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G.I.-30 SEPT. 1976

GEOCRES No. 40P4-39

W.P. No.

CONT. No.

W. O. No.

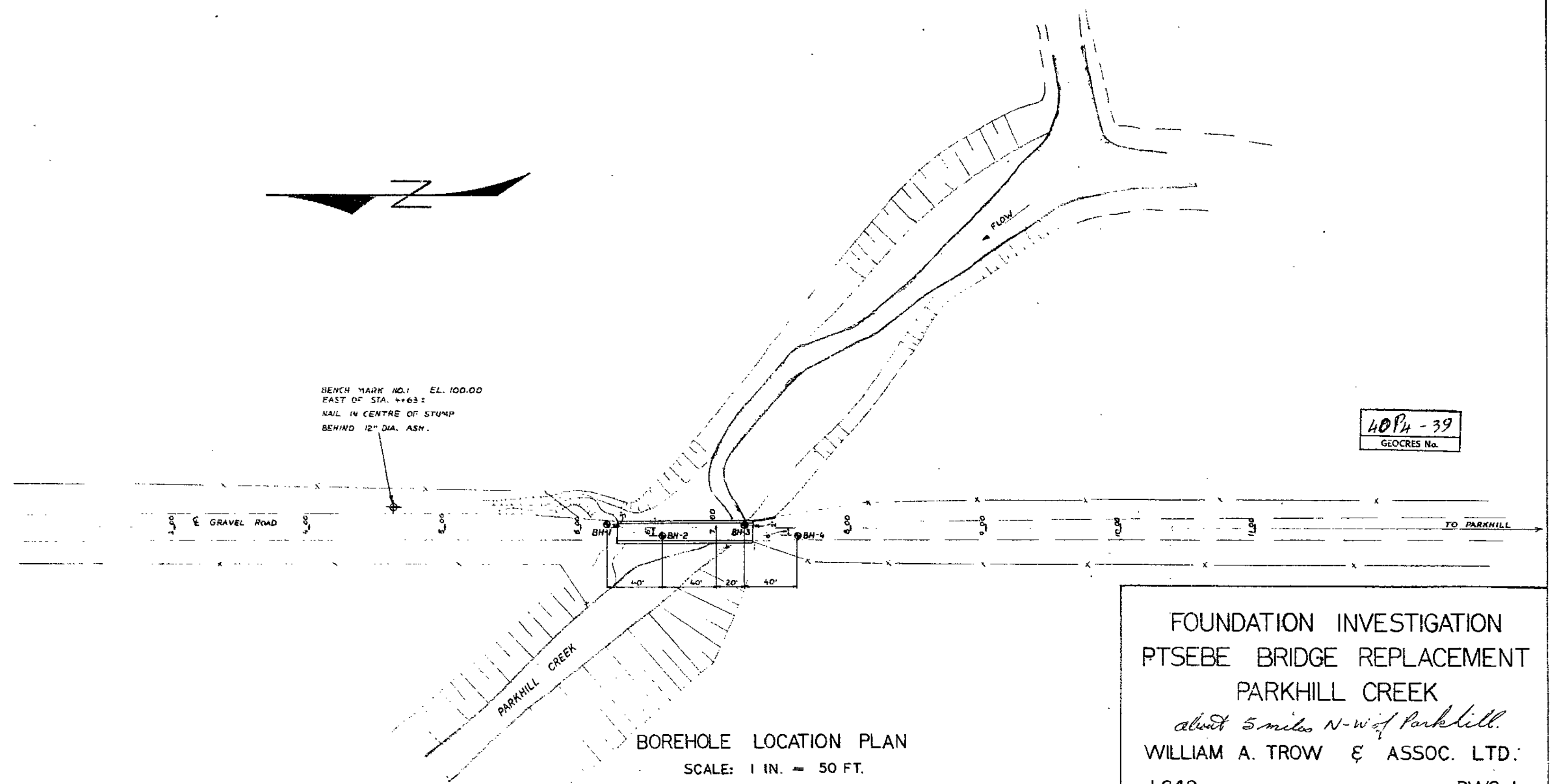
STR. SITE No.

HWY. No.

LOCATION PROP. BRIDGE REPL.
PARKHILL CREEK,
N.W. of PARKHILL

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. ONE

REMARKS:



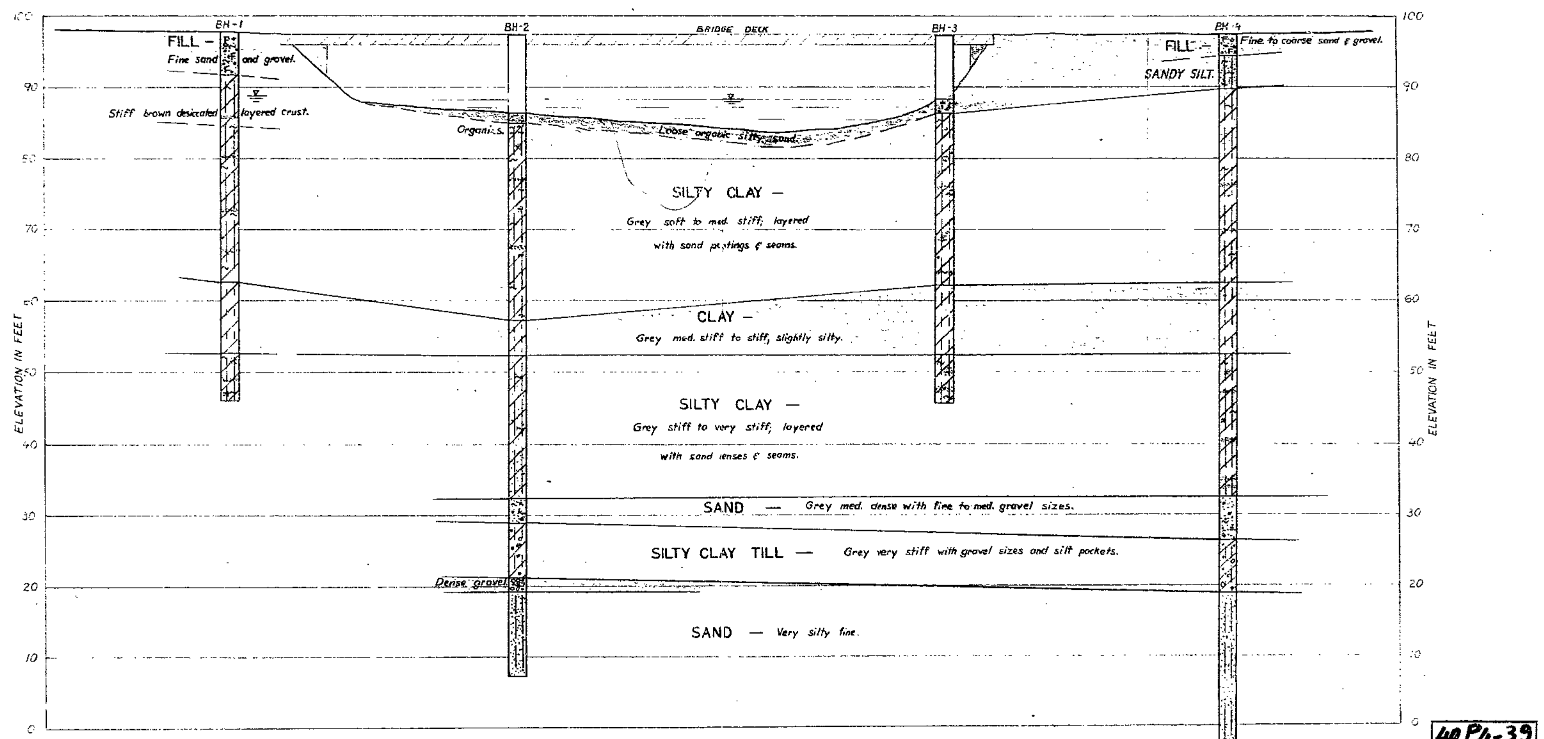
BOREHOLE LOCATION PLAN

SCALE: 1 IN. = 50 FT.

FOUNDATION INVESTIGATION
PTSEBE BRIDGE REPLACEMENT
PARKHILL CREEK
about 5 miles N-W of Parkhill.

WILLIAM A. TROW & ASSOC. LTD.

J 648 DWG. 1



ESTIMATED SUBSOIL STRATIGRAPHY SCALE: 1 IN. = 10 FT.

BA 1279

WILLIAM A. TROW AND ASSOCIATES LTD.

SITE INVESTIGATIONS
LABORATORY TESTING
SOIL MECHANICS CONSULTATION

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

1850 JANE ST.,
WESTON, ONT.
CH. 1-4644

40P4 - 39

40P4 - 39
GEOCRES No.

Project: J648

March 30, 1961

11084

M.M. Dillon and Co. Ltd.,
141 Maple Street,
London, Ontario

Attention: Mr. A. Phillips

Re: Foundation Conditions - Proposed Replacement, Ptsebe Bridge

Dear Sirs:

The enclosed report contains the results of our study of the subsoil conditions existing at this river crossing.

As stated during telephone conversations as the field work progressed, the site is underlain by soft to medium stiff silty clay materials for a depth of 65 feet and then by sand and somewhat stiffer clay below this level.

The soil at the river bed surface is too weak to support concentrated footing loads and therefore support of the bridge structure on wood piles driven to a depth of 55 feet below the present bridge deck level has been recommended. These piles will be essentially floating in the clay. The safe capacity for a class B timber pile has been computed to be 10 tons. The settlement to be anticipated with a pile group supporting a pier load of 440 kips has been estimated as $1\frac{1}{4}$ inches. This movement should be complete soon after load is applied.

The alternative to a pile-supported bridge is a structure composed of a series of corrugated pipes or of larger concrete sections. The unit stress exerted by this arrangement will be very low. For an average bearing stress of 300 psf, the estimated settlement should be less than 1 inch.

We shall be pleased to discuss any matter arising out of your review of this report if you so desire. We regret any inconvenience to you resulting from the delay in submitting this information.

Yours very truly,

W. Trow

William A. Trow, P.Eng.

WAT/gc
Enc.

M.M. DILLON AND CO. LTD.,
141 MAPLE STREET,
LONDON, ONTARIO

40P4-39
GEOCRES No.

FOUNDATION INVESTIGATION
PROPOSED PTSEBE BRIDGE REPLACEMENT
PARKHILL CREEK, N.W. OF PARKHILL, ONTARIO

Project: J648

March 30, 1961

William A. Trow and Associates Ltd.

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APPENDICES

ENCLOSURES

FOUNDATION INVESTIGATION PROPOSED PTSEBE BRIDGE REPLACEMENT PARKHILL CREEK, N.W. OF PARKHILL, ONTARIO

This report contains the results of a foundation investigation for the proposed replacement of the bridge noted above.

Consideration has been given to the use of timber piles for the support of this structure. Safe design loads and pile lengths are recommended, and the settlements associated with this foundation scheme are indicated.

Comments are also made on the soil mechanics aspects of a culvert design for this township road crossing of Parkhill Creek.

Field investigation methods and design calculations are presented in appendices to the report.

Site Description

The Ptsebe bridge is located about 5 miles northwest of Parkhill, Ontario. At the time of the investigation the creek was in flood and it was about 80 to 100 feet wide at the bridge crossing. During normal summer weather, however, it is much narrower and it occupies a deepened channel near the south side of the bridge. Some trees and small bushes line the river bank.

There appears to be little evidence of erosion of the river banks although the high water precluded accurate observations of the amount of scouring. It is reported that the maximum recorded flood level is about relative elevation 97.5 feet, or just above the deck of the bridge.

The existing bridge is a 100 feet long single span, steel truss structure with a timber deck. It is supported on timber piles. In general, the abutments are in poor condition, and replacement and reinforcement of some of the pile members has been required. At the northern abutment these repairs have included the underpinning of the bridge with an additional row of piles. The bridge was closed to traffic at the time of the investigation.

Soil Types Encountered

Four borings were put down at this river crossing to depths approaching 100 feet below the level of the bridge deck. Two of these were made through the existing bridge deck while borings number 1 and 4 were located in the road just back of each abutment. The details of the soil types encountered and a record of their properties are presented on the borehole logs, Dwgs. 2 - 5. An estimated stratigraphical profile has been prepared from these logs and is shown on Dwg. 1.

In general similar material was intersected at each borehole location. Below the loose alluvium in the river bed, and the fill of the bridge approaches, are lacustrine deposits of silty clay and clay which extend to a depth of approximately 65 feet or to El 32 feet. The upper surface of this material lies between El 92 feet and El 85 feet. For depths of 24 to 27 feet below these elevations the deposit consists of a soft to medium stiff layered silty clay with sand partings and seams. Scattered throughout the material are pieces of wood, hair roots, petrified leaves, grass fibres and shells. In many cases it was noted that there was a predominance of this organic material in the sand lenses and seams.

Field vane shear strength measurements indicate the presence of a stiff crust in hole 1 which extends down to about elevation 84.5 feet. This elevation corresponds approximately to the low water table level, and the high strengths are no doubt due to the desiccation of the soil during periods of low flow in the river. Laboratory triaxial and field vane strength tests show that the soil below the water table gains strength linearly with depth to a maximum value of about 600 psf.

Consolidation tests performed on samples from holes 2 and 3, (see Dwgs. 6 and 9), reveal that the soil is slightly preconsolidated by an amount which corresponds roughly to the equivalent weight of soil eroded from the level of the surrounding countryside down to the present river bed.

Natural moisture content and Atterberg Limit tests on samples from hole 2 show that there is little variation in the material with depth. In general, the plastic limit ranged between 19 and 21%, and the liquid limit between 33 and 36%. The natural moisture content is close to or slightly above the liquid limit.

Five to ten feet of slightly silty medium stiff uniform clay was intersected below a depth of 40 feet in hole 2 and below 35 feet in holes 1, 3 and 4. Some small shells and shell pieces were dispersed in this clay. Field and laboratory tests made on samples of this material indicated the following physical properties:

Undrained triaxial shear strength	=	730 psf
Field vane shear strength	=	700 to 1160 psf
Sensitivity	=	3.5 to 5.1
Natural unit weight	=	107 pcf
Liquid limit	=	70.8%
Plastic limit	=	25.4%
Natural moisture content	=	40 to 55%
Compression index	=	0.78

The silty clay, described above, was again intersected below about El 52 feet in all boreholes. It is somewhat stiffer than the material overlying the clay as would be expected for these greater depths. Below 56 feet, in holes 2 and 4, the lake deposit grades to a cohesive sandy silt and silty sand with numerous sand seams. Decaying wood, shells, leaves and grass fibres were found throughout the stratum. The lower interface was at approximately El 32 feet in both test locations.

A stratum of medium dense medium sand with gravel sizes and pieces of wood lies below the clayey silt. This layer is $3\frac{1}{2}$ to 6 feet in thickness and marks the lower limit of the lake deposits at this site.

The aforementioned lacustrine deposits have been formed in geologically recent times and probably originate from glacial Lake Nipissing. It is known that the highest elevation of Lake Nipissing was about El 605 feet and that this level was reached some 4300 years ago. It has also been established that the rate of rise of the lake level just prior to this time was about 7 feet per century. In terms of mean sea level datum, the deck of the bridge would be at about El 590 feet and the lower limit of the clayey silt at El 520 feet.

If it is assumed that this material was deposited in shallow waters at about the same rate as the rise in the lake level, it would appear likely that it was deposited during the Nipissing era from 4300 - 5500 years ago. The presence of the clay at a depth of about 40 feet could possibly reflect the formation of a sand bar on the lakeward side of the site which would give quiet waters for the deposition of the finer particle sizes. These opinions were given by Dr. Karrow of the Department of Mines of Ontario.

A stiff to very stiff silty clay glacial till deposit was intersected between El 26.4 feet and El 19 feet in hole 4 and El 28.8 to El 21.3 feet in hole 2. It contains fine to coarse gravel sizes and numerous silt pockets and intrusions.

A very silty fine sand extends from about El 19 feet to at least a depth of 90 feet in hole 2 and 100 feet in hole 1. A gravel layer 2 feet in thickness was also encountered immediately below the glacial till in hole 2.

According to well drilling records and the advice of local residents, bedrock lies at depths ranging from 150 to 175 feet below the surface.

Discussion of Foundation Requirements

It is understood that the proposed river crossing replacement will be constructed to the same grade and in approximately the same position as the existing structure. Two design schemes have been contemplated. One involves the replacement of the bridge with a three span structure incorporating 2 piers 60 feet apart and abutments on the shores 40 feet farther back. The other proposal envisages the use of a culvert or a series of corrugated metal pipes. The maximum pier loads for the bridge scheme will be in the order of 440 kips.

In view of the soft compressible nature of the underlying cohesive material at this site, it is considered that support for the piers and abutments may best be obtained from timber piles driven into the stiff silty clay found below El 50 feet. These piles would derive about 90 percent of their bearing capacity from the adhesion generated on the pile shafts and the remainder from end-bearing resistance. For example, calculations presented in appendix II show that a Class B timber pile, with an 8 inch tip, would safely support a load of about 10 tons if driven to a depth of about 55 feet below the existing bridge deck or to El 43 feet. No allowance was made for the adhesive resistance of the clay above El 83 feet when estimating this safe design load. It was considered that material above this level could be scoured out by the river. The variation of the adhesive resistance along the length of the pile was taken equal to the shearing strength of the soil up to a value of 600 psf. For overconsolidated soils having strengths greater than this, it has been found that only a portion of the total capacity is available for adhesive resistance and consequently allowance for this field experience has been made. *

In order to estimate the probable settlement of the piers due to the consolidation of the clay and silty clay strata it has been assumed that each pier would be supported on two rows of 11 piles with each member spaced at about 3 foot centres and 3 feet between the rows.

One common method of computing settlements assumes that the load of the pile group acts at the lower third point of penetration into the compressible soil and that it spreads into the underlying clay at 30 degrees to the vertical. Using these assumptions it may be seen that the equivalent footing would be located at about El 56 feet or near the upper surface of the highly compressible clay. As it is impossible for the centre of the piles to settle more than the pile tips it must be assumed that as soon as the clay begins to consolidate the resistance forces will be transferred to the less compressible material below. Hence in the analysis given in Appendix II the equivalent spread footing was taken to be at the lower surface of the clay, namely at El 52 feet.

The analysis indicates that a total consolidation settlement of about $1\frac{1}{4}$ inch will probably occur beneath each pier. The abutments are unlikely to settle more than about half this amount as the soil beneath these supports is somewhat stiffer and the applied loads less than for the piers.

If it is desired to reduce this settlement, it may be achieved by raking alternative piles. This will cause stresses to be spread over a much greater area of soil and piles will tend to act more as individual units. Settlements of the order of $\frac{1}{2}$ inch are anticipated if this method is used.

It is unlikely that refusal to driving of the piles will be experienced. Accordingly they should be stopped when the pile tips are at about El 43 feet.

* The Adhesion of Piles Driven in Clay Soils - M. J. Tomlinson

4th Int. Conference Soil Mechanics and Foundation Engineering, 1957

If the overall bridge load is assumed to be the same for the alternative culvert scheme the unit bearing stresses associated with this river crossing arrangement will be in the order of 300 psf. Since the magnitude of precompression of the main body of clay is estimated to be about 1000 psf, the application of this bridge load will merely cause a recompression of the soil.

According to the corrected consolidation test curves, Dwgs. 6 - 9, the modulus of compressibility of the clay above a depth of 50 feet, is equal to .008 sq.ft.per kip approximately within the range of pressures applicable for the culvert. The anticipated settlement resulting from the surcharge load of 300 psf assuming no dissipation of stress with depth is equal to

$$S = H m_v \Delta p$$
$$= 40 \times 12 \times .008 \times 0.3$$
$$= 1 \text{ inch approx.}$$

This conservative estimate should be well within tolerable limits.

If the culvert scheme is adopted, the very soft deposits of organic material, which cover the creek bed for a depth of about 3 feet, should be removed and replaced with a blanket of clean sand or gravel. This material should be compacted in layers not exceeding 12 inches loose thickness.

Conclusions

The pertinent observations and conclusions contained within this report may be briefly summarized as follows:

1) The site for the proposed Ptsebe bridge replacement is underlain by cohesive lacustrine deposits to a depth of up to 65 feet below the existing bridge deck. They consist of a soft to medium stiff silty clay with thin sand lenses and seams to a depth of 35 to 40 feet, a medium stiff clay to a depth of 45 feet, and a stiff silty clay with an increasing proportion of sand sizes and seams below 45 feet. Pieces of wood, shells, grass fibres and leaves were found scattered throughout these strata. About 3½ to 6 feet of medium sand and 7 feet of very stiff silty clay glacial till were intersected below El 32 feet in holes 2 and 4. A very silty fine sand extends below the till to a depth of at least 100 feet.

2) As soft cohesive deposits underlie this site to a considerable depth it is felt that timber friction piles driven to approximately El 43 feet should provide the best means of support for the Ptsebe bridge. Analyses show that a Class B pile, with an 8 inch tip, should be capable of supporting a safe load of about 10 tons.

3) The maximum settlement of the pile group under each pier should not exceed 1¼ inches. This movement may be reduced by raking some of the piles

within the group and thereby spreading the load to a greater area of soil.

4) The Soil Mechanics aspects of a culvert design at this site are also considered. The maximum settlement with this scheme should not be more than 1 inch.

REGISTERED PROFESSIONAL ENGINEER
CIVIL
G. H. WHEELER
PROVINCE OF ONTARIO

G. H. Wheeler

G.H. Wheeler, P.Eng.

GHW/gc
Mar.30, 1961
J648

REGISTERED PROFESSIONAL ENGINEER
CIVIL
W. A. TROW
PROVINCE OF ONTARIO

W. Trow

W.A. Trow, P.Eng.

APPENDIX I

Field Investigation Methods

Conventional wash boring equipment was used to put down the four borings for the subsoil investigation at this site. The borings ranged in depth from 50 to 100 feet, and were cased with 3 inch pipes to the depths indicated on the borehole logs, Dwgs. 2 to 5.

Soil samples were recovered at intervals of from 5 to 10 feet to a depth of 75 feet. Below this level, the soil profile was estimated from careful examination of the wash water return and noting the resistance to penetration of the wash bit.

Disturbed samples were obtained using a conventional 2 inch O.D. split spoon sampler. The spoon was driven into the ground under an energy of 350 ft. lbs. per blow, the number of blows for the lower foot of penetration being recorded on the borehole logs. A small sample of any clay recovered was carefully wiped, wrapped in aluminum foil and sealed in a polythene bag with the remainder of the sample. The natural moisture content of the clay was determined from a portion of the sealed clay.

Undisturbed samples of the cohesive soils were recovered by levering 2 inch I.D. shelby tubes into the soil in the bottom of the borings. On recovery the ends of these tubes were sealed with aluminum foil and wax to prevent moisture loss.

The results of field vane strength tests performed on the undisturbed soil are shown on the borehole logs. The sensitivity or ratio of undisturbed to remoulded strength is also indicated beside each strength measurement.

The elevation of the road surface or bridge deck adjacent to each boring has been related to the benchmark shown on Dwg. 1. This benchmark consists of a nail in the centre of a stump about 14 feet east of Station 4 + 63. It has been assumed to have a relative elevation of 100.0 feet.

APPENDIX II

Bearing Capacity and Settlement Calculations

1) Safe Capacity of Timber Piles

The total safe load for a timber friction pile may be computed from the expression:

$$Q = \frac{CNA}{F} + \frac{P}{F} \sum c_a l$$

where:

C = the undrained shear strength of the clay

c_a = the adhesion force between the pile and the clay

A = the cross-section area of the pile tip

P = the perimeter of the pile shaft

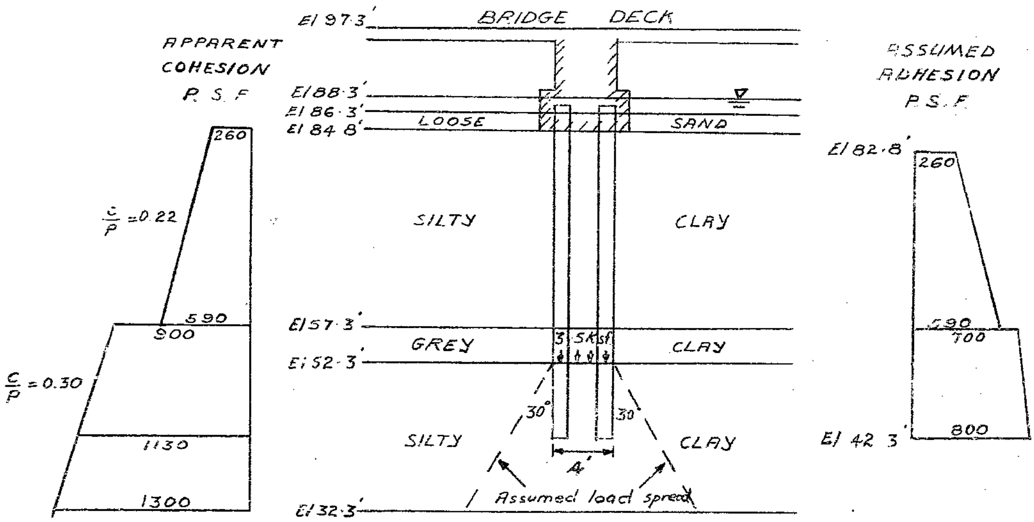
l = the length of the pile shaft in the clay

N = a bearing capacity factor dependent on the shape and depth of foundations.
For deep piles $N = 9$

F = a factor of safety required to keep settlement within tolerable limits.
$F = 3$ for this case.

For a class B timber pile with an average diameter of 10 inches and a tip diameter of 8 inches, driven to a depth of 55 feet below the bridge deck, or to El 42 feet, $P = 2.62$ feet and $A = 0.35$ sq. ft.

Assume the depth/adhesion values shown in the sketch below with no adhesive resistance available above El 82.8 feet. The strength below El 57 has been reduced slightly according to observations by Tomlinson.



$$\text{Therefore:} \quad Q = \frac{1130 \times 9 \times 0.35}{3} + \frac{2.62}{3}(25.5 \times 435 + 15 \times 770)$$

$$= 1190 + 19800$$

$$= 20990 \text{ lbs.}$$

$$= 21 \text{ kips}$$

2) Consolidation Settlement of Pile Groups

Maximum pier load = 440 kips

Number of piles required $= \frac{440}{21} = 21$

Assume 2 rows of 11 piles with a spacing of 3 feet between the rows and the individual piles.

The pile group may be considered to act as a footing having external dimension equal to 31 feet by 4 feet. A load spread of 30 degrees to the vertical through the edges of this footing has been assumed for the estimation of the net increase in pressure below the foundations.

The consolidation settlement is given by

$$S = m_v \, \Delta p \, H$$

where: m_v is the coefficient of volume decrease and is obtained for the range of pressures considered, from the corrected laboratory consolidation curves shown on Dwgs. 6 to 9

H is the thickness of the stratum of clay

Δp is the average net increase in pressure

Depth below footing ft.	Equivalent footing size ft.	Footing Area sq. ft.	Δp ksf	m_v sq.ft./kip	H ins.	ins.
0	4 x 31	124	3.55			
5	9.7 x 36.7	356	1.24	0.006	120	0.89
15	21.2 x 48.2	1020	0.43	0.006	120	0.31

Total Settlement = 1.2 ins.

View of bridge and river
looking west

View of bridge
looking south

View of river and bridge
looking south

View of northern abutment

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 2
PROJECT No. J648

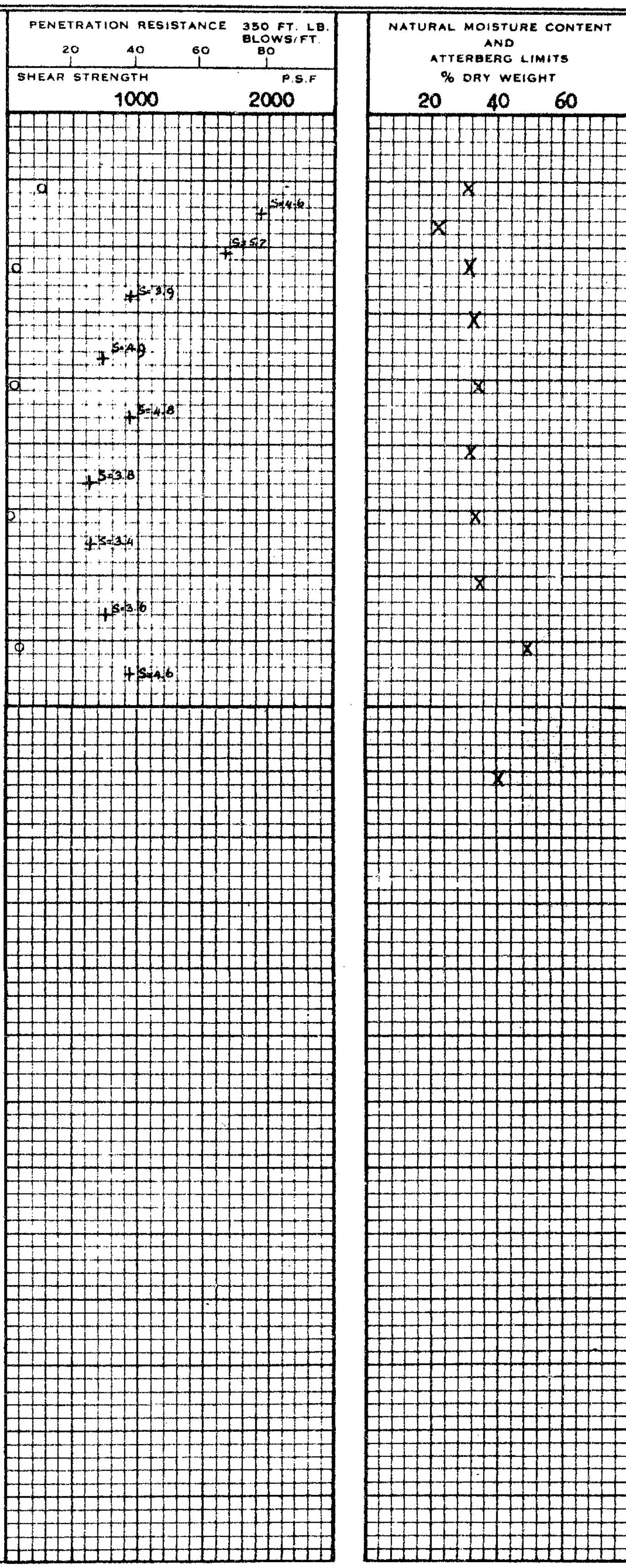
LEGEND

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—o—
2" I.D. SHELBY TUBE	—x—x—x—
2" DIA. CONE	———
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	+ S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X LI
ATTERBERG LIMITS	
LIQUID LIMIT	—o
PLASTIC LIMIT	⊢—
SAMPLE TYPE	
2" O.D. SPLIT TUBE	▨
2" I.D. SHELBY TUBE	■
3" O.D. SHELBY TUBE	▧

BOREHOLE No. 1
PROJECT Proposed Ptsebe Bridge Replacement
LOCATION Parkhill Creek
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 97.7 ft.
DATUM Nail in centre of stump behind 12" Ø Ash, east of St. 4 + 63 ± at El 100.0 ft.

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET
	Road surface	97.7	0
	FILL-Fine sand, & gravel with some wood pieces. 5'-6" to 6'-0" - sandy topsoil	91.7	
	SILTY CLAY-Stiff, brown, desiccated, layered crust with some sand & silt bands to 12 ft. Grey & med. stiff with sand partings & layers below this depth. Sand sizes noted in the silt from 12 - 16½ ft. Pieces of wood, roots, leaves, grass and shells noted throughout.	88.7	10
		62.7	
	CLAY-Grey, medium stiff, slightly silty uniform with small shells.	52.7	40
	SILTY CLAY-Stiff grey layered with sand lenses and seams. Shells, roots, and wood pieces throughout.		50
	End of Hole	46.2	



SAMPLE TYPE AND No.
1
2 -L
3
4 -P
5
6 -P
7
8 -P
9
10 -L

NATURAL UNIT WEIGHT P.C.F.

L = Levered into ground

P = Pushed into ground

Notes:
1) Hole put down with conventional wash boring equipment.
2) Hole cased to 11 ft. Washed ahead below this depth, hole remained open.
3) W.L. at 9 ft. 15 hours after completion.

Depth scale: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING NO. 3

PROJECT NO. J648

BOREHOLE NO. 2

PROJECT Proposed Ptsebe Bridge Replacement

LOCATION Parkhill Creek

HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 97.3 ft.

DATUM As for hole 1.

LEGEND

PENETRATION RESISTANCE
- 2" O.D. SPLIT TUBE —o—o—o—
- 2" I.D. SHELBY TUBE —x—x—x—
- 2" DIA. CONE ————

SHEAR STRENGTH
- UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
- UNCONFINED COMPRESSION ⊗
- VANE TEST AND SENSITIVITY (S) +S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X LI

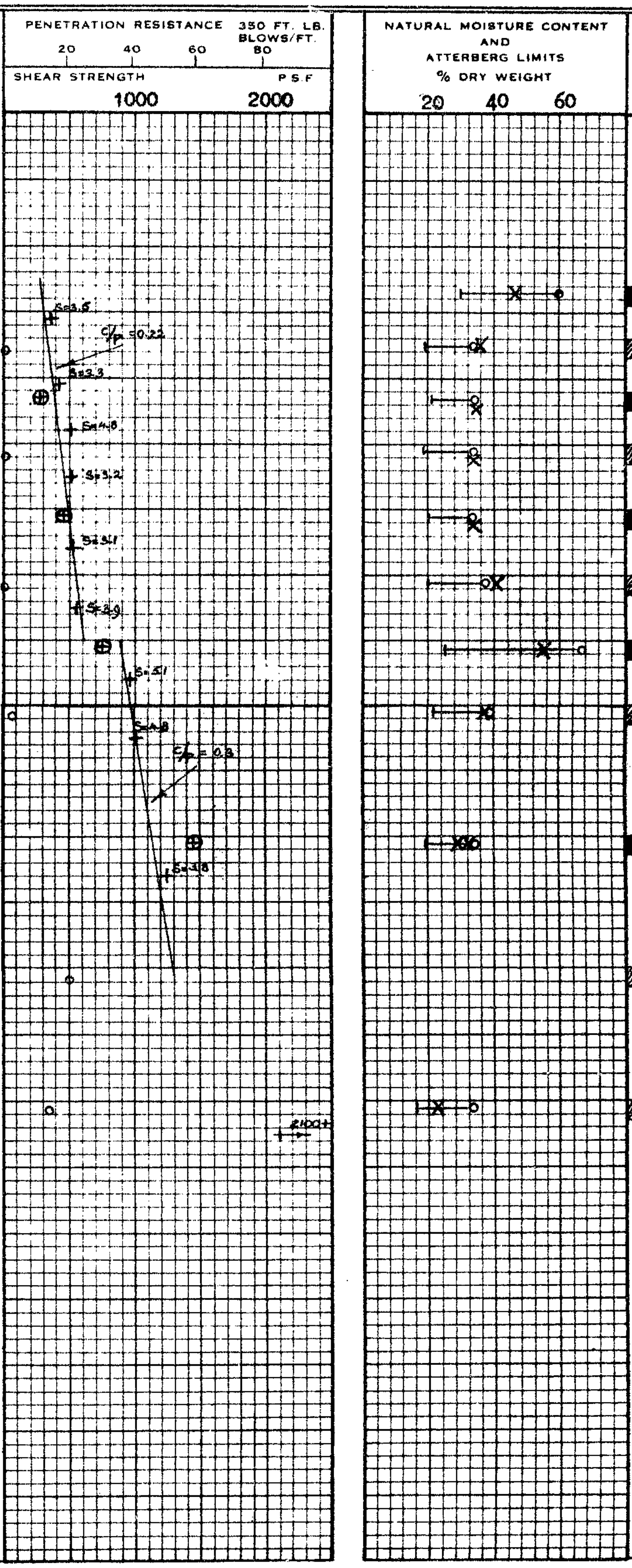
ATTERBERG LIMITS
- LIQUID LIMIT —o
- PLASTIC LIMIT ⊢—

SAMPLE TYPE
- 2" O.D. SPLIT TUBE
- 2" I.D. SHELBY TUBE
- 3" O.D. SHELBY TUBE

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
	Bridge deck	97.3	0		
	River Bed	88.3	10		
	ORGANIC SILTY SAND-Grey, very loose	86.3			
	SILTY CLAY-Grey, soft to medium stiff, layered, irregular coarse silt and fine sand lenses and seams with some hair roots, decayed wood, vegetation, small shells noted in silt throughout, organic dark brown silty, sandy clay with much organics from 12½ - 14½ feet.	84.8	20, 30	1 -L; 2; 3 -P; 4; 5 -P; 6	117 (3); 121 (5)
	CLAY-Grey, medium stiff, slightly silty, uniform, with small shells.	57.3	40	7 -P	107
	SILTY CLAY-Grey, stiff, layered, sandy with some decayed wood, leaves and root hairs.	52.3	50	8; 9 -L	117.5 (9)
	Becoming cohesive silty fine sand with organics below 57 feet.		60		
	SAND-Grey, med. dense, med. with fine to med. gravel sizes. Wood pieces noted.	32.3		10	
	SILTY CLAY TILL-Grey very stiff, with gravel sizes. Silt pockets and intrusions.	28.8	70	11	
	GRAVEL-Dense.	21.3			
	SAND-Very silty fine. (No sampling carried out).	19.3	80		
	End of Hole	7.3	90		

L = Levered into ground

P = Pushed into ground



Notes: 1) Hole put down with conventional wash boring equipment.

2) Hole cased to 40 ft. and then to 70 ft. Washed ahead below this depth, hole remained open to 90 ft. Could not penetrate below 90 ft. because of caving.

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SITE INVESTIGATIONS · SOIL MECHANICS CONSULTATION

DRAWING No. 4

PROJECT No. J648

LEGEND

PENETRATION RESISTANCE	
2" O.D. SPLIT TUBE	—o—o—o—
2" I.D. SHELBY TUBE	—x—x—x—
2" DIA. CONE	———
SHEAR STRENGTH	
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE	⊕
UNCONFINED COMPRESSION	⊗
VANE TEST AND SENSITIVITY (S)	+S

NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX	X LI
ATTERBERG LIMITS	
LIQUID LIMIT	—o
PLASTIC LIMIT	⊢—
SAMPLE TYPE	
2" O.D. SPLIT TUBE	▨
2" I.D. SHELBY TUBE	■
3" O.D. SHELBY TUBE	▧

BOREHOLE No. 3

PROJECT Proposed Ptsebe Bridge Replacement

LOCATION Parkhill Creek

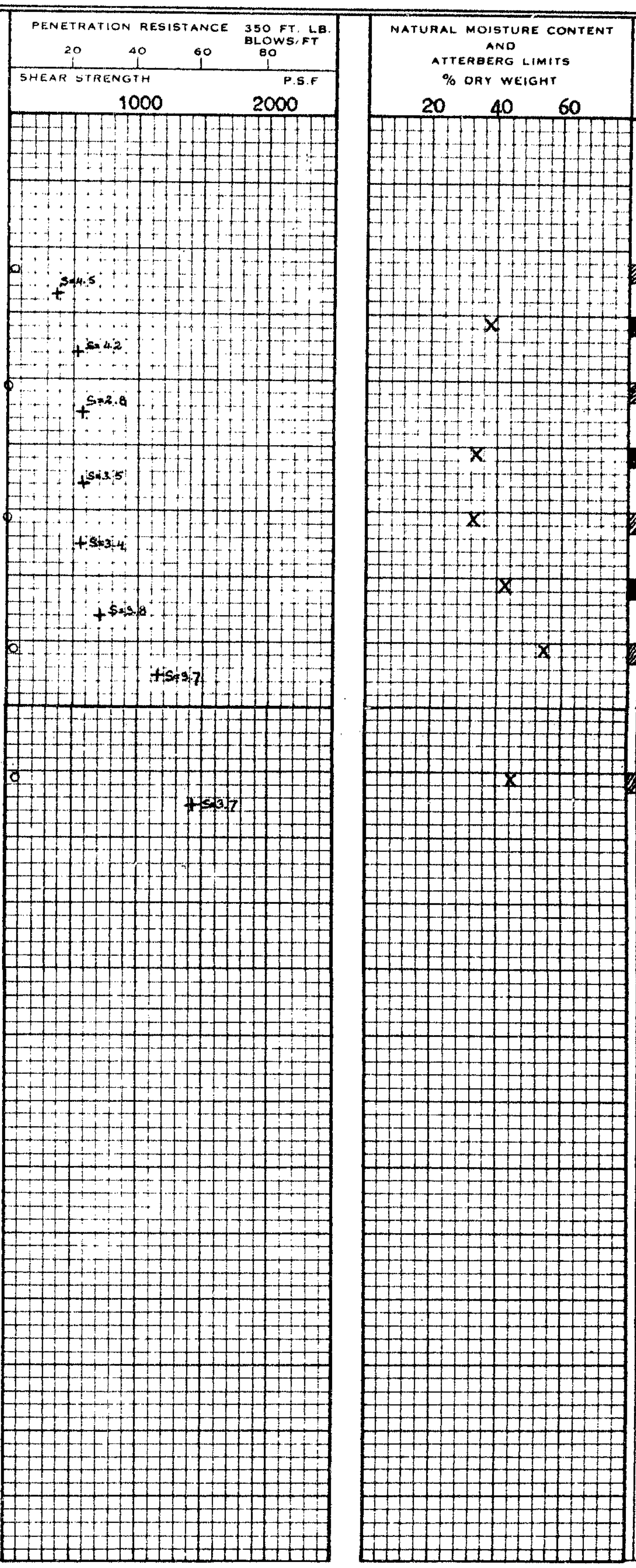
HOLE LOCATION See Dwg. 1.

HOLE ELEVATION 97.3 ft.

DATUM As for hole 1.

SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No	NATURAL UNIT WEIGHT P.C.F.
	Bridge deck	97.3	0		
	River bank				
	SILTY SAND-Med to coarse, loose, gravel and shells.	88.3	10		
	SILTY CLAY-Grey, soft to medium stiff with occasional fine gravel. Shells, roots, grass, wood, and leaf particles throughout. Silt layered below 15 ft. and contains lenses and seams of fine sand and coarse silt.	86.3		1; 2 -P; 3; 4 -P; 5	
	CLAY-Grey medium stiff to stiff, uniform, slightly silty with small shells throughout.	62.3		6 -P; 7	
	SILTY CLAY-Grey, stiff layered with lenses and seams of fine sand. Organics throughout.	52.3		8	
	End of Boring	45.8			

Depth scale: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110



Notes: 1) As for hole 1.

2) Hole cased to 30 ft.
Washed ahead below this depth, - hole stayed open.

P - Pushed into ground

WILLIAM A. TROW & ASSOCIATES LTD.

SITE INVESTIGATIONS   SOIL MECHANICS CONSULTATION

DRAWING No. 5
PROJECT No. J648

BOREHOLE No. 4
PROJECT Proposed Ptsebe Bridge Replacement
LOCATION Parkhill Creek
HOLE LOCATION See Dwg. 1.
HOLE ELEVATION 97.4 ft.
DATUM As for hole 1.

LEGEND

PENETRATION RESISTANCE
2" O.D. SPLIT TUBE —o—o—o—
2" I.D. SHELBY TUBE —x—x—x—
2" DIA. CONE ———

SHEAR STRENGTH
UNDRAINED TRIAXIAL AT OVERBURDEN PRESSURE ⊕
UNCONFINED COMPRESSION ⊗
VANE TEST AND SENSITIVITY (S) $+^{S}$

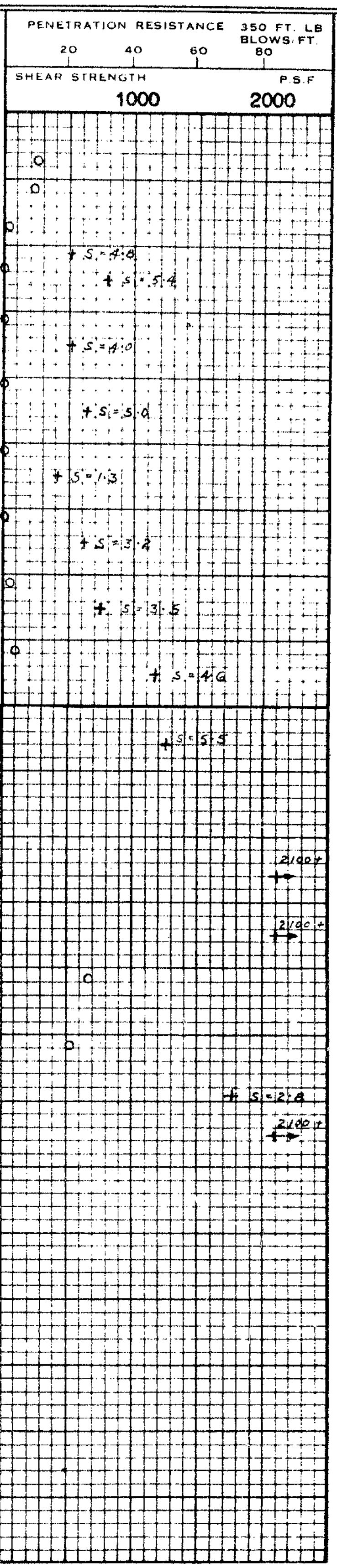
NATURAL MOISTURE CONTENT AND LIQUIDITY INDEX X^{LI}

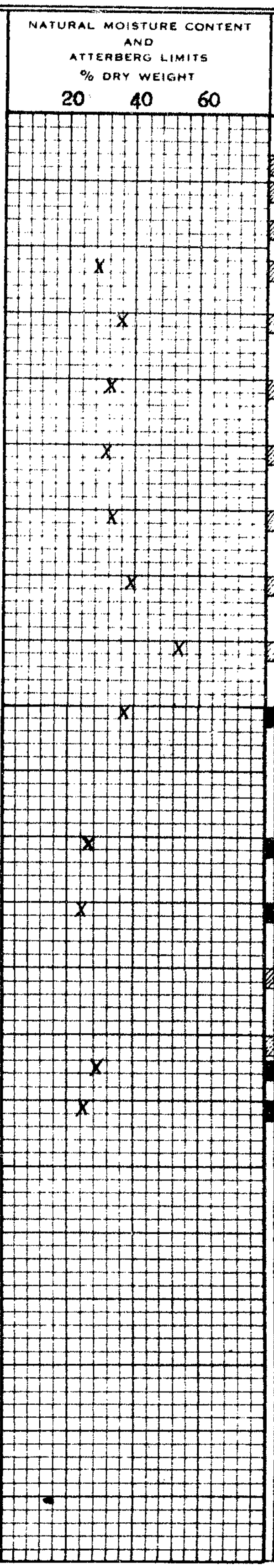
ATTERBERG LIMITS
LIQUID LIMIT —o
PLASTIC LIMIT ⊢—

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

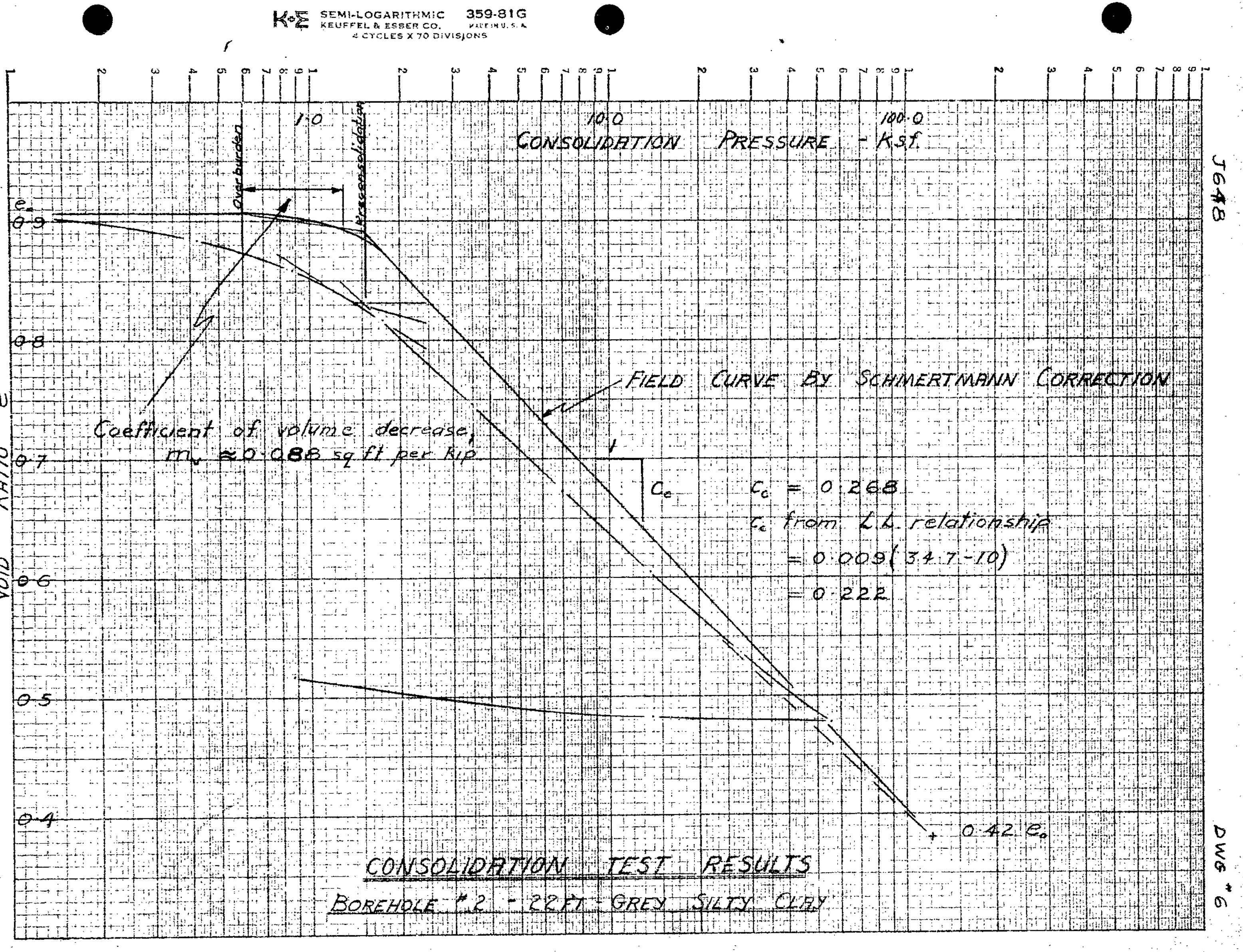
SYMBOL	SOIL DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE TYPE AND No.	NATURAL UNIT WEIGHT P.C.F.
	Road surface	97.4	0		
	FILL-Fine-coarse sand & gravel.	94.4			
	SANDY SILT-Brown, loose, fine, slightly cohesive with pieces of wood, grass and roots.	89.7		1	Lost
				2	
	SILTY CLAY-Grey, soft, to medium stiff, layered with sand partings and seams. Some sand sizes in silt above 15 ft.		10	3	Lost
				4	
				5	
	Pieces of wood, roots, grass, leaves and shells throughout.		20	6	
				7	
		62.4	30	8	
	CLAY-Grey, medium stiff to stiff, slightly silty, uniform with small shells throughout.			9	
		52.4	40	10	
	SILTY CLAY-Grey, stiff to very stiff, layered, with sand lenses and layers. Pcs. of wood, roots, grass, leaves and shells.		50	11-P	
	Sandy cohesive silt with irregularly spaced fine sand seams up to ½ inch in thickness below 56 ft.			12-L	
		32.4	60	13-P	
	SAND-Grey, medium dense, medium with fine to medium gravel sizes. Wood pieces noted 65 - 66½ ft.	26.4		14	
	SILTY CLAY TILL-Grey very stiff with fine to coarse gravel sizes. Numerous silt pockets and intrusions.	19.0	70	15	Lost
				16-L	
				17-L	
	SAND-Very silty, fine (No sampling carried out).		80		
			90		
	End of Hole	-2.6	100		
			110		

P = Pushed into ground
L = Levered into ground



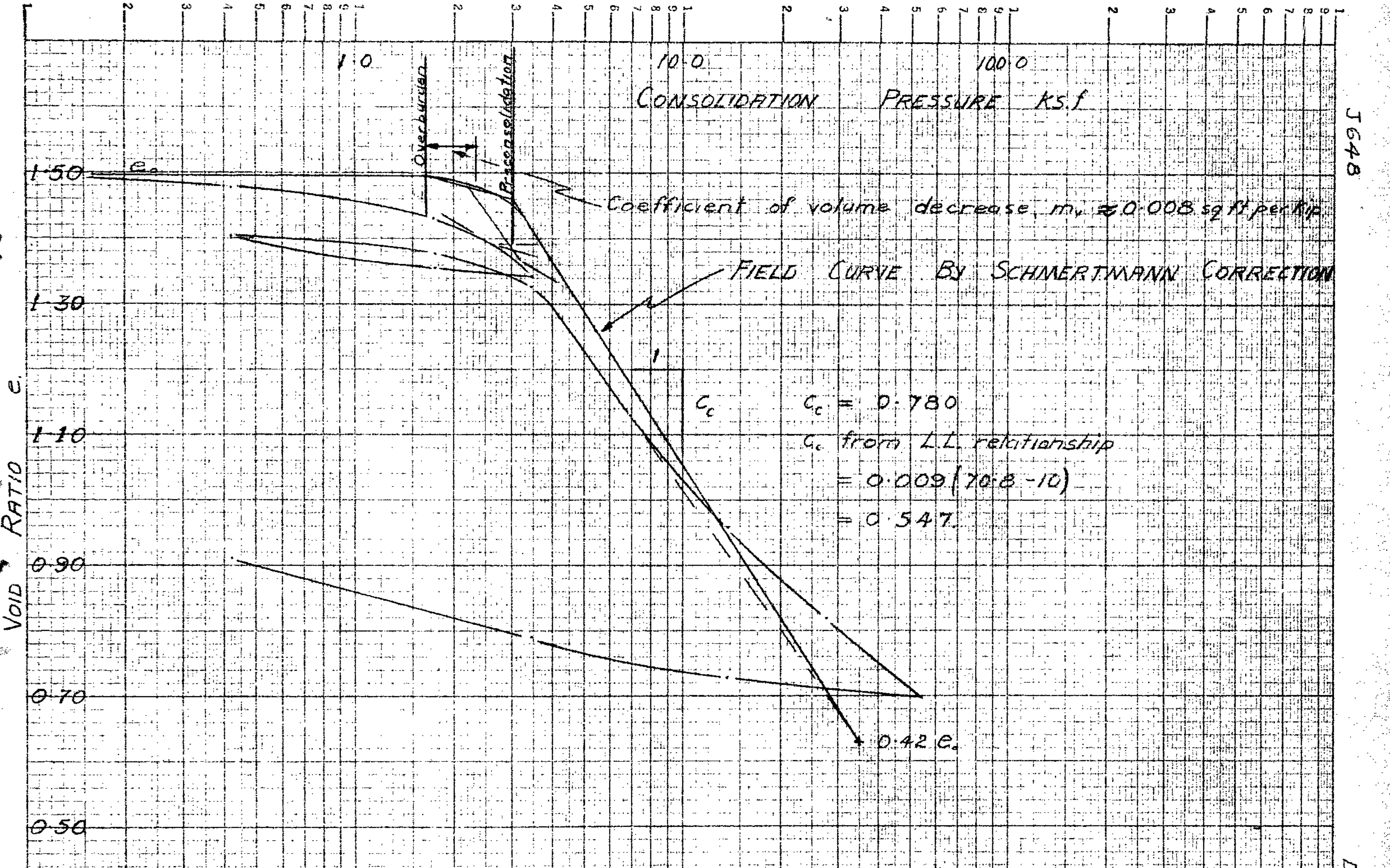


Notes: 1) As for hole 1.
2) Hole cased to 70 ft. Washed ahead below this depth. Could not penetrate below 100 ft. because of caving.
3) With casing to 70 ft., W.L. measured at 11.0 ft. after 60 hours.



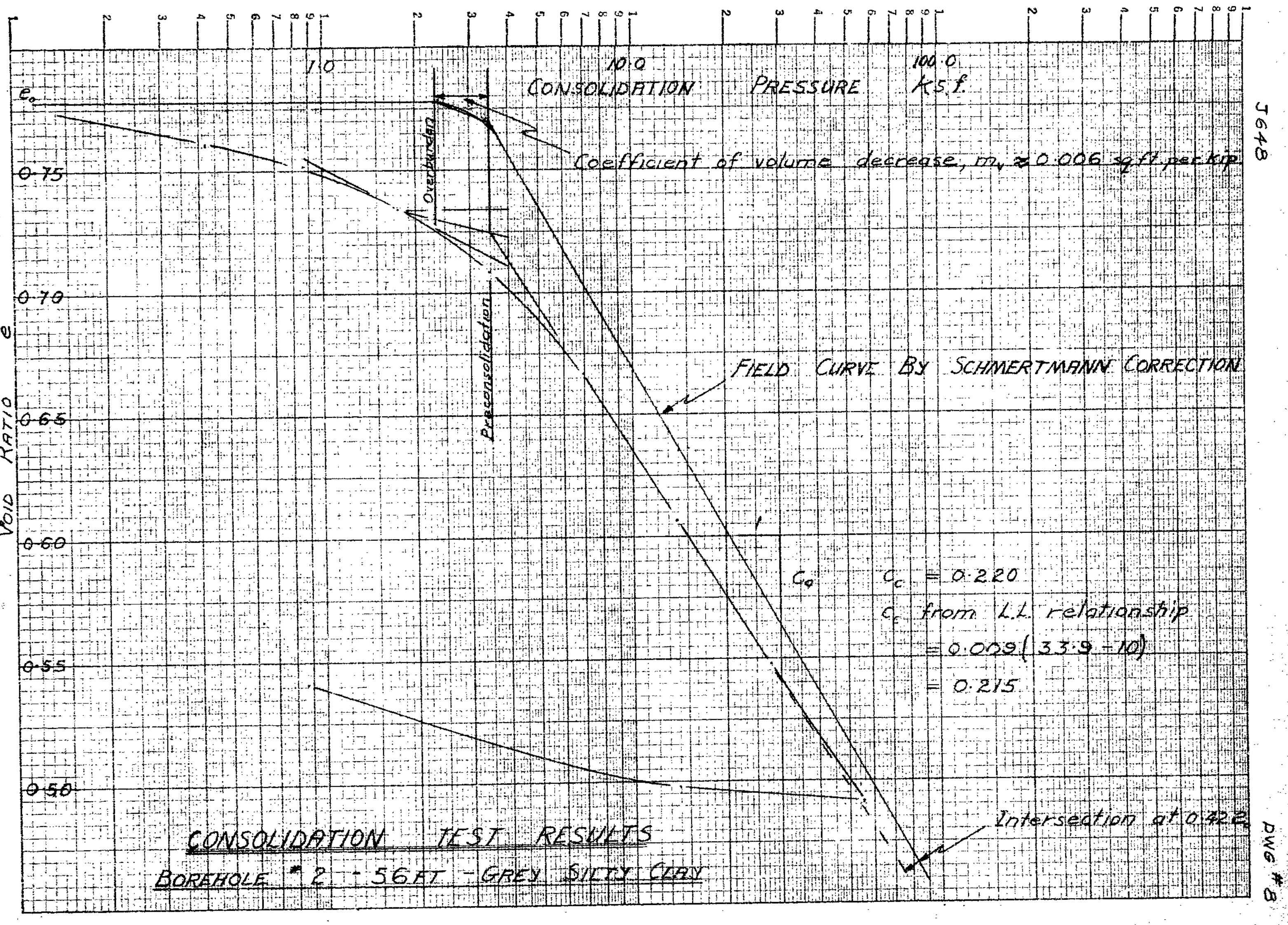
CONSOLIDATION TEST RESULTS

BOREHOLE #2 - 22 FT GREY SILTY CLAY



CONSOLIDATION TEST RESULT

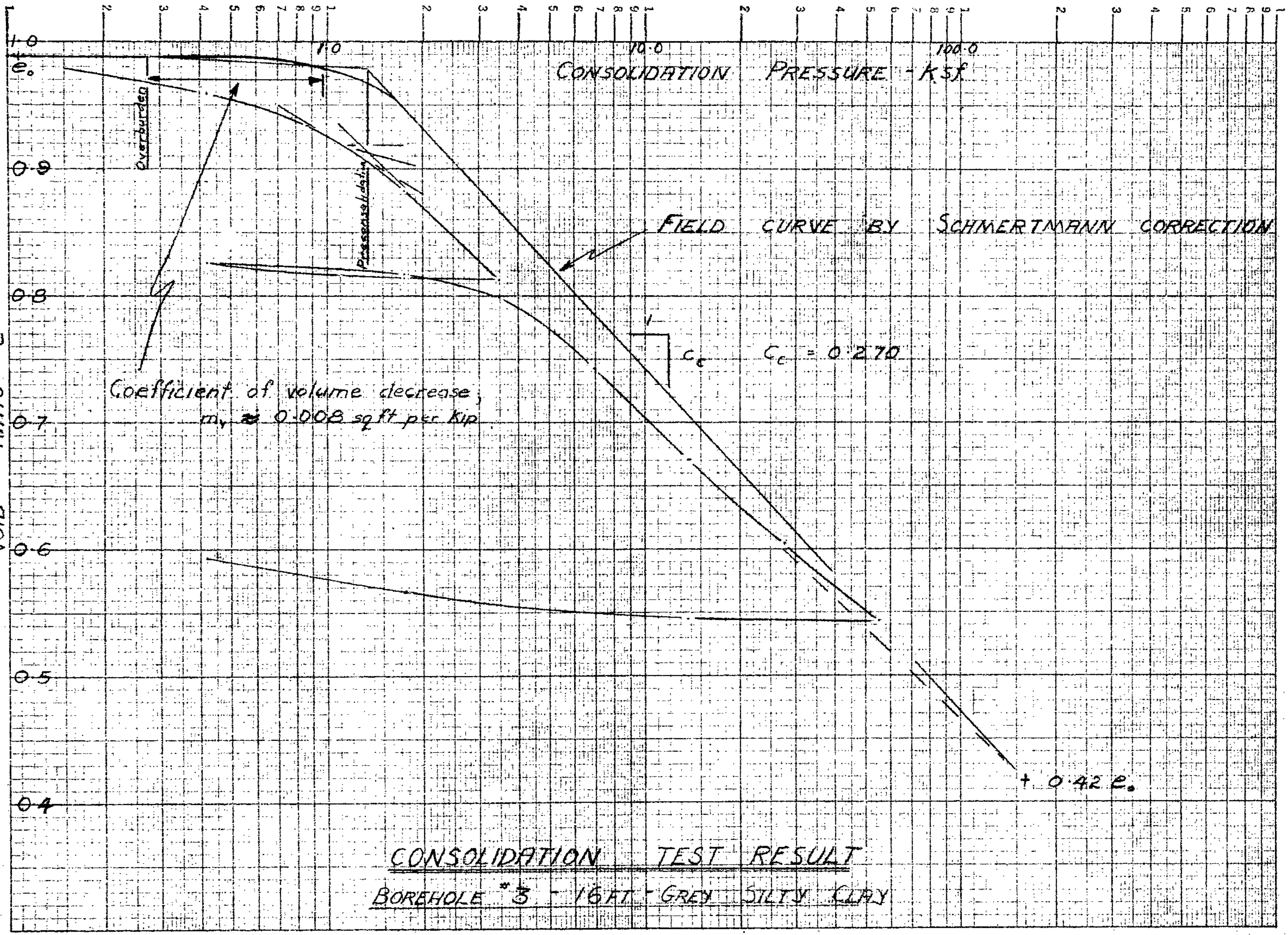
BOREHOLE # 2 - 41 FT - GREY CLAY



K+E SEMI-LOGARITHMIC 359-81G
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS
1.0
10.0
100.0
CONSOLIDATION PRESSURE K.S.F.
e_o
Overburden
Preconsolidation
Coefficient of volume decrease, m_v ≈ 0.006 sq ft per kip
0.75
0.70
0.65
0.60
0.55
0.50
VOID RATIO e
FIELD CURVE BY SCHMERTMANN CORRECTION
C_c = 0.220
C_c from L.L. relationship
= 0.009 (33.9 − 10)
= 0.215
Intersection at 0.422
CONSOLIDATION TEST RESULTS
BOREHOLE # 2 - 56 FT - GREY SILTY CLAY
J648
DWG # 8

K&E SEMI-LOGARITHMIC 359-81G
KEUFFEL & ESSER CO. MADE IN U.S.A.
4 CYCLES X 70 DIVISIONS

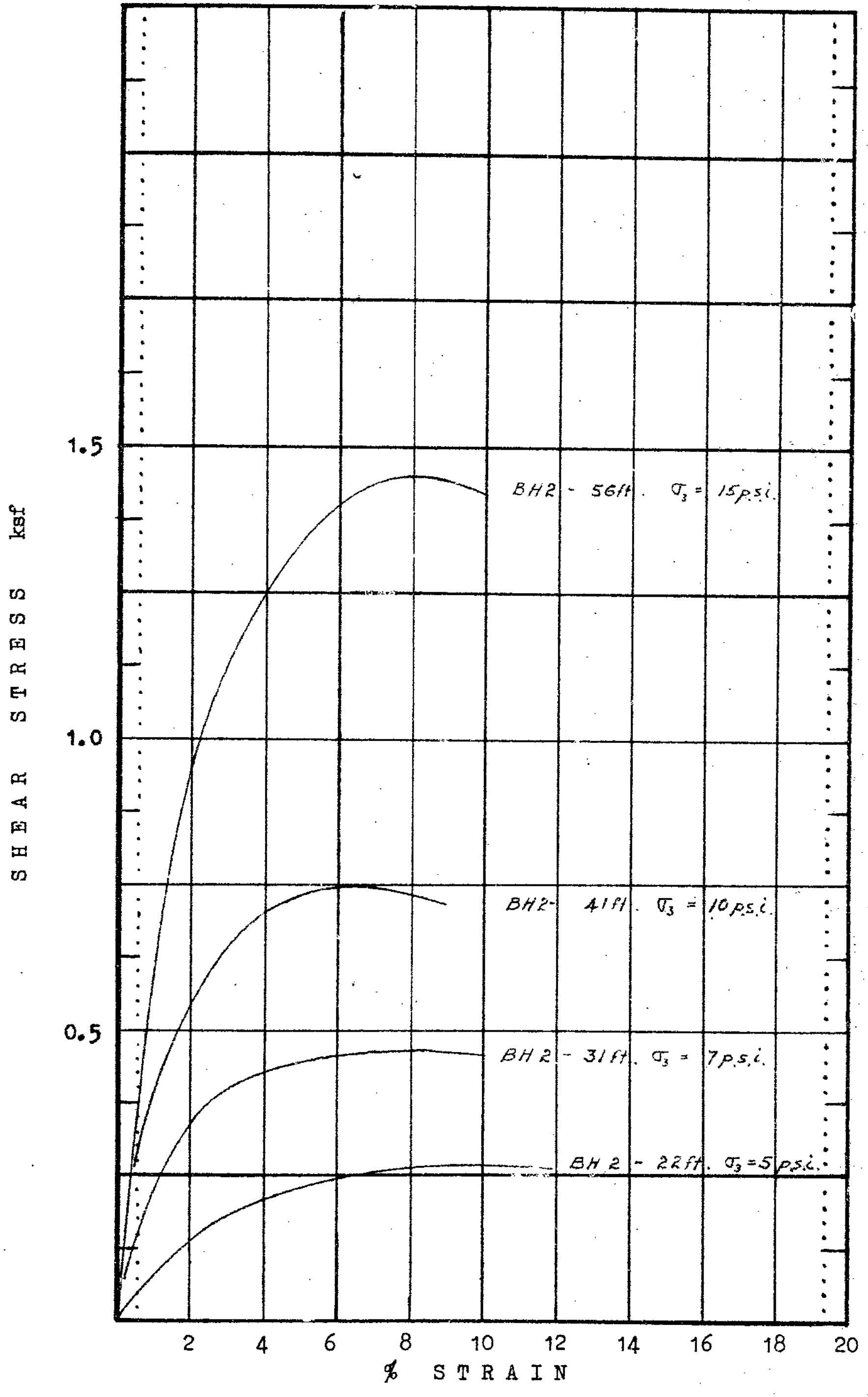
J648



DWG #9

PROJECT NO. J648

DRAWING NO. 10



STRESS STRAIN CURVES - UNDRAINED TRIAXIAL TESTS

WILLIAM A. TROW AND ASSOCIATES