

#66 - F-280 M

CTY. RD. #29

BRIDGE , CON. 11-12

NORTH WALSINGHAM TWP.

CON. 9-10

CHARLOTTEVILLE TWP.

BA 2289



NORFOLK COUNTY ROAD SYSTEM  
P.O. BOX 601  
SIMCOE, ONTARIO

FOUNDATION CONDITIONS  
PROPOSED BRIDGE REPLACEMENT  
AND ROAD IMPROVEMENT  
COUNTY ROAD NO. 29 - CON. 11-12  
TOWNSHIP OF NORTH WALSHINGHAM, CON. 9-10  
TOWNSHIP OF CHARLOTTEVILLE

57E 20-142

Project: J2823

March, 1966

William Trow Associates Limited

90 Milvan Drive  
Weston, Ontario  
749-1290

William Trow

Project: J2823

Soil Mechanics  
Consultants  
W. A. Trow  
MSc. MEIC. P. Eng.  
K. Peaker  
PhD. MEIC. P. Eng.  
D. H. Shields  
PhD. MEIC. P. Eng.



Associates Ltd.

Norfolk County Road System,  
P.O. Box 601,  
Simcoe, Ontario.

March 7, 1966

Attention: Mr. W.C. McDowell, P.Eng.  
County Engineer.

Foundation Conditions  
Proposed Bridge Replacement  
And Road Improvement  
County Road No. 29 - Con. 11-12  
Township of North Walsingham, Con. 9-10  
Township of Charlotteville

Dear Sirs:

In conformance with your authorization given in mid-February we have made several borings at the site of the proposed replacement for the above noted bridge, and in the valley area to the south where fill to a maximum height of 40 feet is proposed.

The location of the site south of Delhi, Ontario, and the scope of the proposed construction, are indicated on Dwg. 1. Briefly it is proposed to replace the existing narrow concrete bridge by a wider and longer 3 span structure with an embankment approach leading to it from the north, reaching a height of 25 feet approximately. The hill immediately to the south will be cut down and used for the high fill in the adjacent valley to the



south. The present bend in the river in this south part of the project will be cut off by the embankment. The river will be diverted through a channel cut to the west of the site.

Preliminary advice on foundation conditions for this project was given while the field work was in progress. Because the soils information is urgently required and since foundation alternatives are somewhat limited we shall be quite brief in order to expedite the submission of our final report to you. Our comments and recommendations are given in the paragraphs that follow.

#### SUBSOIL

The soil encountered at the site is indicated in the 9 borehole logs for this project and in summarized form in the stratigraphical profile of Dwg. 1. In the flood plain area of the bridge and adjacent valley, requiring high embankment fill, (borings 1 to 4 and 8 and 9), alluvial sand was encountered to a maximum depth of about 30 feet below river level. This sand contains some wood fibres, as well as some decayed wood and peat at deeper levels, but there is insufficient of these



materials to be of concern. A layer of stiff stratified clay was encountered in Hole 1 at the north end of the site and in the hillside in Hole 4 to the south. It has been eroded away in between. Farther to the south the clay begins about 23 feet below the surface. This stratum is generally stiff in consistency with a shear strength in the order of 1400 psf although it gets stiffer and stronger at upper levels. The clay is underlain by dense fine sand which contains some thin clay seams at lower levels. Clean fine brown sand comprises the existing embankment fill in the north approach to the bridge.

#### FOUNDATIONS

In view of the loose condition and great depth of the alluvial sand, it is recommended that the new bridge be founded on timber piles, driven to the dense sand about 30 feet below river level. It is possible that refusal will not be encountered for some piles. However, because clay seams exist in the low levels of the sand, it is recommended that piles be terminated at El 600 feet. The working load per pile at this level is estimated to be 25 tons. The calculations to support this are given in the Appendix.



It is possible that the south abutment could be founded on the top of the very stiff clay which begins below El 637 feet or 7 feet above river level in Hole 4. The safe net bearing value to apply at El 637 feet is 5500 psf. If the footing level is lowered, a lower bearing pressure must be applied.

#### EMBANKMENT STABILITY

Although embankments will reach a maximum height of 40 feet there is no embankment stability problem at this site. Calculations in the Appendix indicates that the limiting maximum safe height for an embankment, which will not overstress the clay is 46 feet. Side slopes of 2:1 and a factor of safety of 1.4 are assumed in this computation.

Because the clay thickness is not great, it is bounded by sand at both upper and lower boundaries, and it contains silt seams; any settlement will be complete soon after construction. Considering also that the clay was originally consolidated under the weight of soil up to the original surrounding countryside before the river eroded the ground to present levels the amount of settlement will be small. There is no need to remove the fill in the north approach to the existing bridge; it is fine and clean.



Although borings 8 and 9 were not made in the area of the highest fill in the south flood plain area they are believed to be representative for the entire valley. They are certainly similar to the results obtained in the borings for the bridge site about 2000 feet to the north.

#### EARTH PLACEMENT

Typical gradings of the soil in the mid-hill section of the road, between Sta. 46 + 00 and 37 + 00, which will be cut down and used as fill for the embankments to the south, are shown on Dwg. 17. Because this soil is quite silty and stratified and it is moist to wet, it will be difficult to excavate and place during wet or poor drying weather. Work, therefore, should be scheduled for mid-summer and the excavated silt should be laid out in long thin lifts prior to compaction to aid in drying. A typical Proctor curve for the sand, and in-place moisture contents for it are presented in the report.

It is of interest to learn that excavation equipment became mired in the wet sand at the south end of the original bridge during its construction. In order to minimize this danger



in present construction it is suggested that a perimeter drainage ditch be dug into the slope around the south end of the south abutment excavation at least a foot below footing level, well in advance of actual construction.

It is recommended as well that ditches be installed to a depth of 3 feet below road subgrade level in the area of cut to ensure drainage and minimize the effects of frost heaving. If desired, four inch Hel Cor pipe surrounded by 6 inches of pea gravel and topped with concrete sand can be used in the ditch.

#### EROSION PROTECTION

Presumably, with three span construction, the abutments will be of the spill-through type, i.e., the fill will not be retained. With this arrangement, simple rip-rap, bedded on a 12 inch layer of pit-run gravel and carried up to high water level, should suffice to prevent erosion.

If closed abutments are envisaged steel sheet piling driven about 10 feet below river bed level should be used. The top end should be tied into the abutment.



We hope that the foregoing comments assist you in the construction of the proposed bridge replacement and road improvement.

If you have any queries after your review of this report please do not hesitate to contact us.

Yours very truly,

  
William A. Trow, P.Eng.

WAT/gh  
Encls.

Dist: -Norfolk County Road System (6)



## APPENDIX

### BEARING CAPACITY OF TIMBER PILES

Assume a pile of 8 inch tip and average diameter of 10 inches. The safe capacity is given by the expression:

$$Q = \frac{1}{F} \gamma D N A$$

where:  $\gamma = 65$  is the submerged unit weight of the soil

$D$  is the depth in feet below the surface;  
conservatively taken from river bed level  
to El 600 feet to be 25 feet.

$N = 300$  is a bearing capacity factor considered  
to be appropriate for the depth and sand  
density applicable.

$F = 3$  is the recommended factor of safety.

$A$  is the area in square feet of the pile  
tip.

Solving  $Q = 25$  tons or more - Use 25 tons.

No allowance is made for adhesion or friction generated on the  
side of the pile and therefore this computation is conservative.

### BEARING CAPACITY OF SOUTH ABUTMENT

Set the abutment on the top of the very stiff clay at  
El 637 feet approximate, ( if the abutment is at the Hole 4  
position).

$$\text{Safe bearing value } q = \frac{CN}{F} + \gamma D$$



where:  $C = 26000$  psf the undrained shear strength of the soil just below bearing level.

$N = 6$  is the bearing capacity factor for this application

$F = 3$  is the required safety factor

$\gamma D$  is the minimum surcharge pressure above footing level.

Solving  $q = 5200 + 125D = 125D$ , conservatively take  $q = 5500$  psf.

#### EMBANKMENT STABILITY

The maximum permissible height of embankment fill can be estimated from the expression:

$$h = \frac{CN}{F\gamma}$$

where:  $C$  is the undrained shear strength of the soil  
 $= 1400$  psf for this situation.

$N$  is the bearing capacity factor equal at least 6 for the limited thickness of clay, here, the depth of the clay below the surface and for side slopes of 2 horizontal to 1 vertical.

$F = 1.4$  is the limiting factor of safety to prevent failure.

$\gamma = 130$  pcf the compacted weight of the silty fill proposed for embankment fill.

Solving  $h = 4.6$  feet.

LEGEND

1

## Proposed Bridge Replacement & Road Improvement

County Road 29

Sta. 42 + 43

640.5 feet.

See Dwg. 1

DEPARTAMENTO DE HISTORIA

Figure 1. Schematic representation of the experimental design. The subjects were divided into two groups: a control group and an experimental group. The control group received a standard training program, while the experimental group received a training program with a focus on the specific skills required for the task. The results of the training program were compared between the two groups.

$\Delta_1 = \Delta_2 = 0.54163$   
 $\Delta_3 = \Delta_4 = 0.54163$

#### WEAR STRENGTH

THE UNIVERSITY OF CHICAGO

A = {0, 1, ..., 9} B = {0, 1, ..., 9} C = {0, 1, ..., 9}

[illegible]

NATURAL MOISTURE CONTENT  
AND LOG DRY INDEX

## ATTERBERG LIMITS

### FIELD LIMIT







PLASTIC LIMB

SAMPLE TYPE

2 OF 5 PAGES

2 10 SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE 150 FT. LB BELOW 5 FT BD		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT			SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
				LC	SC	10	20	30		
	EMBANKMENT FILL-fine to medium sand dense to compact, clean, some gravel.	640.5	0							
	ALLUVIAL SAND-silty, loose, relatively clean, some wood fibres at upper levels and peat pockets and organic staining below 15 feet; some thin layers of clayey silt.	634	10							
	-sand with gravel sizes 19½-20½ feet.	620	20							
	CLAY-silty, stratified, very stiff, oxidized to brown colour above 27 feet, some gravel sizes, stiff, grey below.		30							
		601.5	40							
	SAND-fine, silty, grey, dense 2 to 3 inch clay seams at 46 feet approxi- mate, seams of silt.		50							
	-End of Hole	584	60							
Notes:	1) Boring cased to full depth. 2) Wet sampling methods used. 3) Hole open to 9½ feet and dry at end of bore.		70							
			80							
			90							
			100							

PENETRATION RESISTANCE  
 3.0 D. SHIELT TOOL       
 3.0 D. SHIELT TIGER       
 3.0 D. CONE       
 SHEAR STRENGTH  
 ENGINEERED TRIAXIAL  
 ACTIVE DEFORMATION PRESSURE       
 ENGINEERED COMPRESSION       
 LONG TERM AND SENSITIVITY (S)     

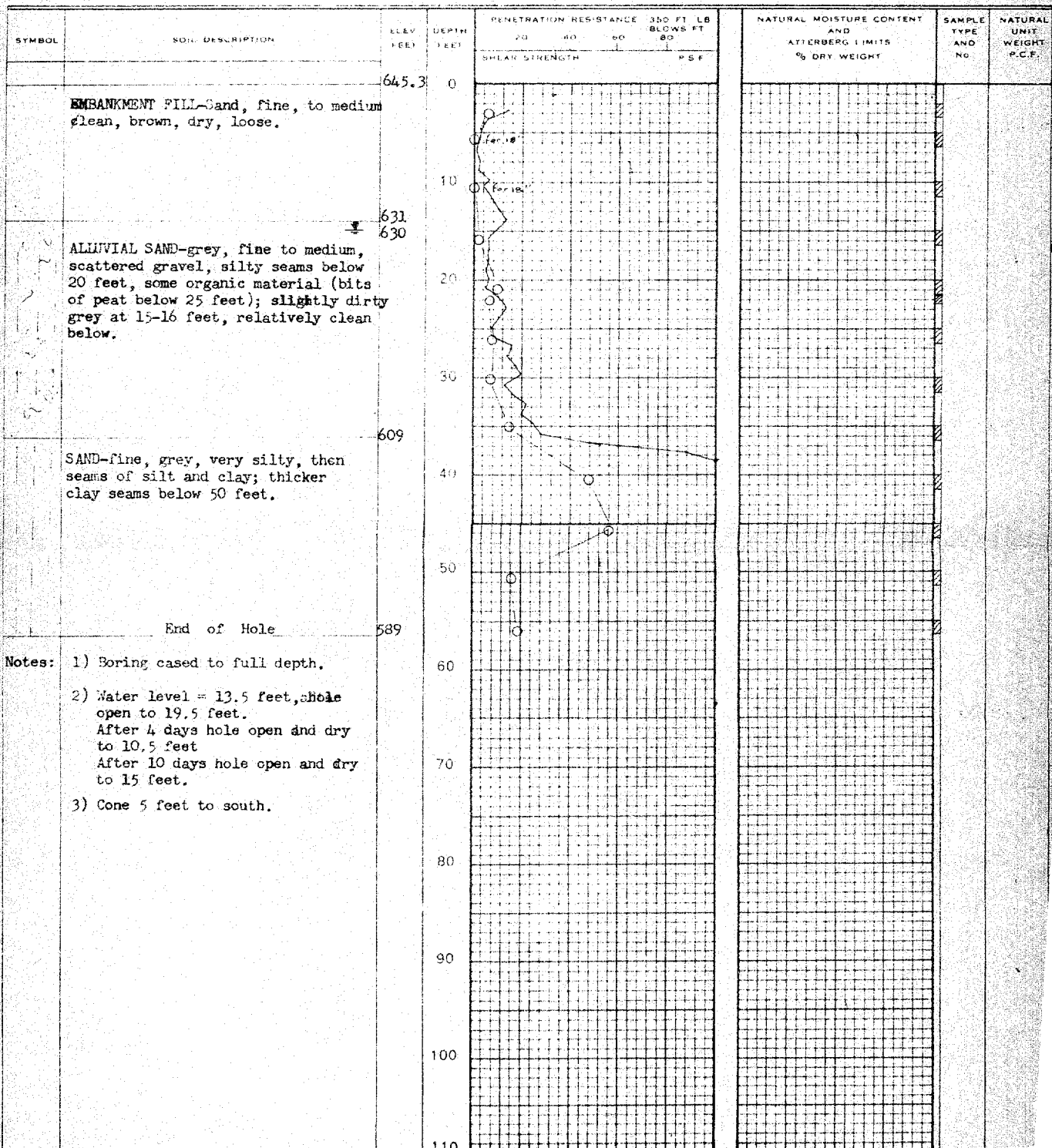
NATURAL MOISTURE CONTENT  
(AND LIQUIDITY INDEX)  
ATTERBERG LIMITS  
LIQUID LIMIT  
PLASTIC LIMIT  
SAMPLE TYPE  
2" O.D. SPLIT TUBE  
1" O.D. SHELBY TUBE  
1" O.D. SHELBY TUBE

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

BOREHOLE NO. 3  
PROJECT Proposed Bridge Replacement and Road Improvement  
LOCATION County Road 29  
HOLE LOCATION Sta. 47 + 00  
HOLE ELEVATION 645.3 feet  
DATUM See Dwg. 1

PENETRATION RESISTANCE  
2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 2" DIA. CONE   
SHEAR STRENGTH  
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE UNCONFINED COMPRESSION VANE TEST AND SENSITIVITY

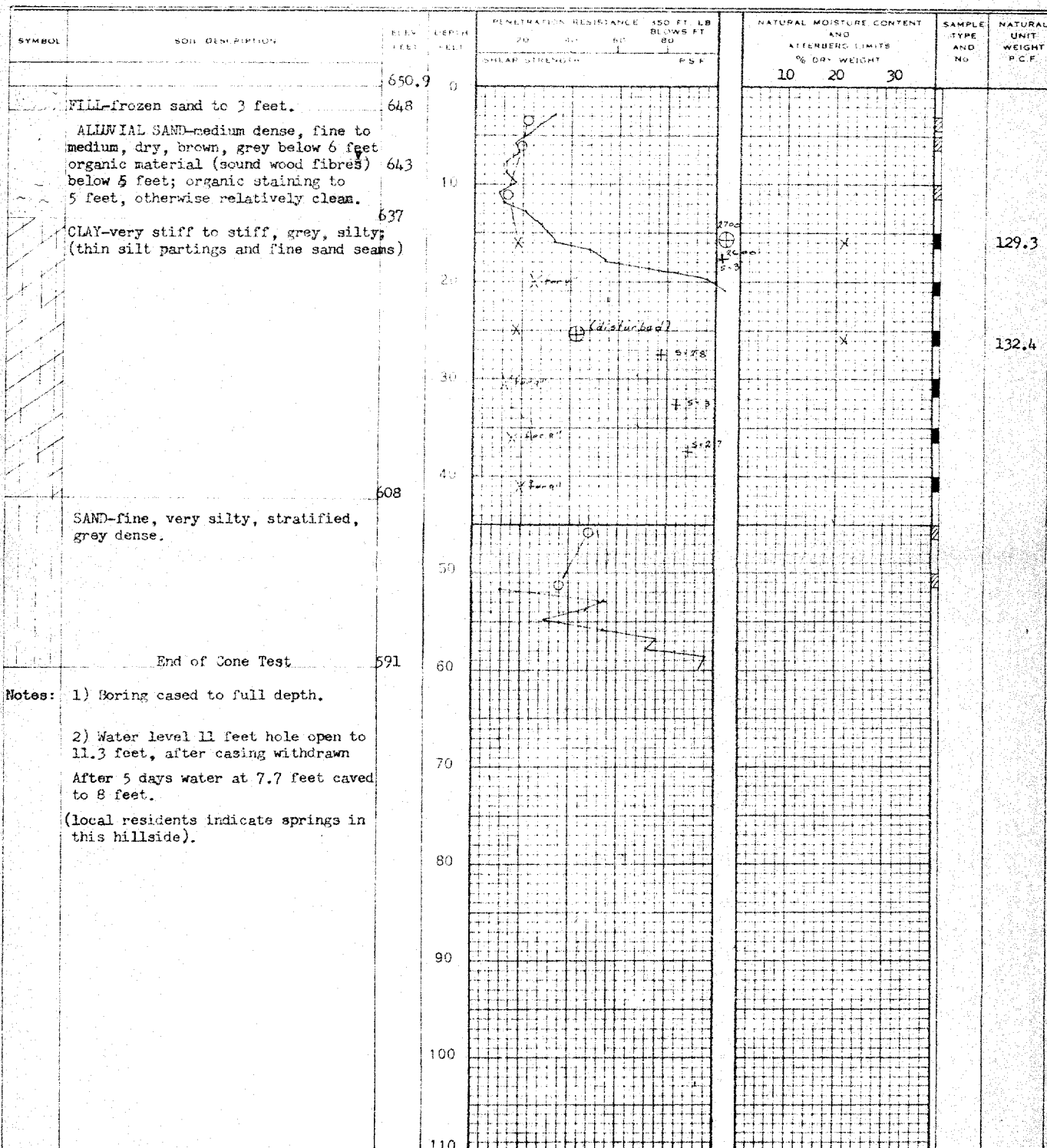
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX LIQUID LIMIT PLASTIC LIMIT SAMPLE TYPE  
2" O.D. SPLIT TUBE 2" I.D. SHELBY TUBE 2" O.D. SHELBY TUBE



BOREHOLE NO. 4  
PROJECT Proposed Bridge Replacement & Road Improvement  
LOCATION County Road 29  
HOLE LOCATION Sta. 46 + 30 - 15 feet west  
HOLE ELEVATION 650.9 feet  
DATUM See Dwg. 1

PENETRATION RESISTANCE  
2" I.D. SPLIT TUBE  $\circ-\circ-\circ$   
2" I.D. SHELBY TUBE  $\bullet-\bullet-\bullet$   
2" DIA. CONE  $\times$   
SHEAR STRENGTH  
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE  $\oplus$   
UNCONFINED COMPRESSION  $\odot$   
VANE TEST AND SENSITIVITY  $\nabla$

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX  $\times$   
ATTERBERG LIMITS  
LIQUID LIMIT  $\circ$   
PLASTIC LIMIT  $\times$   
SAMPLE TYPE  
2" I.D. SPLIT TUBE  $\square$   
2" I.D. SHELBY TUBE  $\blacksquare$   
3" I.D. SHELBY TUBE  $\blacksquare$



# WILLIAM TROW ASSOCIATES LTD.

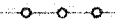
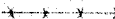

SITE INVESTIGATIONS SOIL MECHANICS CONSULTATION

## LEGEND




DRAWING No. 6  
PROJECT No. J2823

BOREHOLE NO. 5  
PROJECT Proposed Bridge Replacement & Road Improvement  
LOCATION County Road No. 29  
HOLE LOCATION Sta. 41 + 06 - 28 feet west  
HOLE ELEVATION 683 feet  
DATUM See Dwg. 1

### PENETRATION RESISTANCE


2" O.D. SPLIT TUBE   
2" I.D. SHELBY TUBE   
2" DIA. CONE 


### SHEAR STRENGTH

UNDRAINED TRIAXIAL  
AT OVERBURDEN PRESSURE   
UNCONFINED COMPRESSION   
VANE TEST AND SENSITIVITY (S) 




### NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

### ATTERBERG LIMITS

LIQUID LIMIT 

PLASTIC LIMIT 

### SAMPLE TYPE

2" O.D. SPLIT TUBE   
2" I.D. SHELBY TUBE   
3" O.D. SHELBY TUBE 

SYMBOL	SOIL DESCRIPTION	ELEV FEET	DEPTH FEET	PENETRATION RESISTANCE				350 FT. LB BLOWS/FT 80	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS			SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.		
				20	40	60	% DRY WEIGHT								
				SHEAR STRENGTH			P.S.F.								
		683	0						10	20	30				
	8 inches topsoil.														
	SILT-very fine sand to 1½ feet; then									X					
	sandy silt, brown, moist, dense at										X				
	8 feet.														
	End of Hole	672	10							X					
Notes: 1) Hole advanced in dry by driving and withdrawing. 2) Hole dry to 10 feet on completion. Same after 5 days.				20											
				30											

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT






### LEGEND

BOREHOLE NO. 7  
PROJECT Proposed Bridge Replacement & Road Improvement  
LOCATION County Road No. 29  
HOLE LOCATION Sta. 37 + 73 - 18 feet west.  
HOLE ELEVATION 659.3 feet  
DATUM See Dwg. 1

## PENETRATION RESISTANCE

2" O.D. SPLIT TUBE 2 10. SHELBY TUBE ~~-----~~

2 DIA. CONE

### SHEAR STRENGTH

UNDRAINED TRIAXIAL  
AT OVERBURDEN PRESSURE

UNCONFINED COMPRESSION 

VANE TEST AND SENSITIVITY IS +

### NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX

XL

## ATTERBERG LIMITS

LIQUID LIMIT \_\_\_\_\_

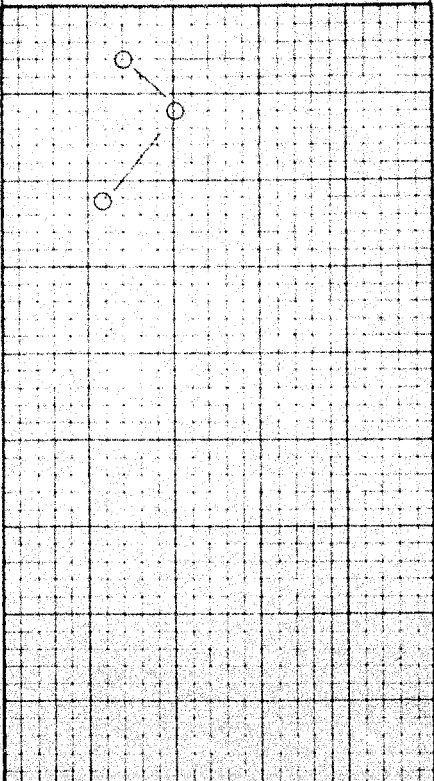
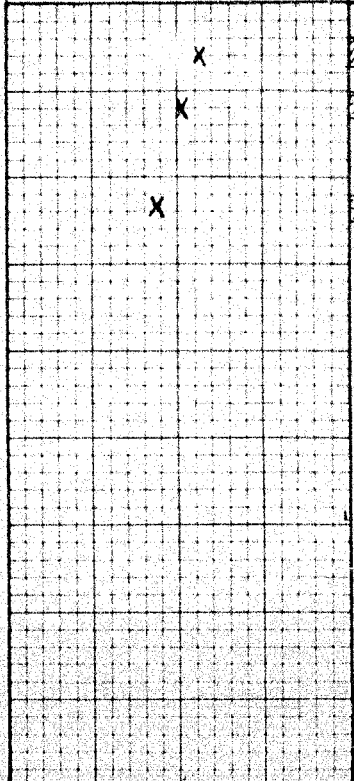
PLASTIC LIMIT \_\_\_\_\_

SAMPLE TYPE

2" O.D. SPLIT TUBE

2 10 SHELBY TUBE

3" O.D. SHELBY TUBE \_\_\_\_\_

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE		350 FT. LB	NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.			
				20	40	60				80		
				SHEAR STRENGTH						PSI		
		659.3	0					10	20	30		
	5 inches road gravel SAND—dense, fine to medium, brown, silty below 3 feet, wet below 4 feet grey below 10 feet.	654	10									
	End of Hole	647	20									
			30									
			40									

Notes: 1) Hole advanced in dry by driving and withdrawing.  
2) Hole open and dry to 7 feet at end of bore. Wet at 5.3 feet; and caved at this depth after 4 days.

## LEGEND

## PENETRATION RESISTANCE

2" O.D. SPLIT TUBE —○—○—○—  
 2" I.D. SHELBY TUBE —+—+—+—  
 2" DIA. CONE ————

## SHEAR STRENGTH

UNDRAINED TRIAXIAL  
 AT OVERBURDEN PRESSURE ⊙  
 UNCONFINED COMPRESSION ⊗  
 VANE TEST AND SENSITIVITY IS: †

NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX

## ATTERBERG LIMITS

LIQUID LIMIT —○—

PLASTIC LIMIT ———

## SAMPLE TYPE

2" O.D. SPLIT TUBE ——— 2  
 2" I.D. SHELBY TUBE ——— 3  
 2" O.D. SHELBY TUBE ——— 4

BOREHOLE NO. 8

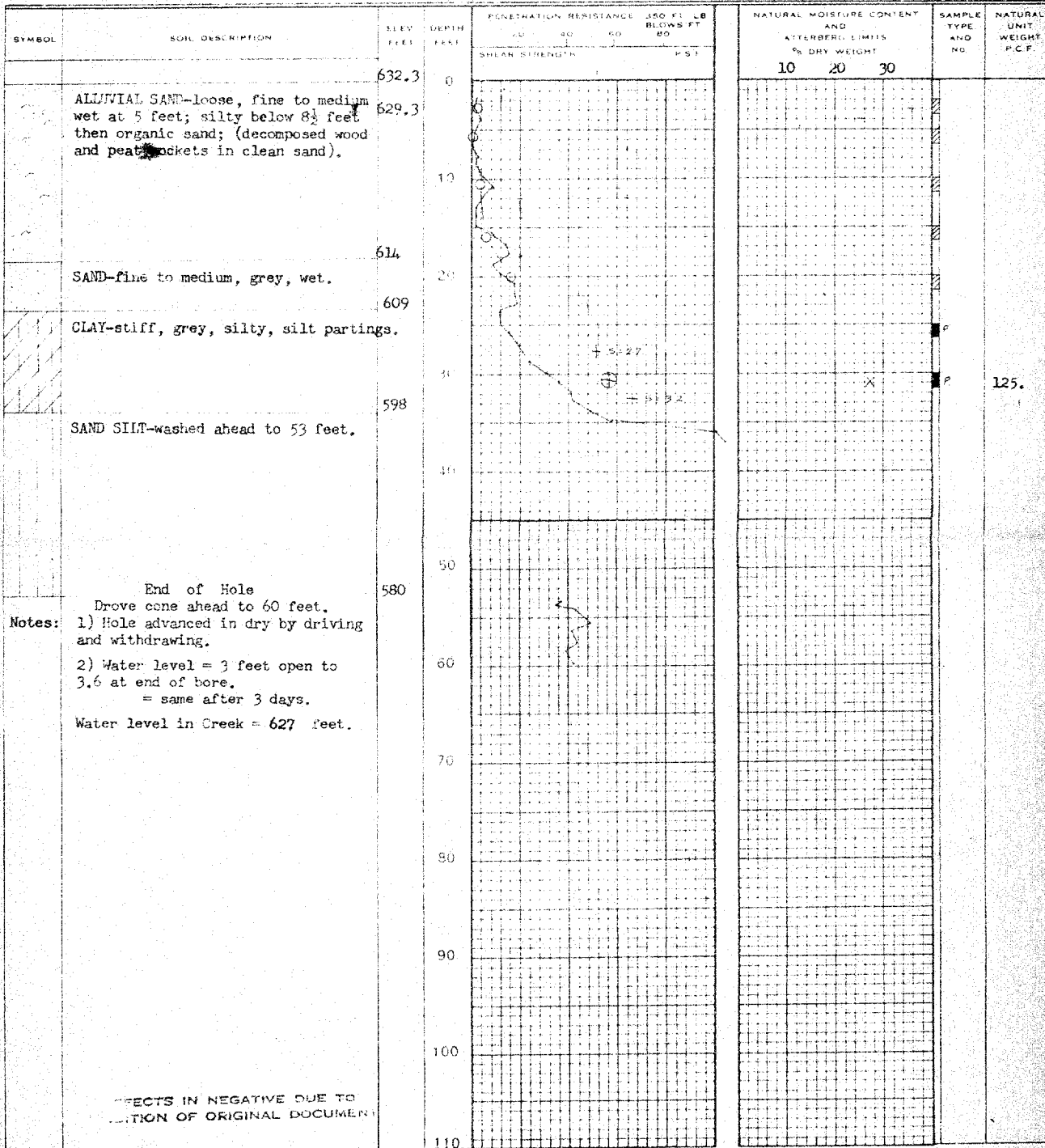
PROJECT Proposed Bridge Replacement &amp; Road Improvement

LOCATION County Road No. 29

HOLE LOCATION 10 feet left Sta. 29 + 92

HOLE ELEVATION 632.3 feet

DATUM See Dwg. 1



BOREHOLE NO. 9  
PROJECT Proposed Bridge Replacement & Road Improvement  
LOCATION County Road No. 29  
HOLE LOCATION Sta. 28 + 06 ~ 5 feet right  
HOLE ELEVATION 633.3 feet  
DATUM See Dwg. 1

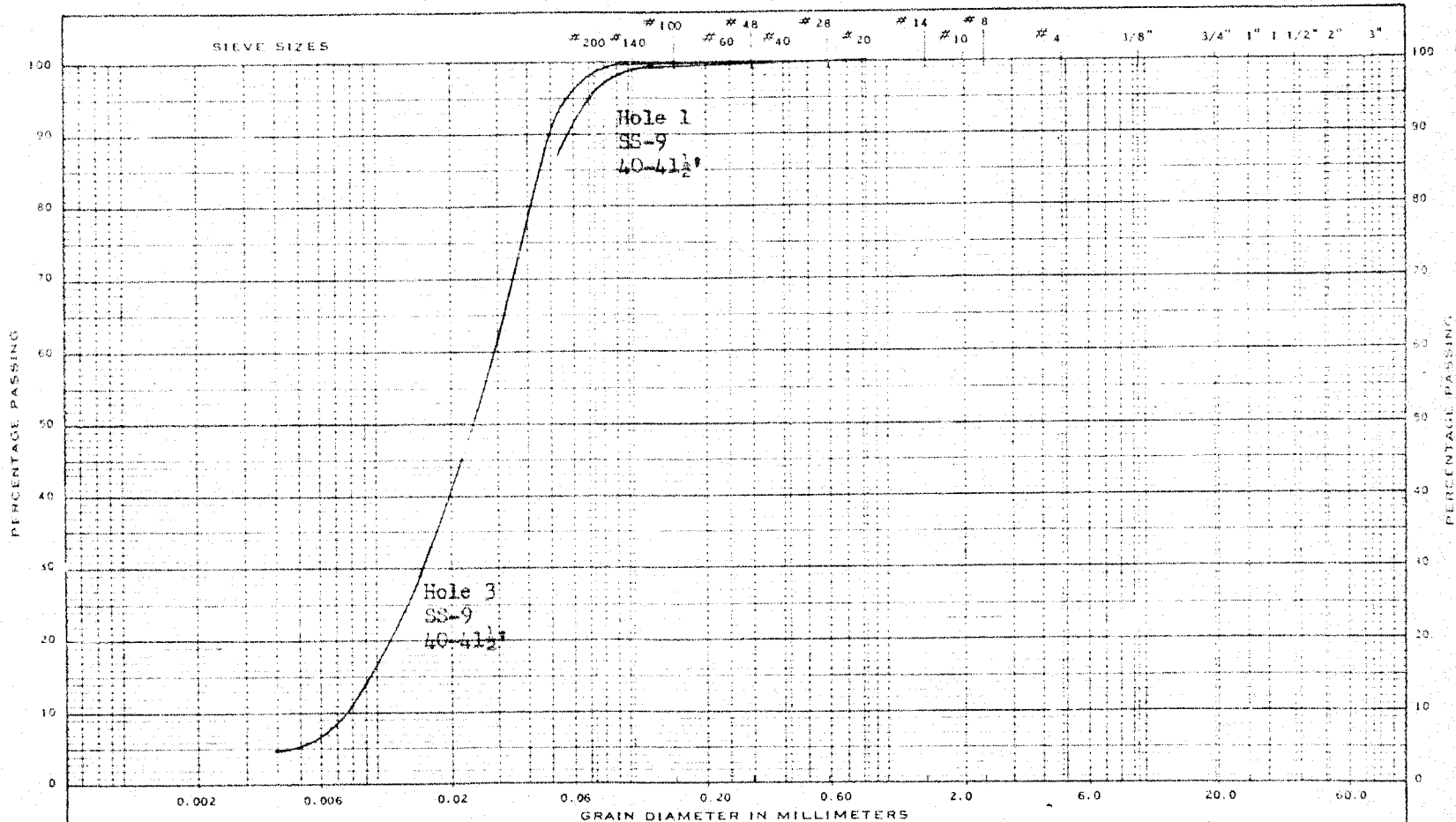
## PENETRATION RESISTANCE

1. S.D. SPIKE TUBE —○—○—○—  
2. S.D. SHELBY TUBE —+—+—+—  
3. S.D. CONE —●—●—●—  
4. SHEAR STRENGTH  
5. UNPAID TUBE  
6. AT CUMBERLAND PRESSURE  
7. UNPAID TUBE  
8. CASE TEST AND SENSITIVITY

NATURAL MOISTURE CONTENT  
AND LIQUIDITY INDEX  
ATTERBERG LIMITS  
LIQUID LIMIT  
PLASTIC LIMIT  
SAMPLE TYPE  
2. S.D. SPIKE TUBE  
2. S.D. SHELBY TUBE  
3. S.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBERG LIMITS % DRY WEIGHT	SAMPLE TYPE AND NO.	NATURAL UNIT WEIGHT P.C.F.
				20	40			
		633.3	0					
	ALLUVIAL SAND—brown, fine to medium, seams of silt below 5 feet, loose, grey below 9 feet, organic material (peat and decomposed wood) in sand from 10 to 20 feet; some rootlets at 20 feet.	627.3	10					
		610	20					
	SAND—fine to medium, grey, compact, (well graded fine to coarse sand and gravel below 5 feet).	600	30					
	CLAY—stiff, grey, silty.	595	40					
	SAND—fine, silty, grey, dense, thin clay seams and silt seams.		50					
	End of Hole	582	60					
Notes: 1) Boring cased to full depth.								
2) Water level = 6 feet open to 6.6 feet at end of bore.								
			70					
			80					
			90					
			100					
			110					

# MECHANICAL ANALYSIS



CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	
	SILT			SAND			GRAVEL			
MODIFIED M.I.T. CLASSIFICATION						WILLIAM A. TROW AND ASSOCIATES LTD.				
GRADING OF SANDY SILT AT APPROXIMATE FOUNDING LEVEL FOR TIMBER PILES HOLES 1 AND 3										

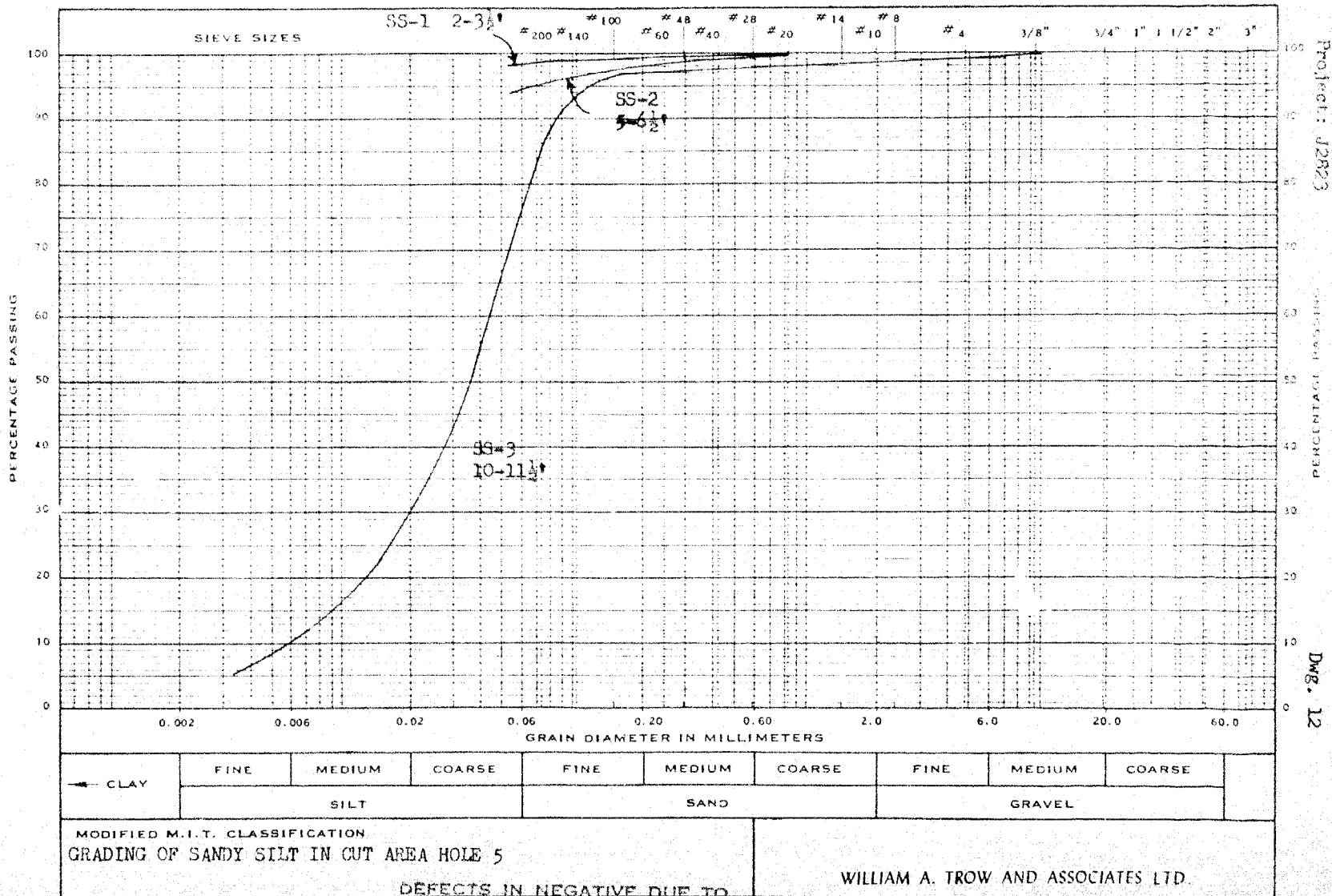
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CONDITION OF ORIGINAL DOCUMENT

Project: J2823

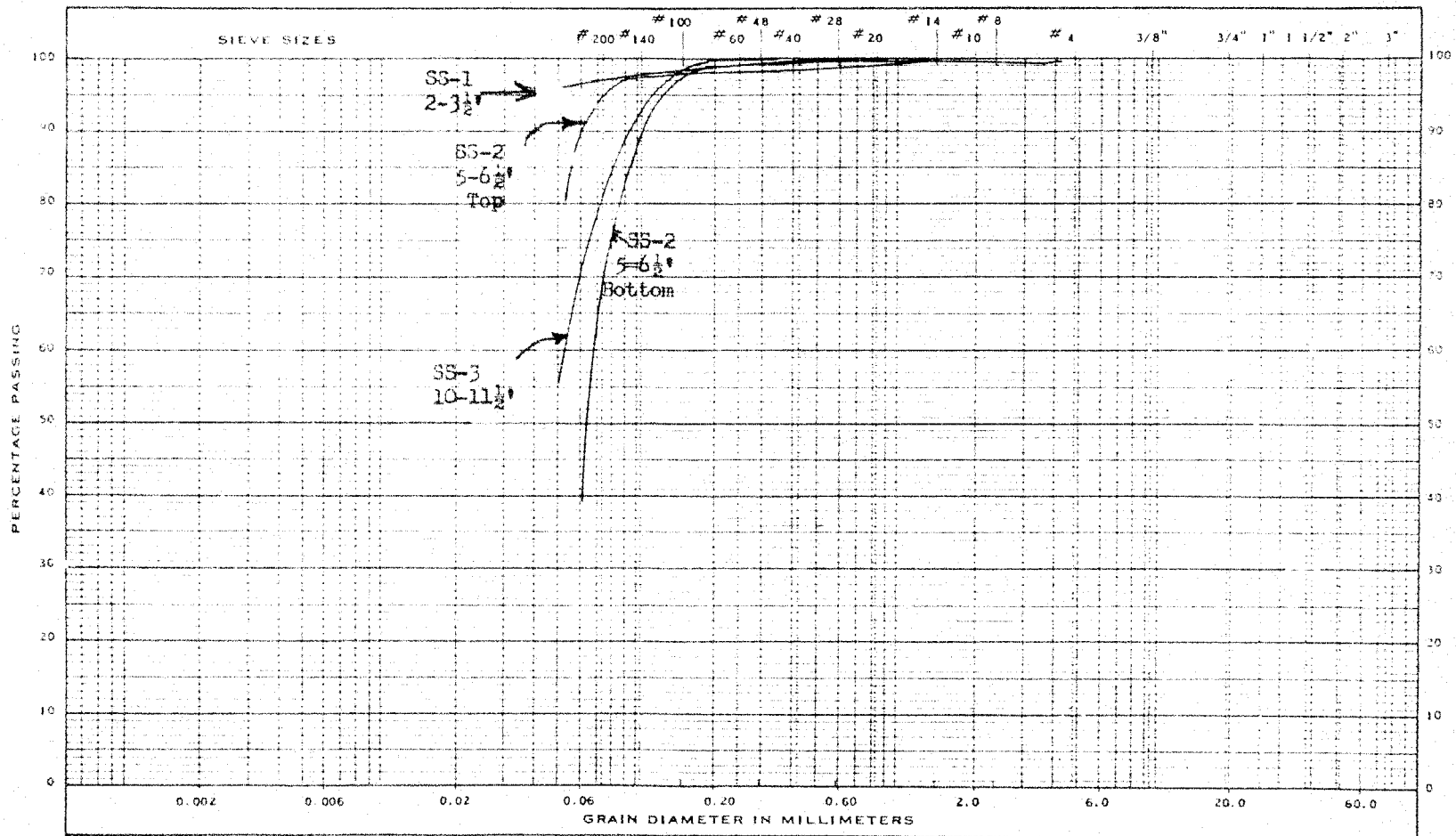
UNISSUED ENGINEERED

DWG. 11

# MECHANICAL ANALYSIS

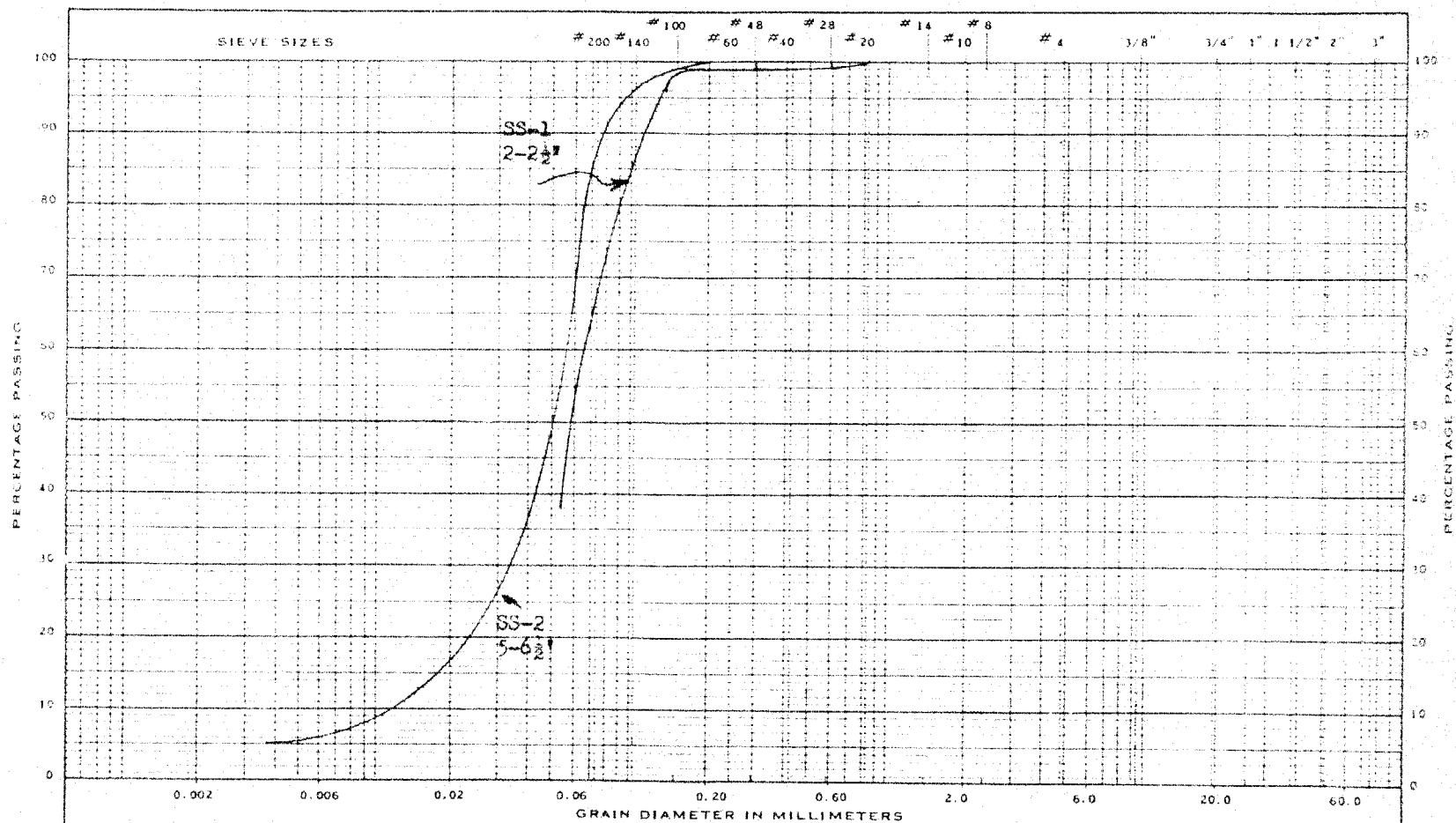


# MECHANICAL ANALYSIS



← CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	
	SILT			SAND			GRAVEL			
MODIFIED M.I.T. CLASSIFICATION GRADING OF SANDY SILT IN CUT AREA HOLE 6						WILLIAM A. TROW AND ASSOCIATES LTD.				

# MECHANICAL ANALYSIS



← CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	
	SILT			SAND			GRAVEL			

MODIFIED M.I.T. CLASSIFICATION

GRADING OF SANDY SILT IN CUT AREA HOLE 7

WILLIAM A. TROW AND ASSOCIATES LTD.

Project: J2823

DWG. 1A





TEST NR 2  
 TEST Cycled Und.  
 B.H. 1 DEPTH 36'  
 C = 1080 P.S.F.  
 $\gamma$  = 124.0 P.C.F.  
 W = 27.5 %  
 $\sigma_3$  = 31.0 P.S.I.  
 SOIL

Brownish grey silty  
 clay with clayey  
 silt intrusions.

E = 260 ksf

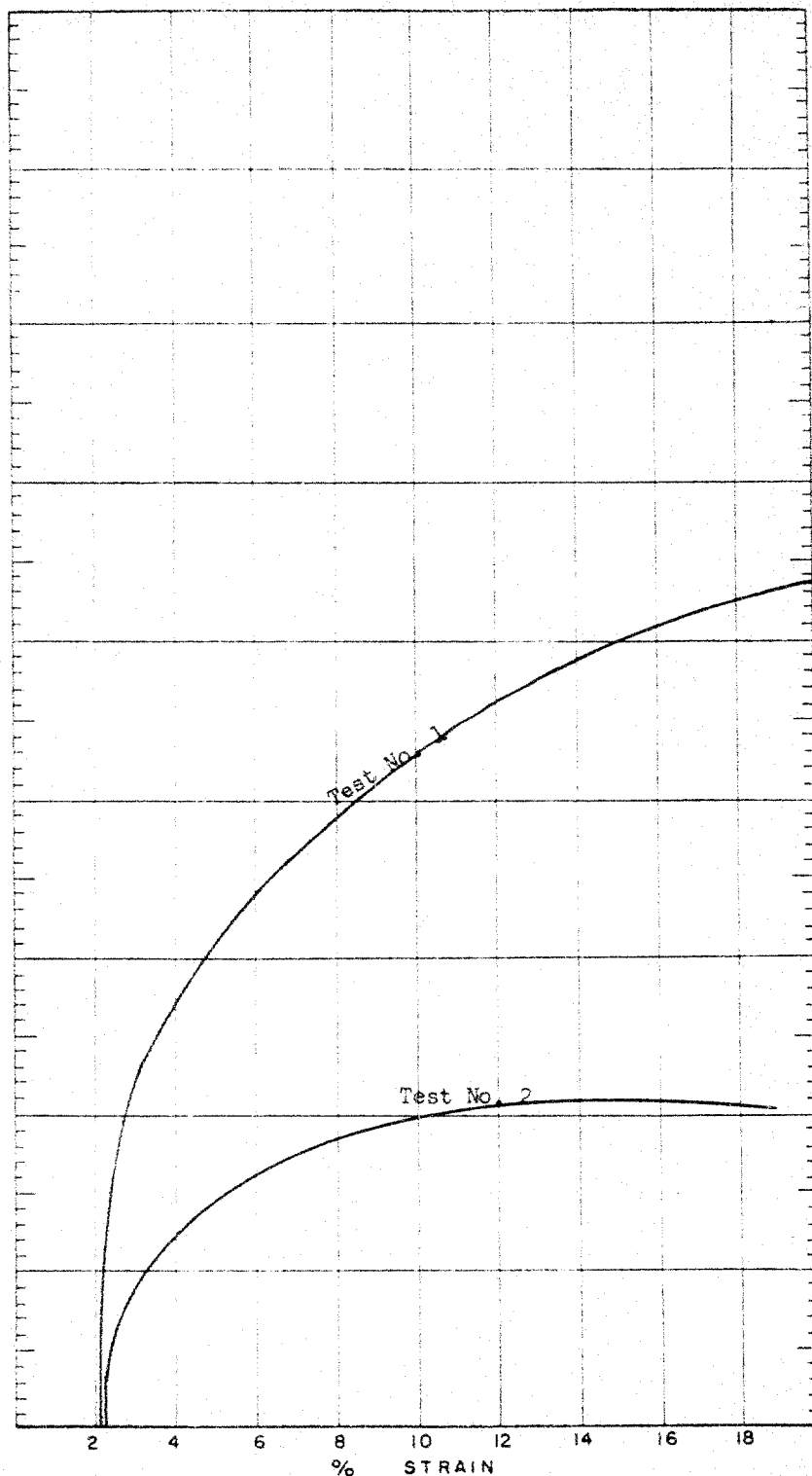
TEST NR 1  
 TEST Cycled Und.  
 B.H. 4 DEPTH 16'  
 C = 2700 P.S.F.  
 $\gamma$  = 129.3 P.C.F.  
 W = 21.1 %  
 $\sigma_3$  = 14.5 P.S.I.  
 SOIL

Brownish grey silty  
 clay with clayey  
 silt lenses.

E = 400 ksf

TEST NR  
 TEST  
 B.H. DEPTH  
 C = P.S.F.  
 $\gamma$  = P.C.F.  
 W = %  
 $\sigma_3$  = P.S.I.  
 SOIL

SHEAR STRESS ksf

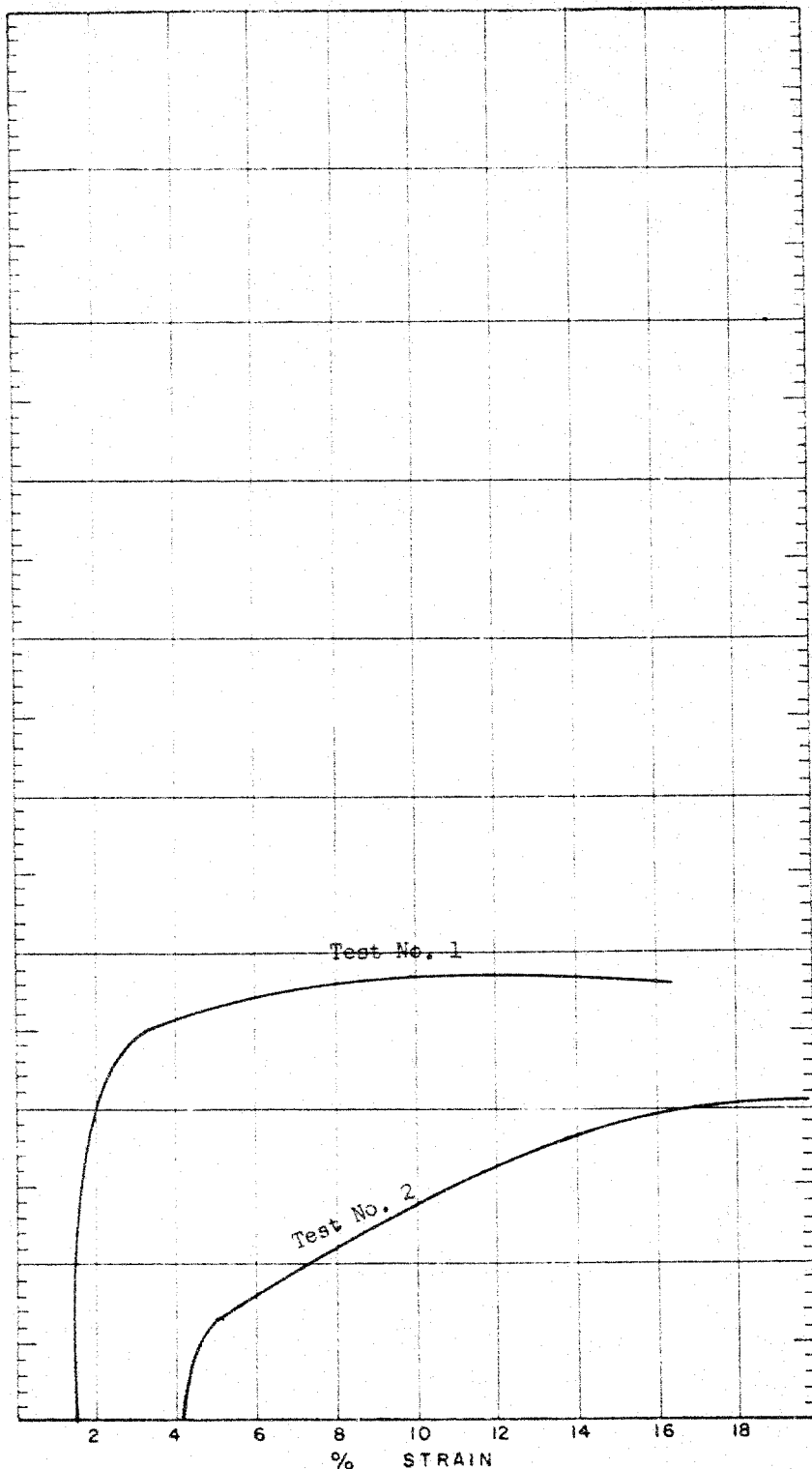


TRIAXIAL TEST RESULTS

DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT



SHEAR STRESS KSI



# TRIAXIAL TEST RESULTS

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

TEST N° 1  
TEST Cycled Und.  
B.H. 8 DEPTH 31"  
C = 1400 P.S.F.  
X = 125.0 P.C.F.  
W = 27.4 %  
G<sub>s</sub> = 27.0 P.S.I.  
SOIL

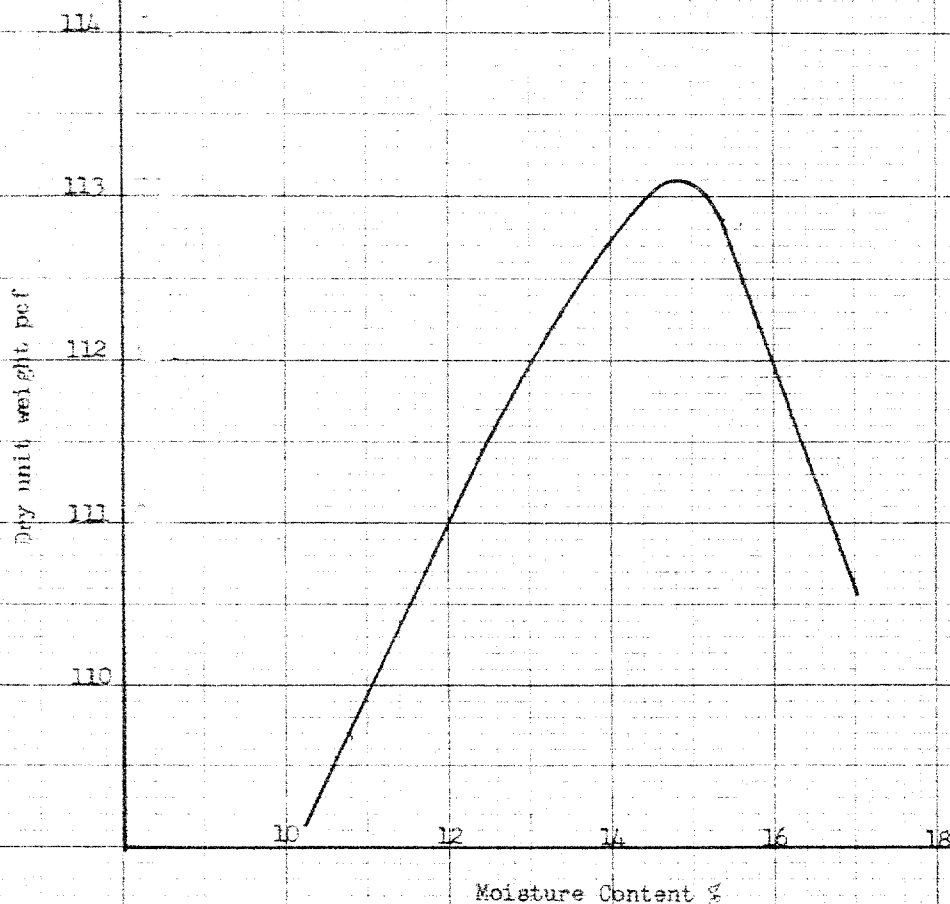
Brownish grey silty  
clay with occasional  
clayey silt lenses.

E = 130 kaf

TEST N° 2  
TEST Cycled Und.  
B.H. 4 DEPTH 26"  
C = 1050 P.S.F.  
X = 132.4 P.C.F.  
W = 21.3 %  
G<sub>s</sub> = 24.0 P.S.I.  
SOIL

Grey silty clay with  
a 1" layer of clayey  
silt. Occasional  
clayey silt  
intrusions.

TEST N°  
TEST  
B.H. DEPTH  
C = P.S.F.  
X = P.C.F.  
W = %  
G<sub>s</sub> = P.S.I.  
SOIL



STANDARD PROCTOR DENSITY TEST RESULT
(ASTM D 698 64T)

MATERIAL FROM HOLE 5 - 0 to 8 FEET