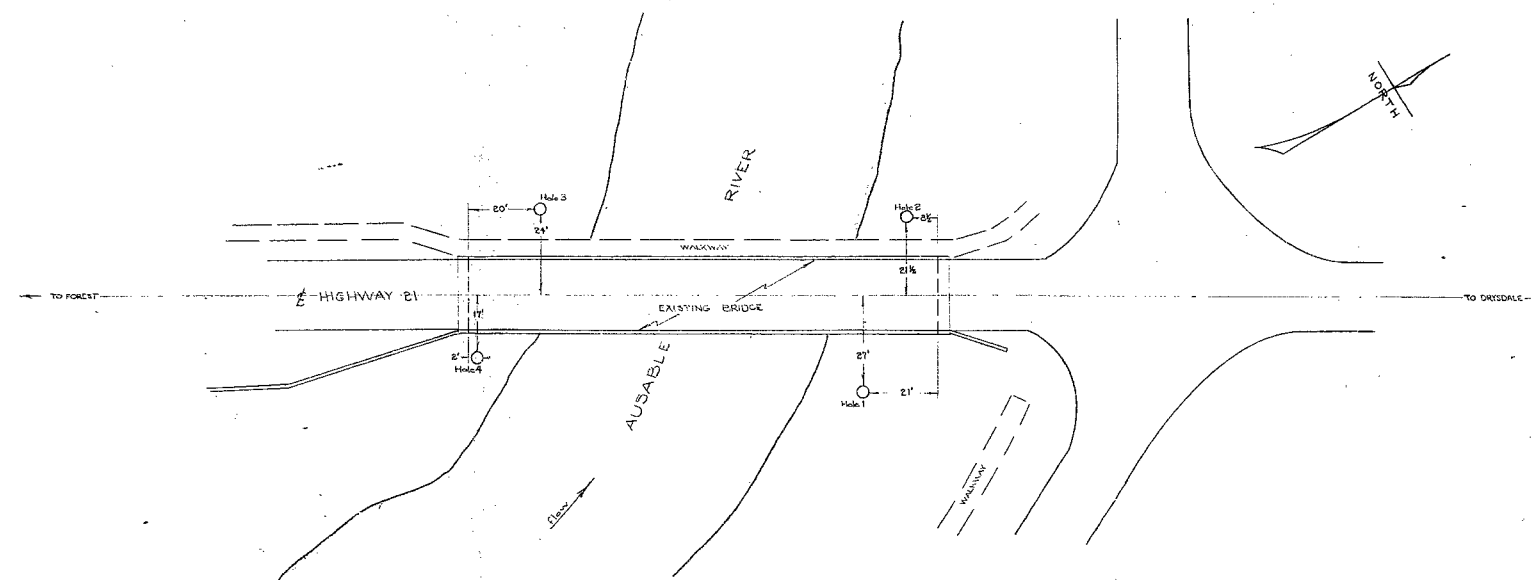


#60-F-258C

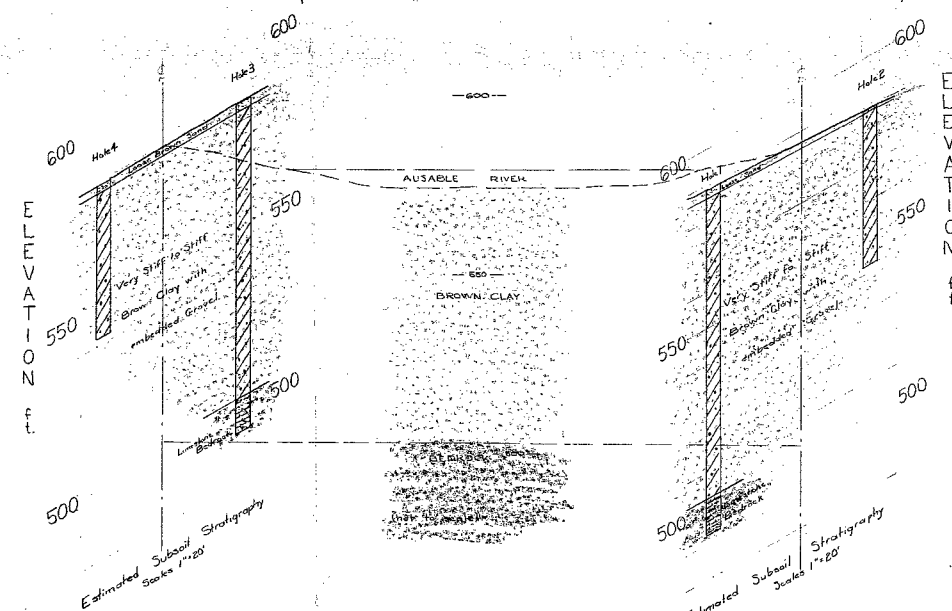
W.P. #178-59

Hwy # 21

AT GRAND BEND



Borehole Location Plan
SCALE 1"=20'



Proposed Bridge Replacement
Hwy. 21 - Grand Bend - Ausable R.
FOUNDATION INVESTIGATION
William A. Tren & Associates, Ltd.

Memo to Mr. A. M. Toye, Date January 20, 1960.
Bridge Engineer. Subject Re: FOUNDATION REPORT - by -
From Materials & Research Section, William A. Trow & Associates.

Attention: Mr. S. McCombie.

Re: Ausable River Crossing -
Hwy. #21 at Grand Bend,
W.P. 178-59 - District #1.

This memo accompanies the detailed foundation report prepared by W. A. Trow & Associates, at the above site. A review of this report by the Foundation Section has resulted in the following comments:-

1. The site is underlain by a stiff to very stiff layer of silty clay (glacial till). Under a thin layer of topsoil this stratum of silty clay continues to the limestone bedrock some 100 ft. below ground surface.
2. The proposed structure may be supported on a spread footing or piled foundation. If a spread footing type foundation is to be used, an allowable bearing pressure of 4500 p.s.f. may be used at or below elevation 580'. If the spread footings are founded between elevation 580' and elevation 565', scour protection must be provided. Steel sheet piles driven to elevation 563' - (10 feet below river bed) will provide positive scour protection; however, because of the resistance to scour offered by the clay, rip-rap protection may be sufficient. This shall be confirmed by the Hydrology Section. Placing the spread footing at elevation 565', as proposed by the Consultant, is not recommended because of the depth of excavation required.

If some delay between the time of excavation for the foundations, and the pouring of the concrete for the foundations is expected, a working mat of approx. 4" to 6" concrete should be placed immediately after the excavation. This working mat will prevent the softening of the exposed silty clay.

cont'd. /2 ...

March 22, 1963

Phone call by John Forster, Soils Engr. London Region;

Detour is over timber trestle. On drawing no indication is given of how far the timber piles have to be driven. Could we find out?²

The detour bridge is designed by the Bridge Maintenance Section (Bill Birch) and they have informed us (March 22, 1963) that they are going to send to the District all necessary quantities. Mr Birch is of the opinion that this being a temporary structure it should be the Contractor's responsibility to build it and for that he should supply the piles and be paid a lump sum for the structure.

This was in turn phone back to John Forster, same day.

afternoon

If a piled foundation is selected, two alternatives may be studied:- Steel 'H' piles approx. 80 feet in length may be driven to bedrock at elevation 503'. Pile sections such as 12" @ 53 lbs. should be designed to carry load of approx. 50 tons/pile. If friction piles are to be used, wooden piles, because of the difficult driving, are not recommended. Thus, friction piles should be of the concrete filled steel tube type pile. These steel tube piles should be designed to carry load less than 35 tons/pile, and the piles should be driven to at least elevation 545'.

No problems associated with embankment stability are anticipated. Seepage water in excavations will be small, and easily handled by low capacity pumps.

If further queries arise regarding this report, please contact the Foundation Section.

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

per:

K Peaker

(K. Peaker,
Foundation Supervising Field Engr.)

KP/MdeF
Encl.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
A. Gater
G. U. Howell
J. Roy
A. Watt
Foundation Section
Gen. Files.

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
AND
SOIL MECHANICS CONSULTATION

W. A. TROW, M.A.S.C., M.E.I.C., P.ENG.

884 WILSON AVE.,
DOWNSVIEW, ONT.
ME. 5-5921

Project: J 453

January 14, 1960.

Mr. A. Rutka,
Acting Materials & Research Engineer,
Materials and Research Division,
Department of Highways of Ontario,
Parliament Bldgs.,
Toronto 2, Ont.

Attention: Mr. L. G. Soderman (P. Eng.)
Principal Soils & Foundation Engineer

Foundation Investigation
Proposed Bridge Replacement
Hwy. 21, Crossing of Ausable River, Grand Bend, Ont.

Dear Sirs:

Enclosed herewith is our report on the soil conditions underlying the proposed bridge replacement at the location noted above. Four borings were involved in this work, which was undertaken during the last two weeks of December.

Reference to the stratigraphical profile of Dwg. 1 in this report indicates that the foundation soil here is a stiff to very stiff clay till. No significant variation in the physical properties of this soil was noted down to limestone bedrock level, found about 100 feet below the bridge deck.

Abutments can be supported either on simple footings or on piles. The safe bearing value for footings has been determined as 4500 p.s.f. In order to provide scour protection, support at Elev. 565 feet, or 8 feet below the river bed is recommended. H piles to bedrock, or shorter cylindrical piles bearing at Elev. 550 feet, are the foundation alternatives. The capacity of a single cylindrical pile has been computed to be 38 tons. Considerable resistance to pile driving in this stiff clay is anticipated.

We believe that the contents of this report contain sufficient information for the design of foundations for this structure. However, if circumstances other than those considered come to mind after you have reviewed the contents, we shall be pleased to discuss them with you.

Yours very truly,

W. Trow

William A. Trow (P. Eng.)

WAT/lr
Encl.

DEPARTMENT OF HIGHWAYS OF ONTARIO
MATERIALS AND RESEARCH DIVISION
PARLIAMENT BUILDINGS, TORONTO 2.

FOUNDATION INVESTIGATION
PROPOSED BRIDGE REPLACEMENT
HWY. 21, CROSSING OF AUSABLE RIVER, GRAND BEND.

Project: J 453

Jan. 14, 1960.

William A. Trow & Associates Ltd.

TABLE OF CONTENTS

Site Description	P. 1
Subsoil Description	1
Discussion of Foundation Requirements	2
Summary of Comments and Recommendations	4

APPENDIX

Field Investigation Methods	(i)
Settlement Calculations	(ii)

ENCLOSURES

Summary of Laboratory Test Data	Table 1
Photographs of Site	
Borehole Location Plan	Dwg. 1
Borehole Profiles, Holes 1 to 4	2-5
Typical Grain size distribution for Clay till	6
Consolidation Curves	7-9
Stress Strain curves - Undrained triaxial tests	10-11

FOUNDATION INVESTIGATION
PROPOSED BRIDGE REPLACEMENT
HWY. 21, CROSSING OF AUSABLE RIVER, GRAND BEND, ONT.

The contents of this report are concerned with the foundation conditions underlying the existing bridge and its proposed replacement at the location noted above. The types of soil and rock at this site are indicated and their competence for the support of abutment loads is considered. Details concerning field investigation methods are presented in Appendix 1.

Site Description

The Ausable River makes a gentle meander to the left through the town of Grand Bend. A steel truss structure presently spans the river at the location of the proposed bridge. This site is approximately one-half a mile from the mouth of the river where it empties into Lake Huron.

The river runs through relatively level terrain over this last part of its course and has cut a channel some 25 feet in depth. Its banks appear stable for the most part with slopes of 2:1 or steeper. There are some evidences of toe erosion on both the convex and concave sides of the stream. A local failure has occurred on the concave side a short distance downstream of the bridge where slopes are generally 1:1.

Because of the proximity to the lake, the river should be quite stable in size at the bridge site. Unless the lake level were to drop, it is not likely that the river will cut its channel to any appreciably greater depth than exists at the present time.

The existing bridge is a single span steel truss type of structure. The abutments appear sound. Concrete rubble placed between the abutments and the river appears to give satisfactory protection against scour of the banks in the immediate vicinity of the bridge.

Subsoil Description

The foundation soil at this site is a glacial deposit composed of very stiff to stiff clay with varying amounts of embedded fine gravel or grits. Except for a very thin surface covering of sand found above Elev. 586 feet, approximately, this material extends from ground surface down to limestone bedrock, which is located about 100 feet below the deck of the existing bridge. Although minor variations in plasticity and in shear strength were noted, the soil can be considered quite uniform both in horizontal and vertical directions.

The description and graphical summary of some of the physical properties of the clay till are recorded in the borehole logs for the four borings, Dws. 2 to 5 of this report. An additional summary of laboratory and field test measurements is given in Table 1.

It is seen that the shear strength of the clay ranges from a low value of about 1400 p.s.f. to resistances in excess of 2400 p.s.f. Its sensitivity to remoulding is in the order of 2. The liquid limit of this material lies between values of 32 and 39 percent dry weight, with the more plastic material occurring below approximate Elev. 555 feet. The moisture content of the soil lies relatively close to the plastic limit, particularly within the first 30 feet of depth. One hydrometer analysis of this material, shown in Dwg. 6, indicates a clay fraction of 47%. The ratio of plasticity to clay fraction is equal approximately to 0.5. Therefore, the soil can be classed as inactive. The results of consolidation tests indicated in Dwgs. 7 to 9 show that the soil is highly overconsolidated, as would be expected in view of the high shear strength and low moisture content of the clay. The tests on 1.9 inch samples are presented for academic interest only, for comparison with the results of tests on larger specimens.

Bedrock, consisting of dense light grey limestone, was encountered at Elev. 503 feet. Good recovery was obtained in two separate locations on diagonally opposite sides of the river, although some thin mud seams were noted just below rock contact in Hole 3 at the south-west corner of the existing bridge.

Discussion of Foundation Requirements

Two design requirements must be met for the satisfactory support of the proposed bridge structure. The foundations must be supported on soil strong enough to carry the bridge weight safely and without objectionable settlement; and the bearing depth must be below the maximum anticipated level of scour.

Since the river bed elevation presently lies approximately at Elev. 573 feet, the footings of the adjacent bridge abutment must extend at least to this level. Although this glacial clay should be relatively resistant to scour, some factor of safety against undermining should be provided and therefore support at approximate Elev. 565 feet is recommended. At and just below this level, the shear strength of the clay ranges from about 1450 p.s.f. to 2400 p.s.f. For design purposes, a value of approximately 1800 p.s.f. is suggested. Therefore, if economical, bridge abutments can be supported directly on the clay till at this level. The safe net bearing value, in excess of overburden weight, to apply for long rectangular bridge footings is equal to-

$$q = \frac{N}{F} C = \frac{7.5}{3} \times 1800 = 4500 \text{ p.s.f.}$$

In the foregoing expression, $N = 7.5$ is the bearing capacity factor for a continuous footing founded well below ground surface; $F = 3$ is the recommended factor of safety for this stiff clay in order to deep settlement within tolerable limits.

The estimated long term settlement to be anticipated under this footing load has been computed in Appendix 2. This estimate is based upon the results of the consolidation test on a sample taken from 35 feet in Hole 2. A value of $s = 1$ inch, approximately, has been determined.

The excavation, for the foregoing proposal of abutment support at Elev. 565 feet, should not involve serious construction difficulties since the clay till is quite strong and is impermeable. Accordingly, the footing beds should remain relatively dry while digging operations are underway. In addition, the sides and base of the excavation will remain stable without lateral support, even though the depth of the cut will be of the order of 28 feet. However, for psychological reasons and also to conform to the Trench Excavator's Act, some shoring of the earth walls will be required.

If the foregoing foundation proposal is considered to be uneconomic, the alternative is to support the structure on piles bearing either in the clay or at greater depth on bedrock. Because of the very stiff nature of the soil, it is unlikely that large displacement piles can be driven the 85 feet to the limestone bedrock and therefore H piles will be required for support at this depth. The capacity of H piles will be determined by their structural properties, considering the units as short columns. Very little penetration into the dense limestone should be expected. According to the observations in Holes 1 and 3, the surface of bedrock is flat.

In order to provide lateral support to large displacement piles, in the unlikely event that the river bank scour extends back to the abutments, they should be driven to approximate elevation 550 feet. Considerable resistance to driving to this depth may be experienced and, as a consequence, it may be necessary to pre-bore for part of the depth.

The safe capacity of a single one-foot diameter pile, in this stiff clay till, is given by the expression:

$$\begin{aligned} q &= \frac{9C}{F} A + \pi d l S \\ &= \frac{9 \times 2000}{3} A + \pi \times 1 \times 23 \times 1000 \\ &= 38 \text{ tons, approximately,} \end{aligned}$$

where C is the shearing resistance of the soil at Elev. 550 feet.; F is the required factor of safety; S is the estimated adhesion force on the shaft of the pile; and $\pi d l$ is the surface area of the pile for a length, l , below Elev. 573 feet, or river bed level. No factor of safety has been applied to the second term, since S has been reduced to the estimated remoulded strength of the clay after driving and, in addition, no allowance has been made for the strength of the soil above river bed level. The possibility of river bank erosion extending back to the abutments appears to be remote, particularly shortly after construction when the adhesive force S is at its lowest value of 1000 p.s.f.

The estimated long term settlement of a single one-foot diameter pile has been computed in the Appendix to be about $1\frac{3}{4}$ inches. If the spacing of piles is of the order of 5 diameters or greater, each unit should perform as indicated in the foregoing computations. At closer spacing, the pressures transmitted by each pile will tend to overlap. The ultimate condition for very close spacing will be similar to the one computed for the deep continuous footing considered earlier in this discussion. Since the strength and compressibility of the clay appears to be uniform with depth, settlement of a similar magnitude should be experienced for a group of piles.

Summary of Comments and Recommendations

- 1) The proposed bridge replacement site is underlain below foundation depth by very stiff to stiff clay with gravel, a glacial deposit. This material extends to limestone bedrock about 100 feet below present bridge deck level and it appears quite uniform both in lateral and vertical directions. Some sand overlies the clay above about Elev. 586 feet. It is probable that the upper levels of the river bank consist of sand, although this was not confirmed visually in the investigation program because the ground was frozen and snow covered.
- 2) Support of the bridge structure either on footings or piles appears quite permissible provided the foundation depth provides protection against river bed scour. For direct support, footings should be taken down to Elev. 565 feet, or about 8 feet below the river bed level. The safe bearing value here has been computed to be 4500 p.s.f. If a pile foundation proves to be more economic, support either on bedrock, using H piles, or at Elev. 550 feet, using cylindrical steel piles, is recommended. The capacity of a single one foot diameter steel pile has been computed to be 35 tons, and the associated long term settlement should be less than $1\frac{3}{4}$ inches.
- 3) Some resistance to driving piles should be experienced and it may be necessary to pre-bore part of the way for the cylindrical units. Excavations for footings at Elev. 565 feet should be stable and remain relatively dry.

WAT/lt
Jan. 14, 1960
J453



W. Trow
William A. Trow (P. Eng.)

APPENDIXField Investigation Methods

Four borings were performed at this site to a maximum depth of about 95 feet. Wet sampling methods were used. The procedure involves driving casing to the required sampling depth, washing out the soil contained in it and then sampling below casing level. In this instance, the holes were cased with 4-inch pipe for the first 10 to 15 feet, but below this level, the soil was unsupported. The boring was advanced by washing to the desired sampling levels using BX casing.

Samples were taken at relatively close intervals of depth, particularly in Hole 1. Vane tests were performed 18 inches below sampling level, although in many cases, the soil was too stiff to permit insertion or turning of the vane. The samples from Holes 1 and 3 were recovered in thin-walled 2-inch I.D. Shelby tubes. In the other two holes, 2-inch O.D. split spoon samples were obtained. The driving energy required to obtain the desired penetration was equal to 350 ft. lbs. per blow. A large 3-inch shelly tube sample was recovered in Holes 2 and 3. All shelly tube samples were carefully sealed with low melting point wax after removal and then stored to prevent freezing prior to test. A representative portion of each split spoon sample was wiped clean and wrapped in tinfoil in order to preserve its natural moisture content. This and additional material of the sample was placed in a plastic bag.

The elevations of all holes were referred to the deck of the adjacent bridge which level was taken as 603.4 feet. The positions of the borings were referred to the existing abutments.

APPENDIXSettlement CalculationsFootings at Elev. 565 Feet

- Assumptions: 1) Footings 5 ft. wide by 40 ft. long,
net bearing pressure = 4500 p.s.f.
2) Abutment load dissipates into the ground
below footing level at 30° to vertical.
3) Natural Unit Wt. = 135 p.c.f. above water
table and 70 p.c.f. below water table, or El. 578 ft.
Average ground level = Elev. 595 ft.
4) Consolidation as per test result for Hole 2 - 35 ft.

Elev.	Depth Incr. inches	In Situ Press. Po	Press. Size	Press. Area (Sq.ft.)	Bearing Press.	P ₁	e	S = $24 \times \frac{e}{1.68}$
565		3200	5x40	200	4500			
564	24	3270	6.15x41.15	253	3550	6820	.017	.243
562	24	3410	8.45x43.45	367	2450	5860	.012	.172
560	24	3550	10.75x45.75	492	1830	5380	.010	.143
558	24	3690	13.05x48.05	627	1430	5120	.008	.114
556	24	3830	15.35x50.35	772	1165	4995	.007	.100

Total Settlement Approx. 1 inch.

Cylindrical Piles at Elev. 550 ft.

- Assumptions: 1) Each pile unaffected by adjacent pile
2) Soil compressed below lower third of
pile length = Elev. 563 ft.

563	24	3340	1 ft. diam.	.785	89200			
562		3410	2.15ft. "	3.65	19150	22560	.068	.97
560		3550	4.45 "	15.6	4490	8040	.024	.333
558		3690	6.75 "	36.0	1940	5630	.009	.129
556		3830	9.05 "	64.4	1090	4920	.006	.085

Total Settlement Approx. $1\frac{3}{4}$ ins.

This estimation is believed to be high since some redistribution of stresses along the pile must take place. It is impossible for the section of pile between Elev. 563 ft. and 561 ft. to move 0.97 inches without similar movement at greater depth.

TABLE NO. 1

SUMMARY OF LABORATORY AND FIELD TEST
MEASUREMENTS

Elev. Ft.	Shear Strength K.s.f. Hole	Moisture Content % Dry Weight	Atterberg Limits % Dry Wt.	Liquidity Index	Natural Unit Weight
Surf.-	1 3 4	1 2 3 4	L.L. P.L.		1 2 3
Elev.: 588.5	585.2 584.8	585.5			
580	1.86	17.5	Hole 1 except where noted.		136.0
576	2.40 V=1.76	18.6 18.1 21.7	18.5		137.2
572	1.83 V=1.85 V=1.51	19.4 16.7 21.6	20.5		
	1.51 1.45	20.2	33.2 14.9 .246	134.9	
568			34.0 15.3 .262	131.5	132.8
	1.92 V=1.85 V=1.60	20.1		134.0	
564	2.37	19.9	2 --	135.0	
		18.8			
560	1.97 1.44 V>2.1	20.6 21.4 21.3	33.7 15.7 .272	134.4	133.0
	V>2.1 V>2.1				
556	1.57	21.0 20.9 21.0		132.2	
			21.5		
552	1.96 V>2.52	23.4 18.6	(Hole 2)		
	2.11 1.94	23.2 22.4 23.4 22.7	39.6 17.3 .274	132.3	
548	V=1.99	21.5	39.2 17.6 .259	125.2 129.5	132.9
	2.03 V>2.1	23.5		129.7	
544	V>2.10 V>2.1	21.6 22.9 23.0			
	1.50 V>2.1	25.5	38.3 17.0 .399	128.2	
540	V=1.51 1.85	24.6			129.0
	V=1.85				
536	1.38 1.35	26.7 26.4		124.3	127.6
	V=1.68				
532	V=1.57				
	V=1.81		27.0		
528	1.6	25.0		37.7 17.0 .287	127.5
	V=1.85				
524	V>2.1	29.8	37.9 (H-3) 17.1 .612		
	V=1.60	26.4			
520	V=1.76 1.49	24.8			124.5
	V=1.89				



View Downstream - to West



View Upstream - to East



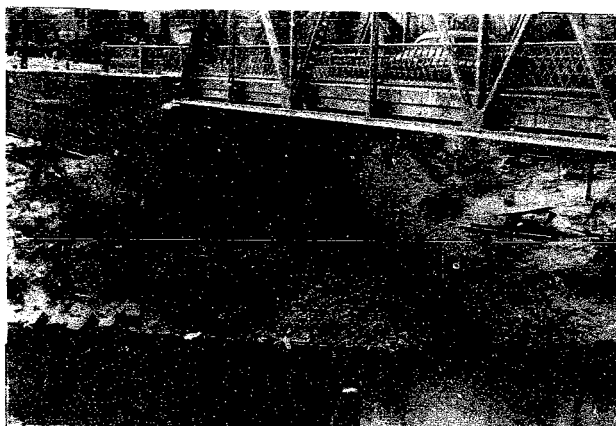
View Downstream - to West



View Upstream - to East



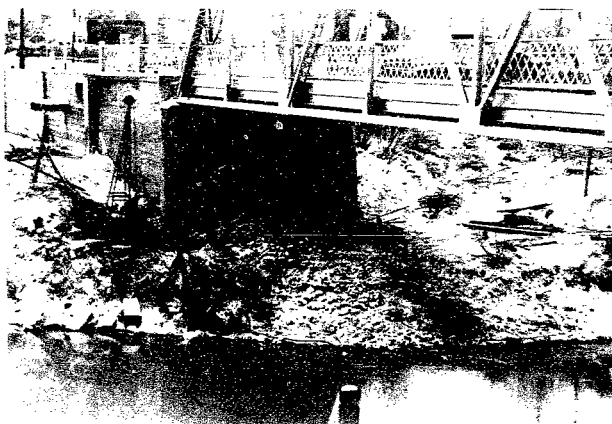
West side, South Abutment



South Abutment - drill on Hole 4



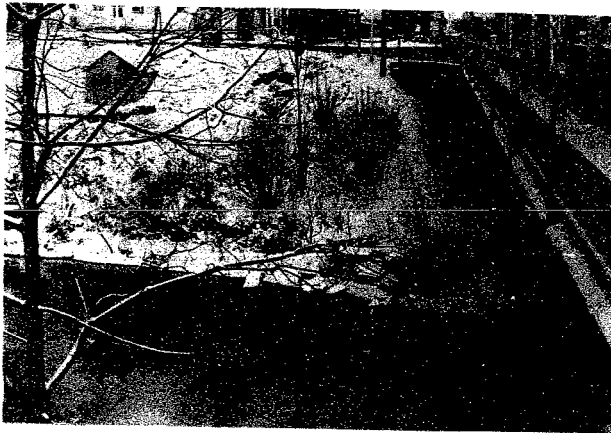
West side, South Abutment



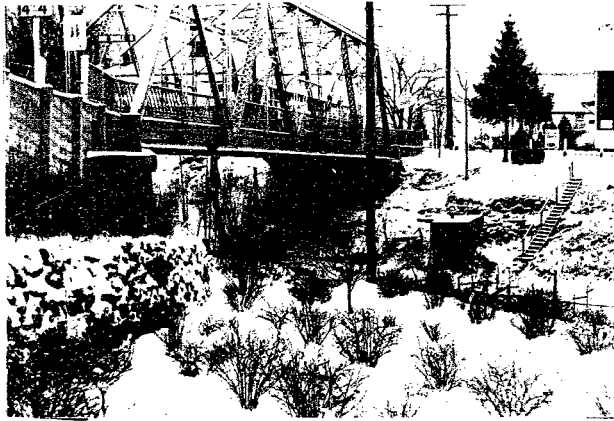
South Abutment - drill on Hole 1



East side of Bridge, looking North



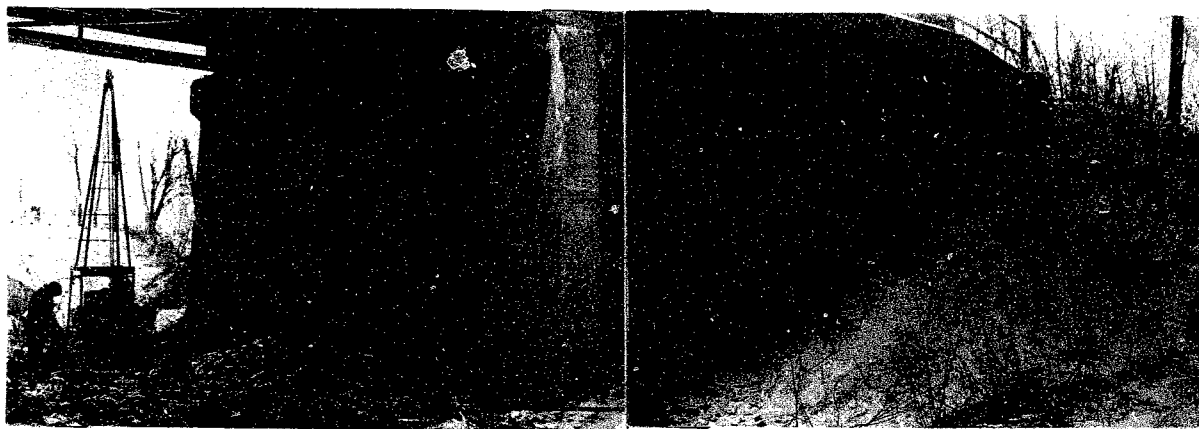
West side of Bridge, looking North-east.



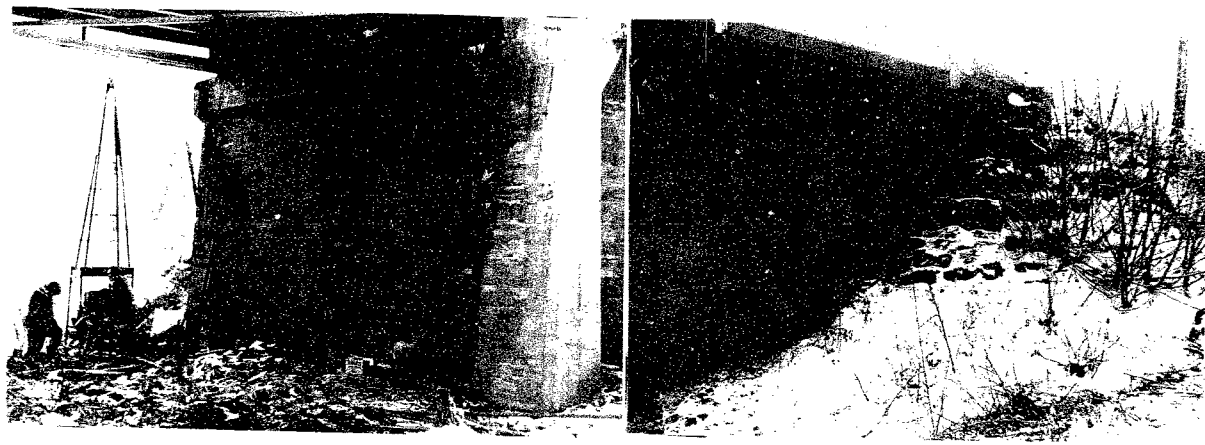
East side of Bridge, looking North



West side of Bridge, looking North-east.



South Abutment, looking from the North



South Abutment, looking from the North

WILLIAM A. TROW & ASSOCIATES LTD.

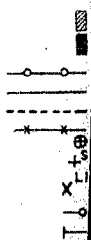
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Bridge Site, Highway 21 - Ausable R.
 LOCATION Grand Bend, Ontario
 HOLE LOCATION See dwg. 1
 HOLE ELEVATION AND DATUM 588.5 - Deck of exist.
 Bridge = 603.4 ft.

BOREHOLE NO. 1
 FIELD SUPERVISOR
 DRILLER
 PREP.

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (Qu)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				Shear Strength	1000	P.S.F. 2000	BLOWS/FT.
	Ground surface.	588.5	0				
	Loose brown med. sand (moist).	586.2					
	Glacial Till. Very stiff to stiff brown clay with embedded gravel. (Grey in colour below 15 ft.) Water level uncased hole = 575.6 after 2 days.		10				
			20				
			30				
			40				
			50				
			60				
			70				
			80				
		502.7					
	Bedrock - dense light grey limestone (100% recovery).		90				

- Notes: 1) Boring by wet sampling method; hole cased with 4 inch pipe to 11 ft.; cleared uncased hole below this level using EX casing.
 2) All Shelby tube samples pushed as far as possible then driven to final depth as noted; driving energy = 350 ft.lbs. per blow.
 3) Accurate water levels not possible, assumed = to river level.

SAMPLE	NATURAL UNIT WT. P.C.F.	CONSISTENCY		
		MOIST. CONTENT - % DRY WT.		
		10	20	30
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				
16				
17				
18				
19				
20				
21				
22				

End of hole at 95.9 ft.

PROJECT NO. J453

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Bridge Site, Highway 21 - Ausable R.

LOCATION Grand Bend, Ontario

HOLE LOCATION See dwg. 1

HOLE ELEVATION AND DATUM 585.5 - BM as in hole 1

BOREHOLE NO. 2

FIELD SUPERVISOR

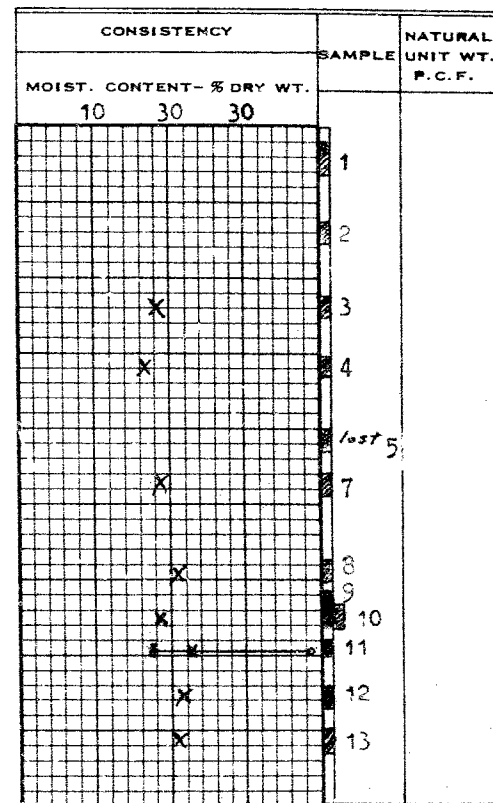
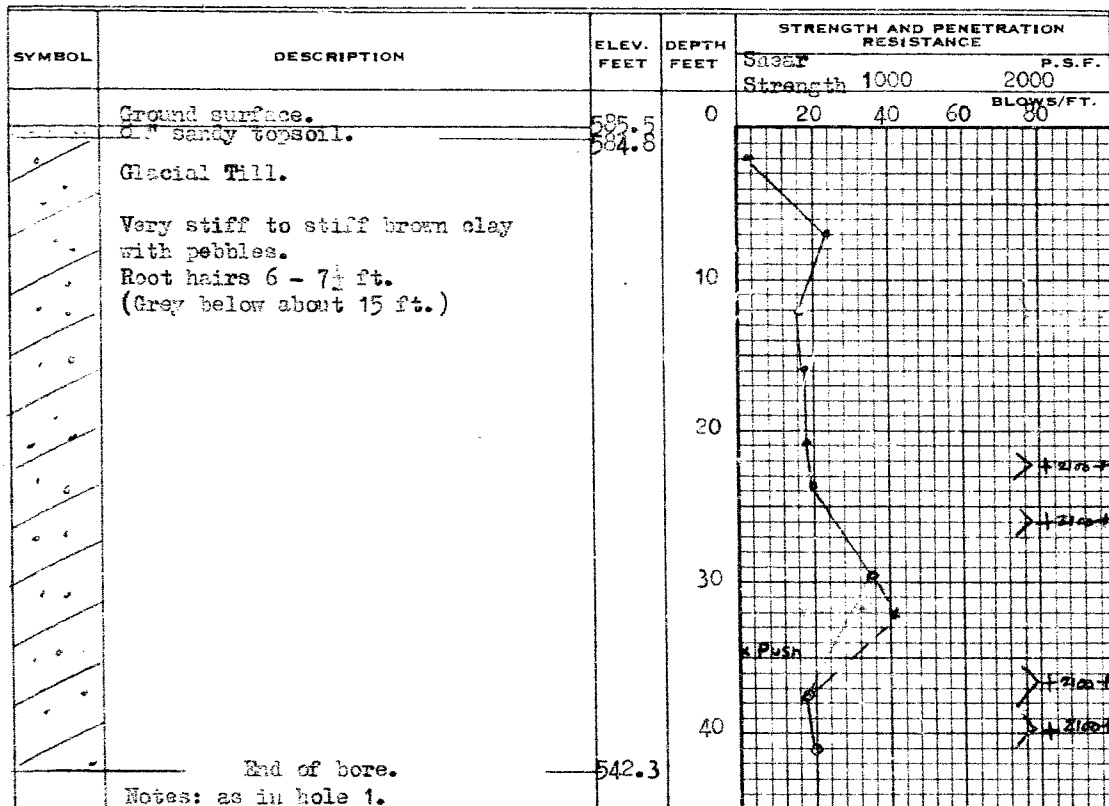
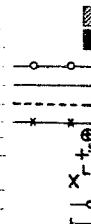
DRILLER

PREP.

DRAWING NO. 3

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



PROJECT NO. J453

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Bridge Site, Highway 21 - Ausable R.

LOCATION Grand Bend, Ontario.

HOLE LOCATION See dwg. 1

HOLE ELEVATION AND DATUM 685.2 - BM as in hole 1

BOREHOLE NO. 3

FIELD SUPERVISOR

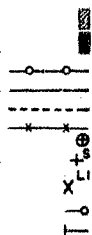
DRILLER

PREP.

DRAWING NO. 4

LEGEND

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONE
 CASING
 2" SHELBY
 1/2 UNCONFINED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				Shear Strength 1000	P.S.F. 2000
	Ground surface.	685.2	0		
	8" sandy topsoil.				
	Moist med. to coarse sand.	683.0			
	Glacial Till.				
	Very stiff to stiff brown clay with embedded gravel.		10		
	(Gray in colour below about 15 feet).		20		
			30		
			40		
			50		
			60		
			70		
			80		
			90		
	Bedrock. Hard light gray limestone some thin mud seams at 82.7, 84.1	503.3			
	81.9 - 82.7 - 90% recovery				
	82.7 - 87.8 - 27.5% recovery				
	87.8 - 91.3 - 25.5% recovery.				

CONSISTENCY			SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.				
10	20	30		
			1	
			2	
	x		3	
	x		4	
	x		5	
	x		6	
	x		7	
	x		8	
	x		9	
	x		10	
	x		11	
	x		12	
	x		13	
	x		14	
	x		15	
	x	x	16	
		x	17	

End of bore 91.3 feet.

Notes as in hole 1.

PROJECT NO. J453

DRAWING NO. 5

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Bridge Site, Highway 21 - Assiniboine R.

BOREHOLE NO. 4

LOCATION Grand Bend, Ontario.

FIELD SUPERVISOR..

HOLE LOCATION See dwg. 1

DRILLER.....

HOLE ELEVATION AND DATUM.. 534.8 - BM as in hole 1. PREP.....

PREP.

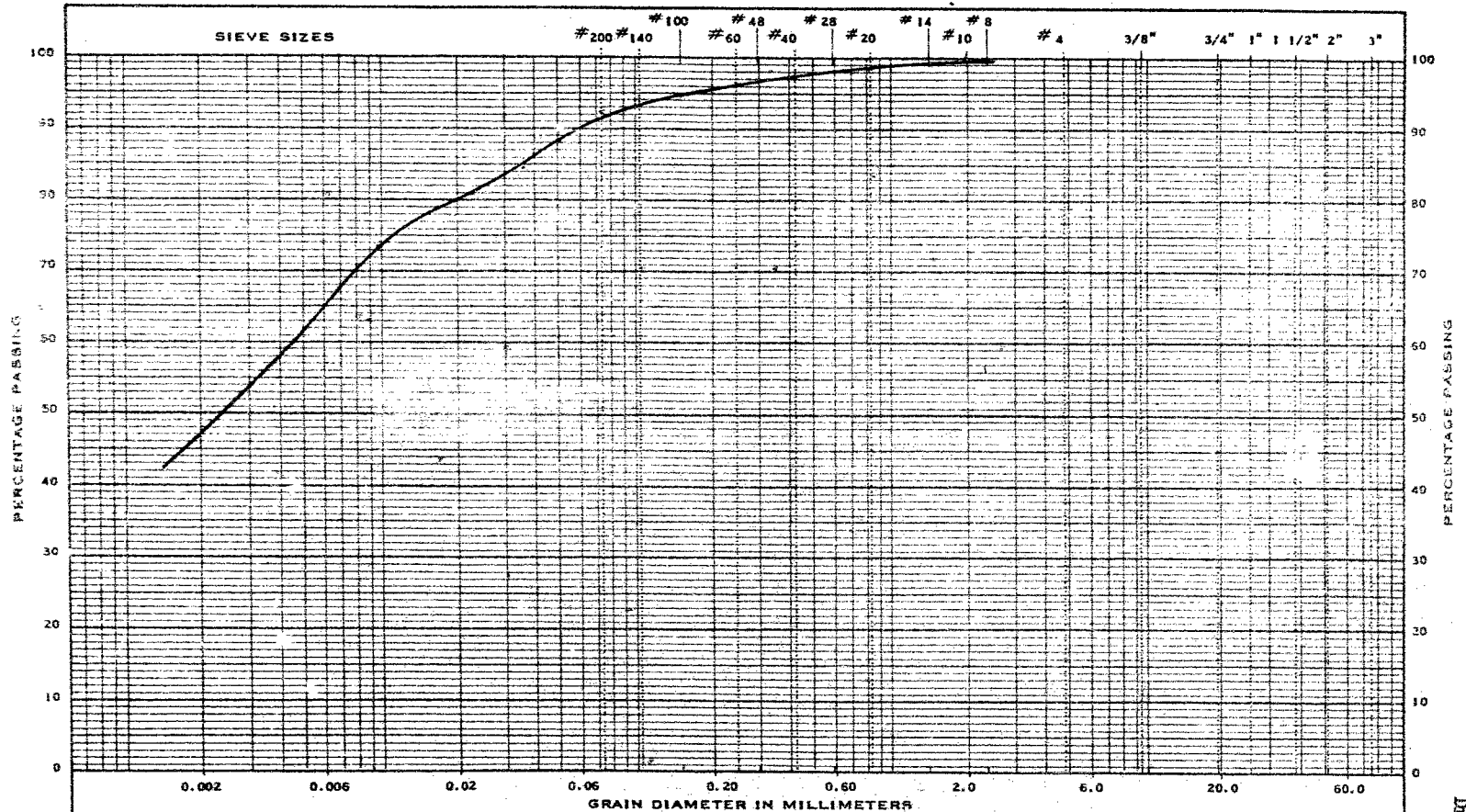
LEGEND

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
CASING
2" SHELBY
1/2 UNCONFINED COMPRESSION [Qu]
VANE TEST [C] AND SENSITIVITY [S]
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

[illegible]

CONSISTENCY			SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT- % DRY WT.				
10	20	30		
			1	
			2	
			3	
			4	
			5	
			6	
			7	
			8	
			9	

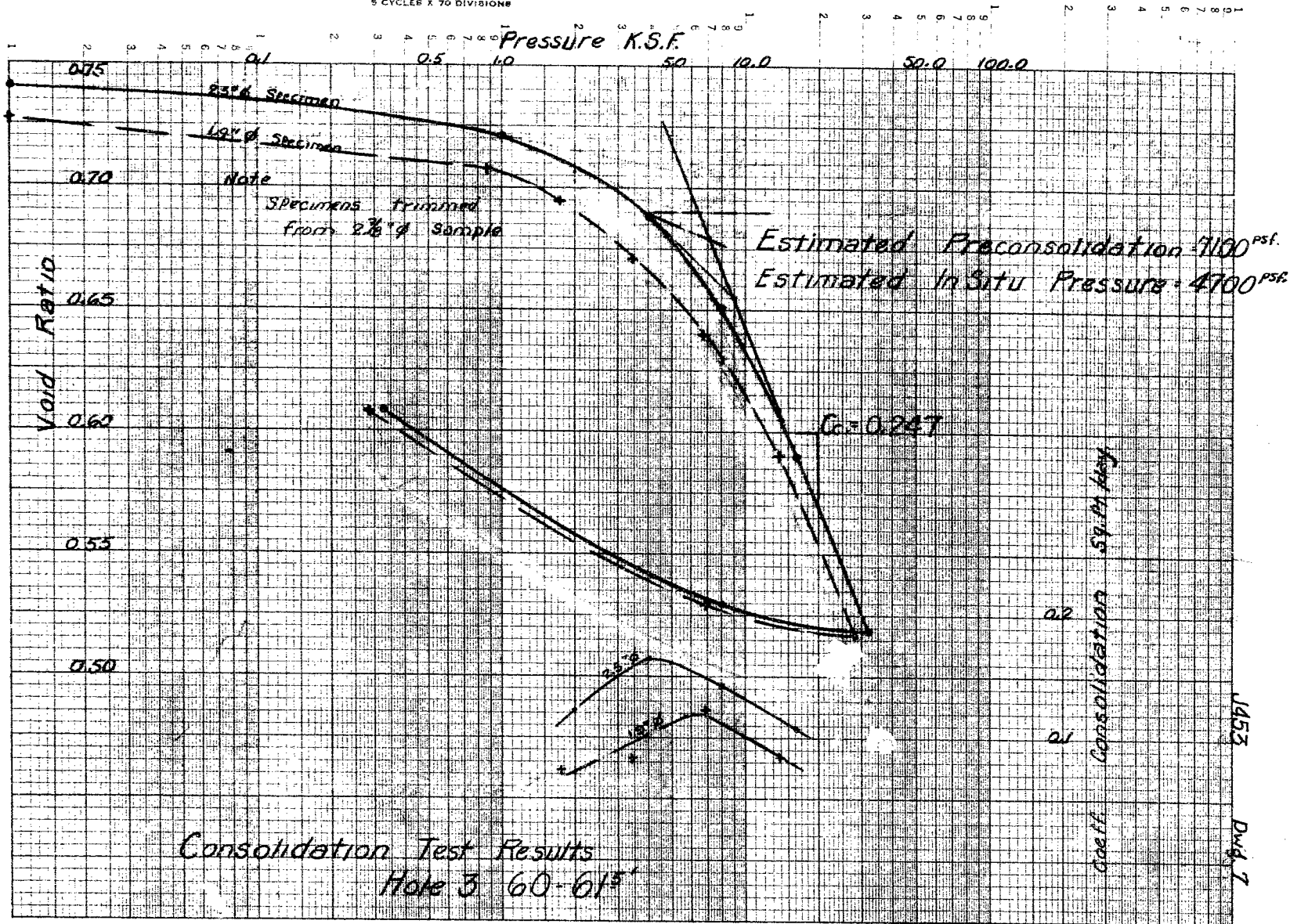
MECHANICAL ANALYSIS

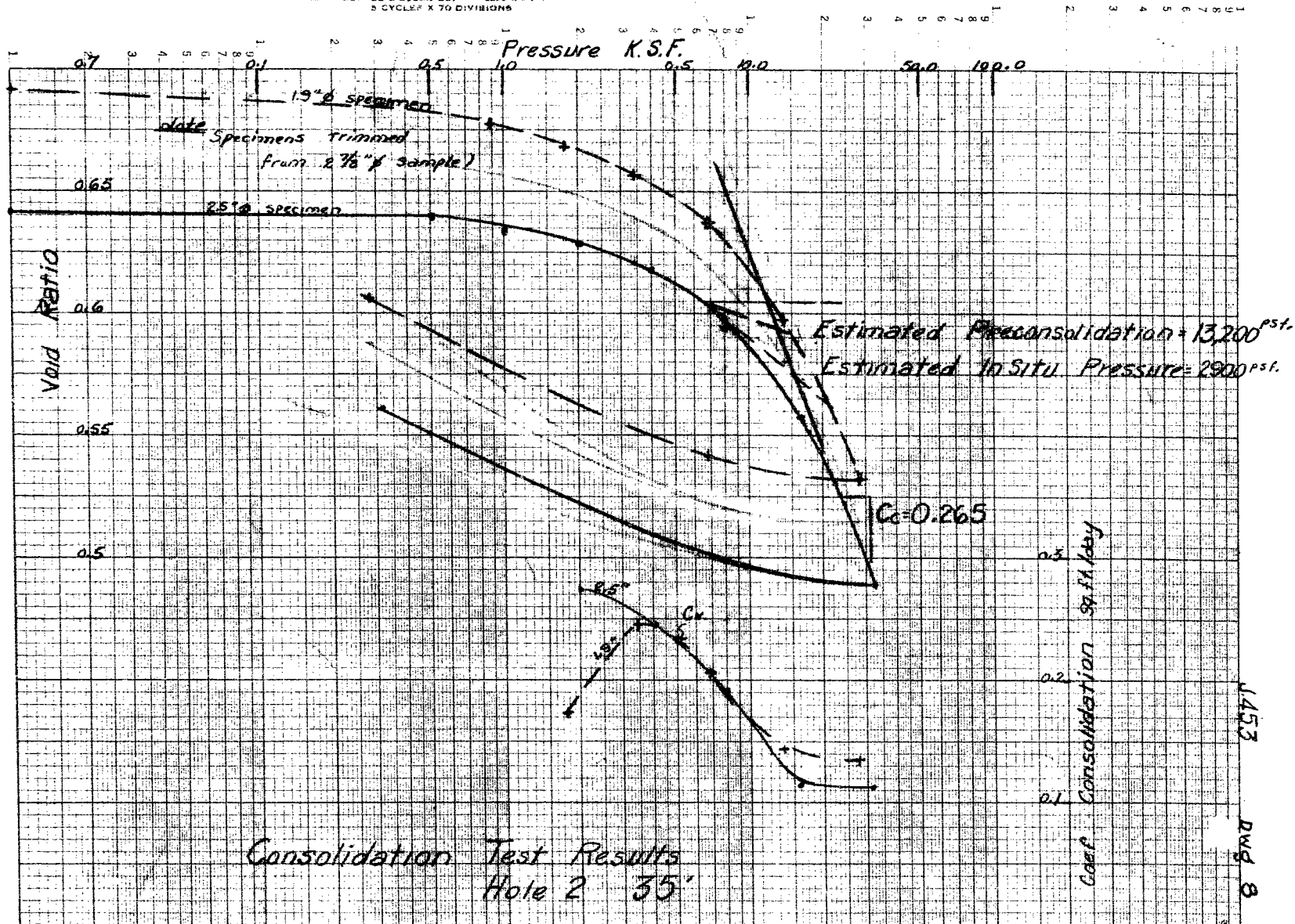


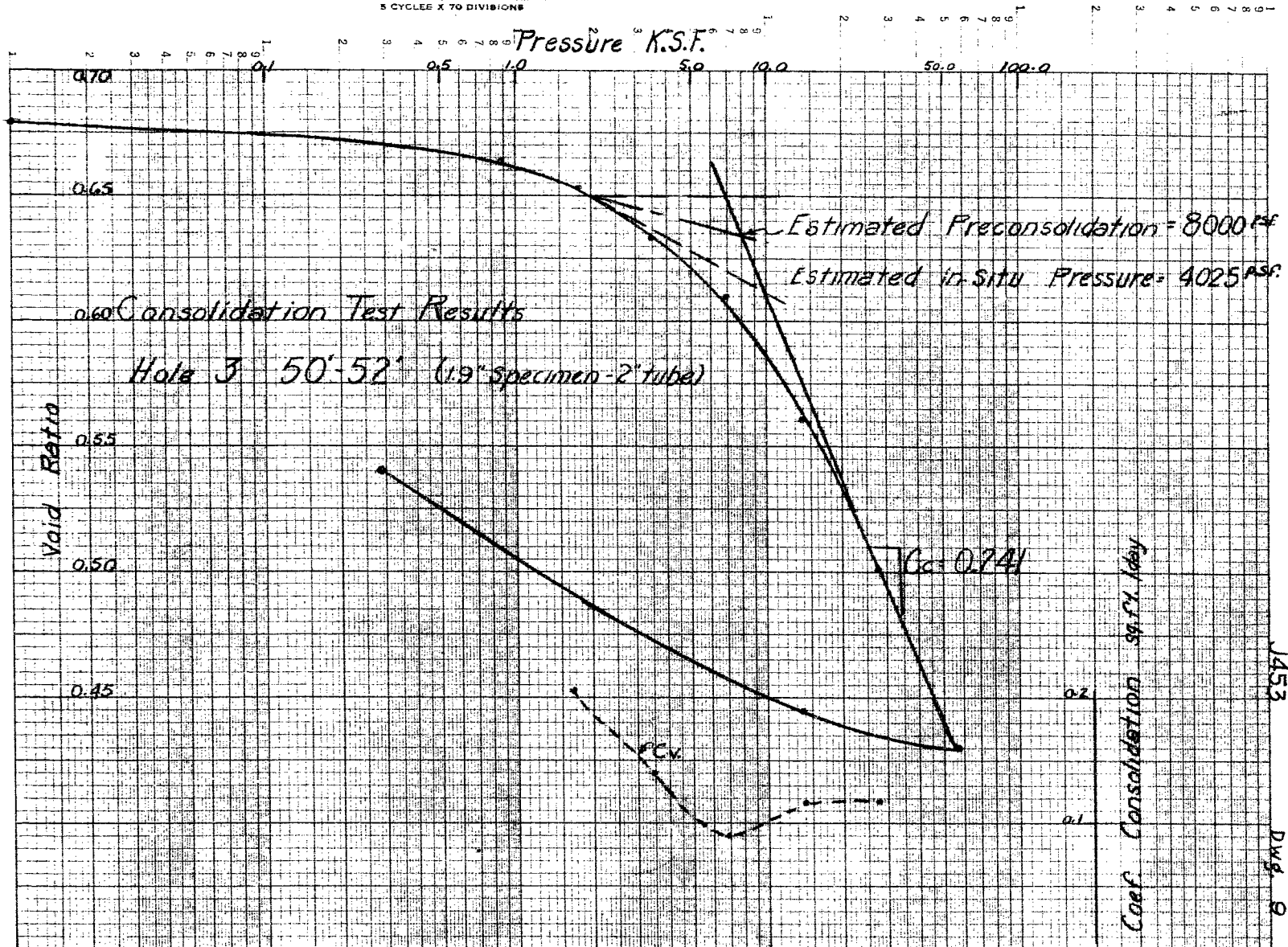
MODIFIED M.I.T. CLASSIFICATION

GRAIN SIZE DISTRIBUTION Hole 2, 34 - 35 Feet.

WILLIAM A. TROW AND ASSOCIATES

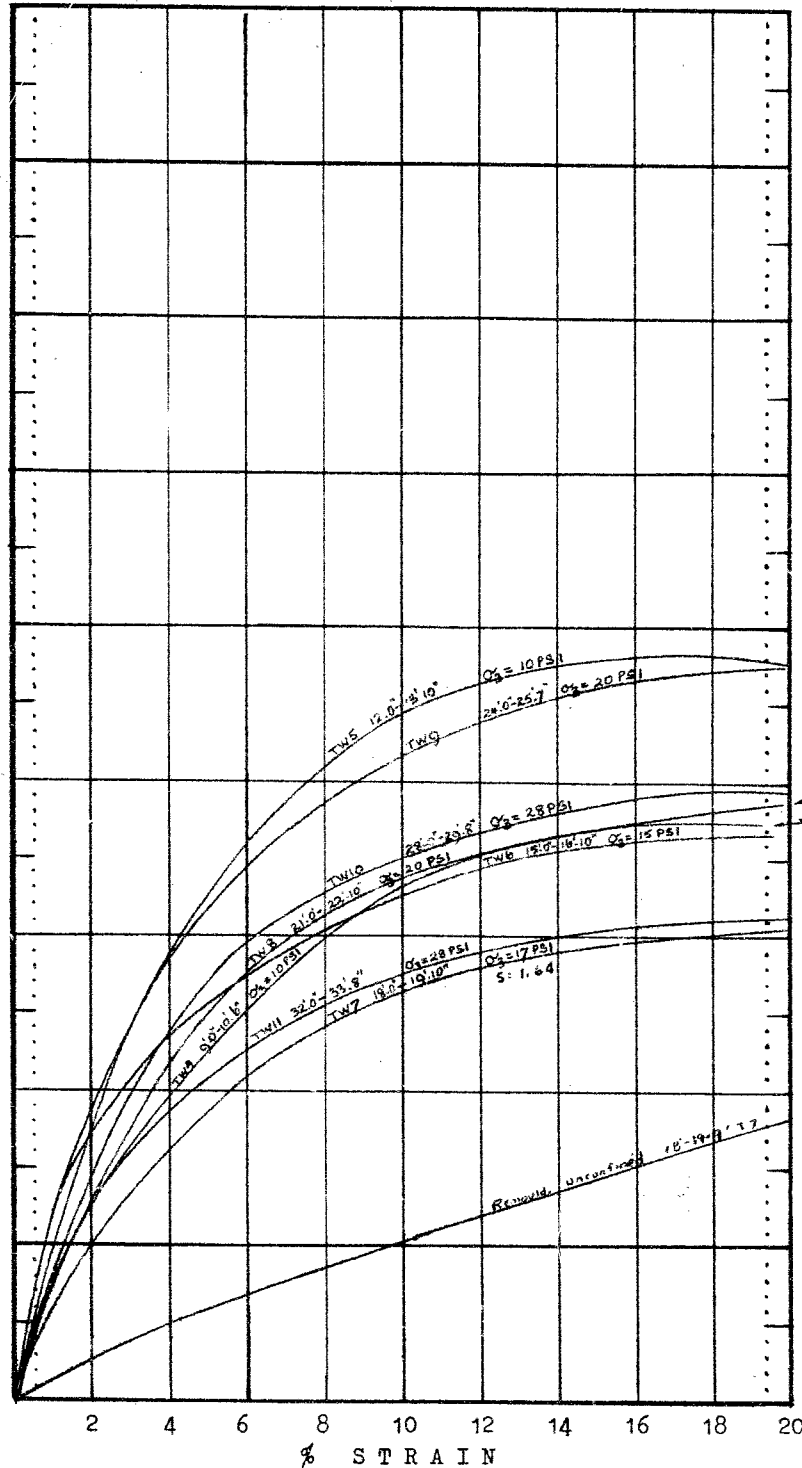






BOREHOLE 1

SHEAR STRESS ksf

2.5
2.0
1.5
1.0
0.5

BOREHOLE 1

SHEAR STRESS ksf

2.5

2.0

1.5

1.0

0.5

% STRAIN

2

4

6

8

10

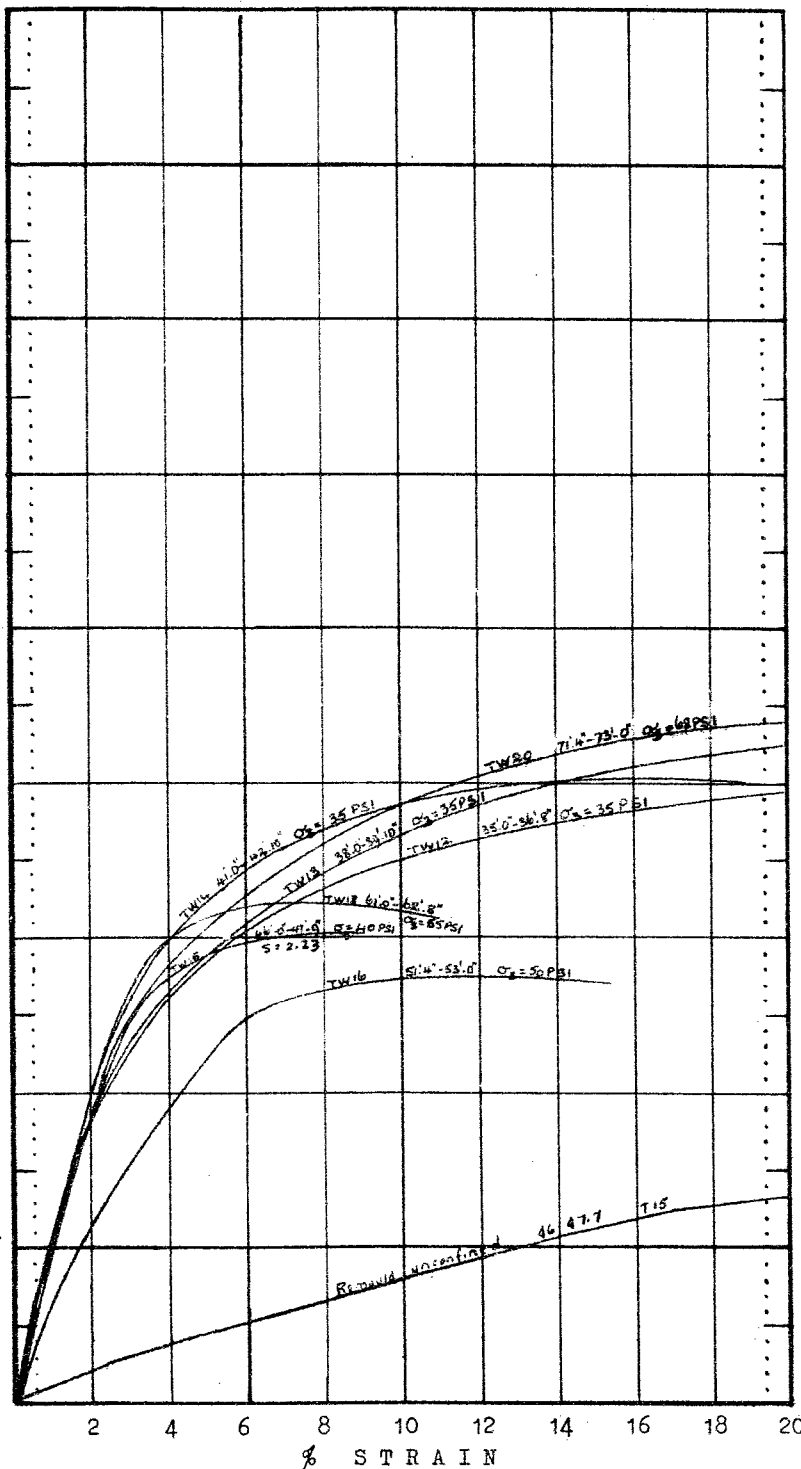
12

14

16

18

20



BOREHOLE 3

SHEAR STRESS kef

2.5

2.0

1.5

1.0

0.5

% STRAIN

2

4

6

8

10

12

14

16

18

20

