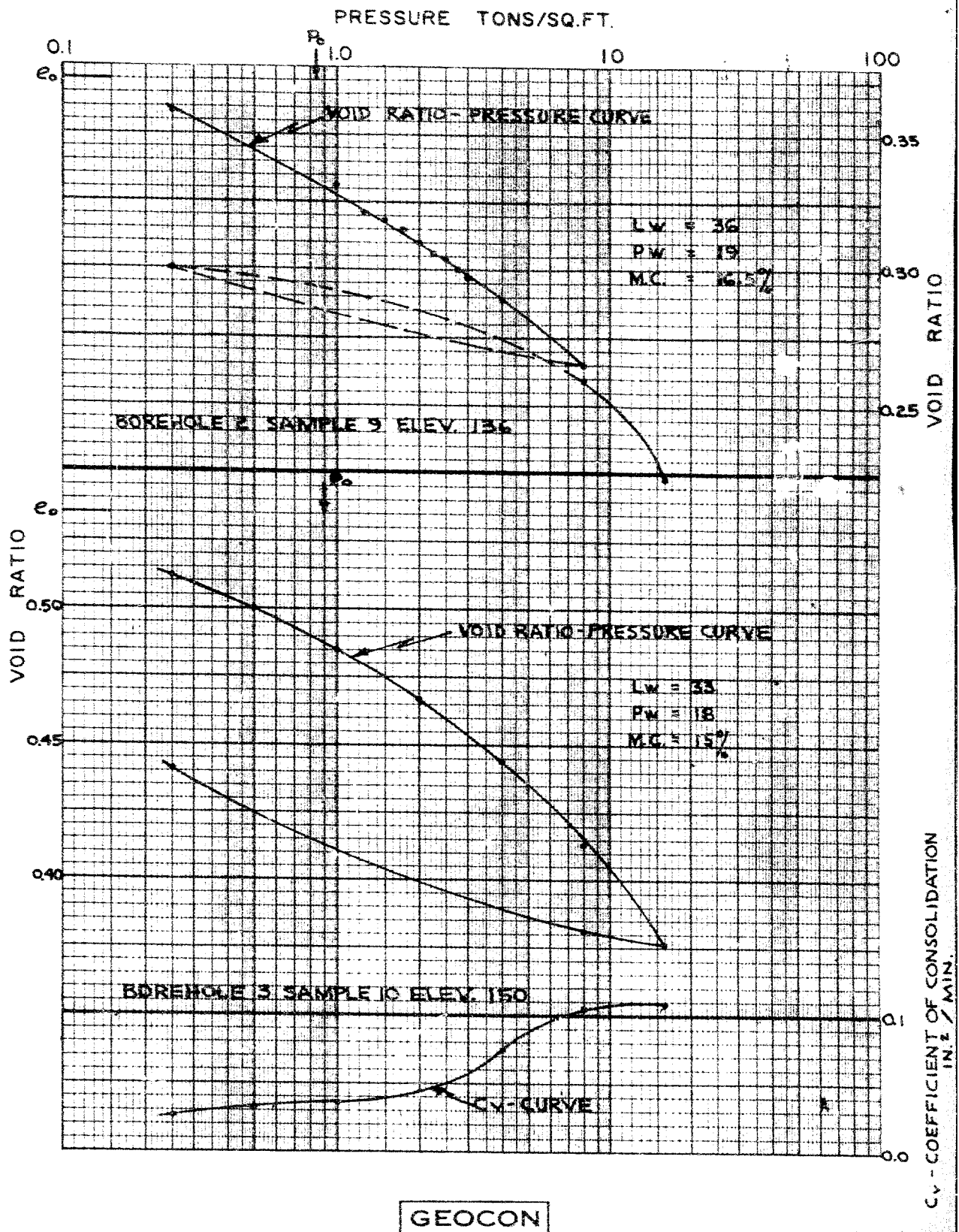


M.I.T. GRAIN SIZE SCALE

GEOCON

VOID RATIO-PRESSURE CURVES CONSOLIDATION TEST

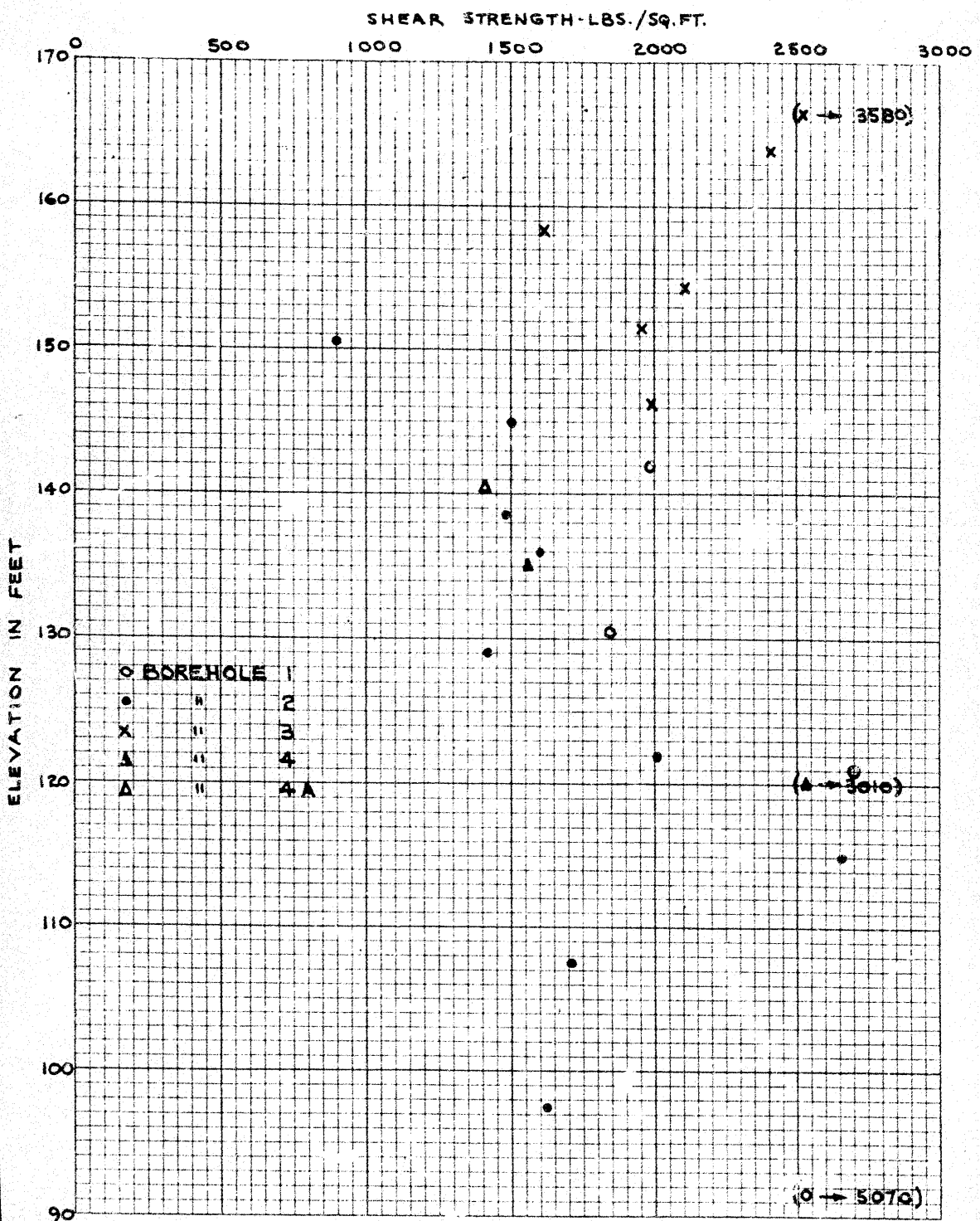
APPENDIX II
FIGURE 2
PROJECT S7035 A



SHEAR STRENGTH VS ELEVATION

GREY SILTY CLAY

APPENDIX II
FIGURE 3
PROJECT S7035 A



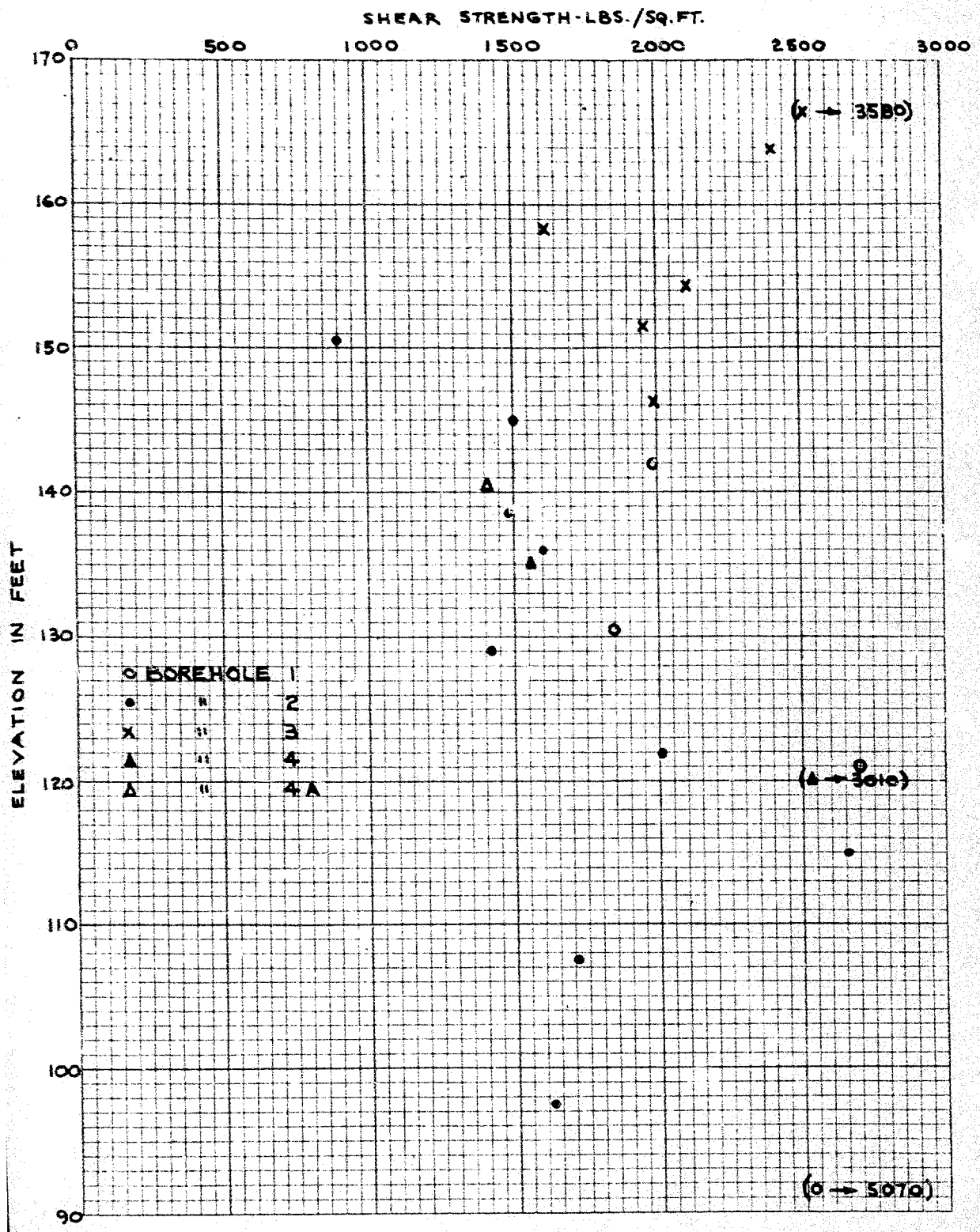
SHEAR STRENGTH VS ELEVATION

GREY SILTY CLAY

APPENDIX II

FIGURE 3

PROJECT 57035 A



40I 12-13

<u>Maintenance Record</u>		<u>Inspection Record</u>		
<u>Date</u>	<u>Remarks</u>	<u>Date</u>	<u>Remarks</u>	<u>Date</u>
	Inspected in spring of 1965 by R.C. Dunn & D.H.C. Bridge Engineer. Some settlement of Abutments and Piers occurring. Checks continuing to appear in Deck. See Office File	Jan.23/68	Some checks in Deck Asphalt, Otherwise condition good	

5-37

STRUCTURAL APPRAISAL

53. MATERIAL:

REINFORCED CONCRETE	_____	COMPOSITE SECTION CONCRETE AND TIMBER	_____
PRESTRESSED CONCRETE POST-TENSIONED	_____	GLUED-LAMINATED STRUCTURAL TIMBER	_____
PRESTRESSED CONCRETE PRE-TENSIONED	_____	TREATED TIMBER	_____
STRUCTURAL STEEL	_____	UNTREATED TIMBER	_____
STRUCTURAL STEEL WELDED	_____	STONE	_____
COMPOSITE SECTION STEEL AND CONCRETE	<u>X</u>	STEEL WELDED COMPOSITE	_____
STEEL AND TIMBER	_____		

54. FRAMING:

SIMPLE SPAN	_____	BAILEY	_____
CANTILEVER SUSPENSION SPAN	_____	TUNNEL	_____
CONTINUOUS SPANS	<u>X</u>	HOLLOW DECK	_____
SEMI-CONTINUOUS SPANS	_____	OPEN CULVERT	_____
SINGLE RIGID FRAME	_____	BOX CULVERT	_____
MULTI-SPAN RIGID FRAME	_____	MULTI-PLATE PIPE ARCH	_____
ARCH	_____	CORRUGATED STEEL PIPE	_____
SWING	_____	BARREL ARCH	_____
BASCULE	_____	OTHER	_____
LIFT	_____		

55. TYPE OF DESIGN:

SLAB	_____	PLATE GIRDER	<u>X</u>
BEAMS	_____	TRUSS (ALSO SEE BELOW)	_____
T. GIRDER	_____	BOAT TYPE	_____
BOX GIRDER	_____	UNDEFINED	_____

TRUSSES ONLY

STEEL	THROUGH	FLOOR BEAMS:	STRINGERS:	DECK:
TIMBER	LOW	STEEL	STEEL	TIMBER
	DECK	TIMBER	TIMBER	CONCRETE

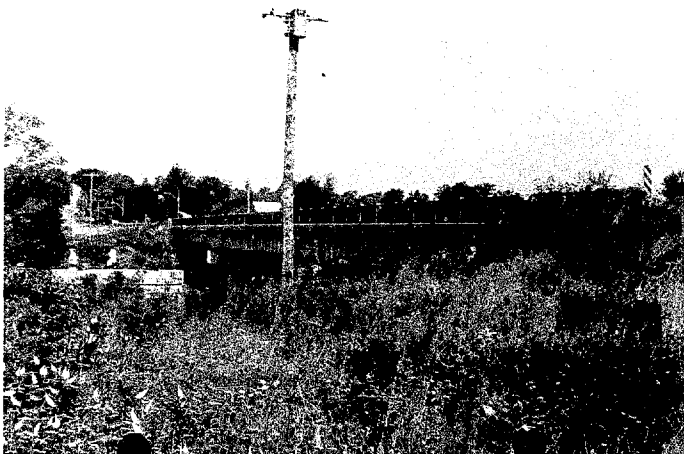
56. NUMBER OF SPANS 1 @ 140' 2 @ 100' = 340'

57. YEAR CONSTRUCTED 1961 58. DECK LENGTH 346 FT. 59. DECK WIDTH 36 FT.

60. PRESENT CONDITION (DESCRIBE):

(A) DECK AND SUPERSTRUCTURE ASPHALT SURFACE CRACKED - NORTH
ABUTMENT ROCKERS TIPPED, OTHERWISE GOOD

(B) SUB-STRUCTURE GOOD



5-37

STRUCTURAL APPRAISAL

53. MATERIAL:

REINFORCED CONCRETE	_____	COMPOSITE SECTION CONCRETE AND TIMBER	_____
PRESTRESSED CONCRETE POST-TENSIONED	_____	GLUED-LAMINATED STRUCTURAL TIMBER ..	_____
PRESTRESSED CONCRETE PRE-TENSIONED	_____	TREATED TIMBER	_____
STRUCTURAL STEEL	_____	UNTREATED TIMBER	_____
STRUCTURAL STEEL WELDED	_____	STONE	_____
COMPOSITE SECTION STEEL AND CONCRETE	<input checked="" type="checkbox"/>	STEEL WELDED COMPOSITE	_____
STEEL AND TIMBER	_____		

54. FRAMING:

SIMPLE SPAN	_____	BAILEY	_____
CANTILEVER SUSPENSION SPAN	_____	TUNNEL	_____
CONTINUOUS SPANS	<input checked="" type="checkbox"/>	HOLLOW DECK	_____
SEMI-CONTINUOUS SPANS	_____	OPEN CULVERT	_____
SINGLE RIGID FRAME	_____	BOX CULVERT	_____
MULTI-SPAN RIGID FRAME	_____	MULTI-PLATE PIPE ARCH	_____
ARCH	_____	CORRUGATED STEEL PIPE	_____
SWING	_____	BARREL ARCH	_____
BASCULE	_____	OTHER	_____
LIFT	_____		

55. TYPE OF DESIGN:

SLAB	_____	PLATE GIRDER	<input checked="" type="checkbox"/>
BEAMS	_____	TRUSS (ALSO SEE BELOW)	_____
T. GIRDER	_____	BOAT TYPE	_____
BOX GIRDER	_____	UNDEFINED	_____

TRUSSES ONLY

STEEL	THROUGH	FLOOR BEAMS:	STRINGERS:	DECK:
TIMBER	LOW	STEEL	STEEL	TIMBER
	DECK	TIMBER	TIMBER	CONCRETE

56. NUMBER OF SPANS 1 @ 140', 2 @ 100'

57. YEAR CONSTRUCTED 1961 58. DECK LENGTH 346 FT. 59. DECK WIDTH 36 FT.

60. PRESENT CONDITION (DESCRIBE):

- (A) DECK AND SUPERSTRUCTURE ASPHALT CRACKED, SHOULD BE
SURFACED SEALED EXPANSION BEARING ROLLERS
LEAKING ON NORTH ABUTMENT
- (B) SUB-STRUCTURE OK



PHOTO, BRIDGE

#60-F-292-C
PROPOSED THAMES
RIVER BRIDGE,
WARDSVILLE

