

W.O. 73-F-214M

M^cMILLAN

BRIDGE

RECONSTRUCTION

40J9 · 16

73-F-214M

PERO ASSOCIATES LIMITED

40-1-161
GEOCRE No.

1039-16
GEOCRE No.

Soils Investigation Report
McMillan Bridge Reconstruction
for
The Township of Sombra
c/o Todgham & Case

Distribution:
4 cc Client
1 cc File

JOB NO. 70-F117

NOVEMBER, 1970

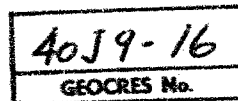


PETO ASSOCIATES LTD.
CONSULTING SOIL ENGINEERS
2150 Highway 7, Concord, Ontario • Phone: (416) 889-8384

JOB NO. 70-P117

November 23, 1970.

The Township of Sombra,
c/o Todgham & Case,
Consulting Engineers,
151 Thames Street,
P.O. Box 386,
Chatham, Ontario.



Attention: Mr. H.H. Todgham, P. Eng.

Dear Sirs:

Re: Soils Investigation Report
McMillan Bridge Reconstruction

We are pleased to submit four (4) copies of our soils report on the above site.

The subsoil stratigraphy at the site includes the following:

Topsoil
Grey Brown Sandy Silt
Organic Clayey Silt
Grey Brown Clayey Sand Silt
Grey Silty Fine Sand
Light Brown Silty Clay
Grey Brown Silty Clay

The light brown and grey brown silty clays predominate extending from a depth of 19'± to a depth of 120'± below existing grade. This is partially based on our report #6074.

Based on the information acquired from this investigation, as well as on information taken from reports in this same area, the use of friction piles have been discussed fully in this report. Mention has also been made regarding the suitability of end bearing piles.

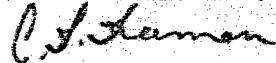


The water table on the site was found to be at the same elevation as the water in the St. Clair River at the time of the investigation.

We believe we have provided the information requested, however, we will be pleased to be of further assistance.

Yours very truly,

PETO ASSOCIATES LTD.



C.F. Freeman, P. Eng.,
Chief Engineer.

CFF/vs

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SITE PLAN

1. AUTHORITY

Authority for this work was received in a transmitted note from Todgham & Case dated September 11, 1970.

2. SITE LOCATION

McMillan Bridge is an existing plank bridge over a canal to a Marina development. The location is south of Port Lambton on Highway 40 to the road junction at Babys Point, keeping to the right on the gravel road, and taking a right turn to Riverside Drive (official description - south end of Sombra Township, Concession V, Lot E).

3. PROPOSAL

It is proposed to straighten out the existing alignment by replacing the existing bridge in the most economical way possible, perhaps by 3 or 4 bearing piles to rock with a concrete sill poured on top with a precast deck set on this. Light steel or wooden sheeting may then be driven to line the channel and retain the soil. The new span will be about 20 ft. in length. The maximum load is expected to be a 20 ton truck.

4. OBJECTIVES

The objectives of this investigation are to determine the stratigraphic and strength parameters of the subsoil for the purpose of recommending a suitable foundation design.

5. SITE AND GEOLOGY

The proposed bridge will replace the existing timber bridge structure at this location. The site on both sides of the proposed bridge is flat and dry. The approaches to the existing bridge are gravel roads at the same elevation as the bridge. The level of the

existing bridge was some 2½ feet above the recorded high water level in the channel. The width of the canal at the site investigated was approximately 20 feet.

The clay deposits, as encountered at this site, are clearly of lacustrine origin. The glacial lakes that covered this area in Pleistocene time failed to leave deep stratified beds of sediment on the underlying clay. Based on other work done in the area (Ref. #6074) it is believed that a till of very dense consistency underlies the clay and confirms the lacustrine character of these deposits. The bedrock consists of black shale which is an important component of the clays encountered.

6. FIELD WORK

The field work consisted of one borehole to 50'± and a probe driven in the hole to refusal. Thin wall samples were taken at 10' intervals below 20' from surface. All field work was carried out using a Standard drill rig mounted on skids. Samples were obtained by driving a split-spoon sampler in accordance with the requirements of the standard penetration test. Field identification of samples was verified by a qualified engineer.

The final locations and elevations of all holes were established by the Client and forwarded to this office. These may be seen on the Site Plan appending this report.

7. LABORATORY WORK

Laboratory work consisted of moisture content determinations on all split spoon samples, a grading on a representative sample of sand, determination of the liquid limits of a clay sample, one consolidation test on a thin walled sample of the same clay and three shear strength tests (Figure 1).

8. SOIL CONDITIONS

The following main soil types were encountered:

- a) Topsoil
- b) Grey Brown Sandy Silt
- c) Organic Clayey Silt
- d) Grey Brown Clayey Sandy Silt
- e) Grey Silty Fine Sand
- f) Light Brown Silty Clay
- g) Grey Brown Silty Clay

A detailed description of each soil layer encountered follows:

- a) **TOPSOIL:** The uppermost layer consisted of brown silty loam topsoil. The depth of this layer was found to be about 1'.
- b) **GREY BROWN SANDY SILT:** Underlying the topsoil, a layer of grey brown sandy silt was met between 1 and 3 feet 6 inches below the existing grade. This material contains a substantial amount of root hairs and organic material. According to the results of the standard penetration test, the N value (i.e. the number of blows per foot penetration as obtained in standard penetration test) was in the order of 3 indicating the soft consistency of the layer. The natural moisture content of this material is about 19%.
- c) **ORGANIC CLAYEY SILT:** Underlying the grey brown sandy silt is a layer of dark brown organic clayey silt. The layer extends from 3'6" to 5'4". The natural moisture content showed that the material was very wet. An N value of 1 indicates a very soft consistency for this material.
- d) **GREY BROWN CLAYEY SANDY SILT:** Underlying the organic silt a layer similar to that described in (b) was encountered to a depth of 8'±. An unexpected high N value of 13 was recorded in this layer probably the result of scattered roots and woody organic. The moisture content in this area was about 27%.

- e) **GREY SILTY FINE SAND (FIGURE 2):** Underlying the grey brown clayey sandy silt is a layer of grey silty fine sand. The layer was found to be fully saturated at the time of the investigation. The results of the standard penetration test as obtained in this layer indicate a loose condition with N values between 5 and 9 blows/ft. The moisture content is about 22%.
- f) **LIGHT BROWN SILTY CLAY:** Underlying the grey silty fine sand is a layer of inorganic clay of low to medium plasticity. N values of 1 per foot were found classifying the clay as very soft. Moisture contents determined ranged from 32% to 46%.
- g) **GREY BROWN SILTY CLAY:** Underlying the light brown silty clay a grey brown silty clay was encountered. The odd grit and pebble which were actually small pieces of black shale were found throughout the layer. As mentioned in the geology of the area, these deposits are mainly lacustrine clays. The deposit was found to be extremely homogeneous with very little change in colour with depth. N values of about 5 were found throughout. The natural moisture contents remained relatively constant at about 30%. The clays described in sections (f) and (g) may be of the same deposit but this is not relevant.

Information from other work done in the area indicates that the layer of grey brown silty clay extends to a depth of about 120'± below existing grade. Underlying it is a layer of grey to dark grey silty clay with pockets of silt followed by a very hard grey silt and a very dark brown grey clayey till. It has been established (Report #6074) that the very dark brown grey till is extremely hard with the N values, in every instance, over 100.

9. WATER CONDITIONS

The detailed water level readings are given on the borehole log. The final water level corresponded to the water level in the channel at the time of the investigation which was at elevation ±197', 14 feet below existing grade.

10. OBSERVATIONS AND CONCLUSIONS

The clay deposits as found at the site were found to be normally loaded deposits, which means that these deposits were never subjected to loads greater than the present overburden pressures.

Foundations of any type (apart from the end bearing type of pile) placed upon this material will be subjected to considerable settlements.

Spread footings have been considered but are thought to be uneconomical in this situation because of the quantity of soft, wet material that would have to be excavated.

The most economical type of foundation is, in our opinion, a friction pile foundation. The bearing value of the friction pile foundation may be calculated using the following expression:

$$Q \text{ total} = A_s \times S_u \text{ (for cohesive material)} \quad \text{(Equation 1)}$$

where A_s - "skin" area of 1 pile

S_u - undrained shear strength of the soil

the value obtained gives the "ultimate" bearing value of a single pile.

The following table gives the calculated safe bearing values of a single pile of 1'0" diameter for various embedded lengths of the pile in cohesive material. A factor of safety of 2 is used in the calculations.

Embedded Length of Pile (ft.)	Skin Area ₂ (ft. ²)	Average S_u (KSF)	Q (FS=2) (Kips)
10	31.4	.310	5.0
20	62.8	.450	14.0
30	94.2	.600	28.0
40	123.6	.740	46.0
50	157.0	.900	71.0
60	188.4	1.040	98.0

Values obtained for the bearing capacity of friction piles in the top 20 feet of granular are small compared to those for clay and may be neglected.

The bearing capacity of a pile group has also been considered. The allowable bearing value of such a type of foundation is given by the following expression:

$$\text{allowable} = \frac{N_c \times S_u}{F.S.} + P_o \quad \text{(Equation 2)}$$

where N_c - bearing capacity factor which depends on foundation depth (D) and foundation width (B)

S_u - the undrained shear strength at the pile tips

F.S. - factor of safety

P_o - overburden pressure

A typical table showing the calculations made for a pile group of 1 row of 6 piles (total 6 piles) having a foundation area of 1x21 ft. = 21 sq. ft. (c.c. of piles 4xd, where d - diameter of the pile) follows. A factor of safety of 2 was used in the calculations.

Embedded Length of Pile (ft.)	D/B	N_c	S_u (KSF)	P_o (KSF)	Gall. (KSF) (FS=2)	Q (K)	q/pile (k)
10	10	7.5	.390	1.77	3.23	67.9	11.3
20	20		.540	2.30	4.325	91.0	15.2
30			.680	2.83	5.38	113.0	18.8
40			.830	3.36	6.47	136.0	22.7
50			.980	3.89	7.57	159.0	26.5
60	20	7.5	1.12	4.40	8.61	181.0	30.2

As may be seen, to obtain a bearing value of a single pile of a pile group, the allowable bearing value per 1 sq. ft. of deep footing is calculated (gall.), then multiplying by the foundation area of pile group (in the above case 21 sq. ft.) the total bearing value of the pile group is obtained (Q total) and finally dividing by the number of piles (6) the allowable bearing value of a single pile is obtained.

As may be seen (Figure 5), the line giving the allowable bearing values of a single pile (say of a pile group of 1 row of 6 piles = 6 piles) is given by two curves; the upper curve extending from a bearing value of 0 Kips for an embedded length of 0 ft. to a value of 15.8 Kips for an embedded length of 21.5 ft. This curve was obtained on the basis of equation 1 given in this paragraph. From a value of 15.8 Kips, the allowable bearing values of a single pile for 6 pile foundation group is then given by the second curve as calculated from the equation 2. This curve gives a bearing value of 25 Kips for an embedded length of 45 feet for the friction pile foundation.

The above bearing value holds only for this configuration. When the final design is chosen, it may be desirable to recalculate the allowable bearing capacity.

If the centre to centre spacing of the piles is greater than 6 or 7 times the pile diameter, the action of the pile group may be neglected and bearing capacities calculated from equation 1 may be used.

Actually, the "true" bearing value of the pile group will be somewhere between the values as obtained from equations 1 and 2 given above. But due to the uncertainties involved, we feel that the bearing values for design purposes should be taken as given on the graph by the solid line.

It is recommended that the "embedded length" of the pile should be taken from the upper surface of the clay layer. Any contribution of the sand layer should be neglected since it is so small.

Based on work done in the area (Report #6074) an end bearing type of pile will have to extend to a layer of clayey till believed to be at a depth of about 135'. This appears to be uneconomical for this job because of the small size of the bridge.

A settlement analysis was carried out for a friction pile foundation assuming a pile length of 45 feet with 1 row of 6 piles and a cross sectional area of the foundation of $1 \times 21 = 21$ sq. ft. and a bearing value of a single pile of 25 Kips.

Assuming the unit vertical pressure of 7.15 ksf will act at $2/3$ of the pile length (i.e. at 30 feet below the top of the clay) a settlement of 2.73 inches was calculated using the uncorrected laboratory consolidation curve. This, in our opinion, will be a possible maximum settlement.

The time-settlement relationship was found to be as follows:

Degree of Consolidation in %	Time Required in Years
10	3.04
20	12.4
30	27.9
40	49.7
50	77.5
60	113.0
70	159.0
80	224.0
90	335.0

As may be seen, the settlement of the cohesive deposits, due to the applied loads, will be extremely slow. Fifty percent of the consolidation will take place in about 77 years. Assuming that the life of the bridge structure will be 50 years only, about 40% of the settlement or about 1.1 inches will have taken place in this period.

Routine maintenance should allow for settlement of this magnitude.

BEVO ASSOCIATES LTD.

A. Richard Markell

A. Richard Markell,
Engineering Geologist.

ARM/vs

APPENDIX I

Laboratory Test Results

JOB NO. 70-F117

SHEAR STRENGTH RESULTS

Figure 1

BH #	Sa. #	Depth (ft.)	Natural Moisture Content (%)	Void Ratio	Strain (%)	γ dry (pcf)	γ wet (pcf)	Shear Strength (PSF)
1	7	20'-22'6"	48	1.28	5	74	109	260
1	10	33'-35'	31	.875	7	90	118	622
1	12	40'-42'	35	1.01	11	84	113	547

ATTERBERG LIMITS

BH #	Sa. #	Depth	W %	Liquid Limit	Plastic Limit	Plasticity Index
1	7	20'-22'6"	48	51	24	26
1	8	23'-25'	46	50	22	28

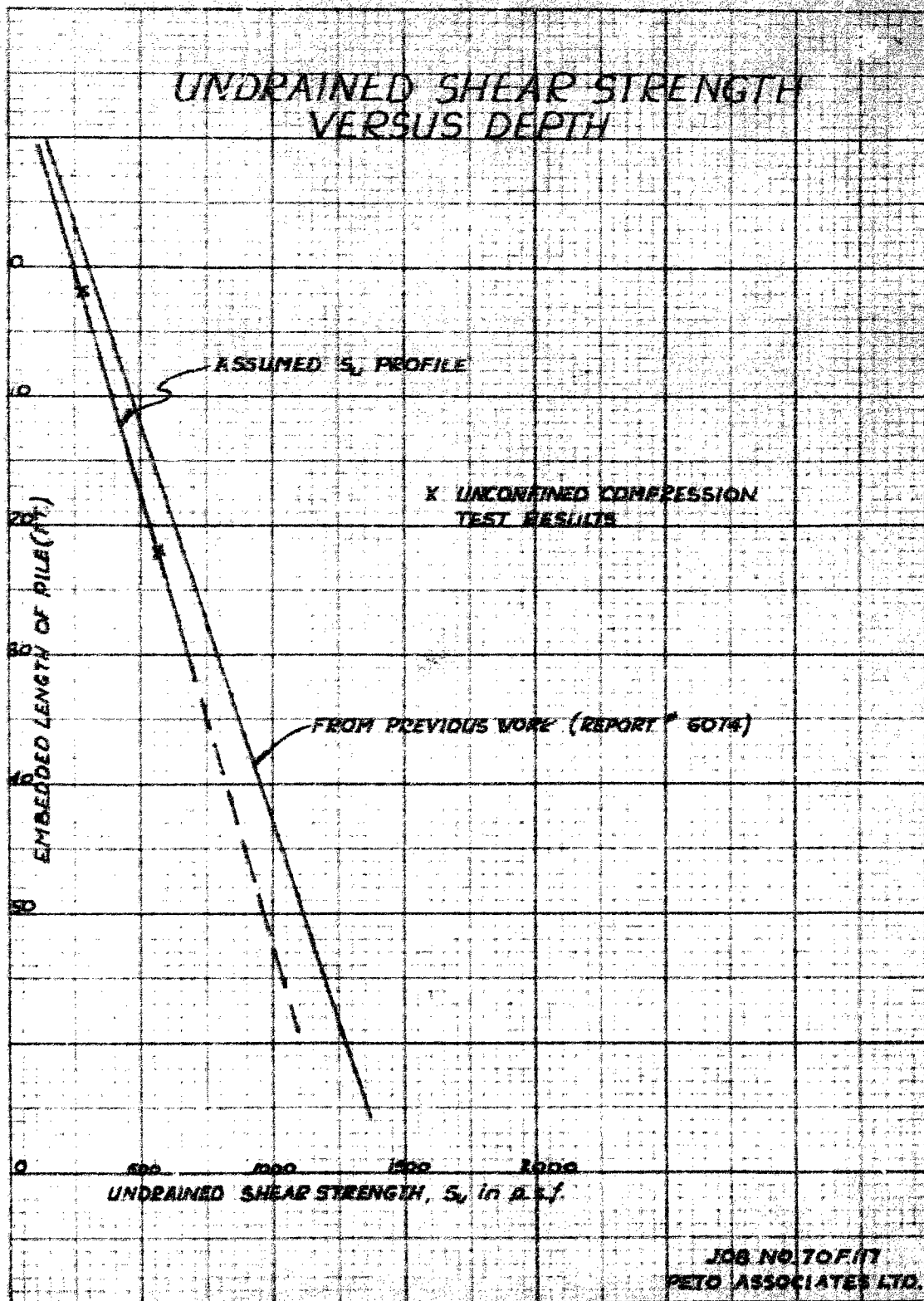


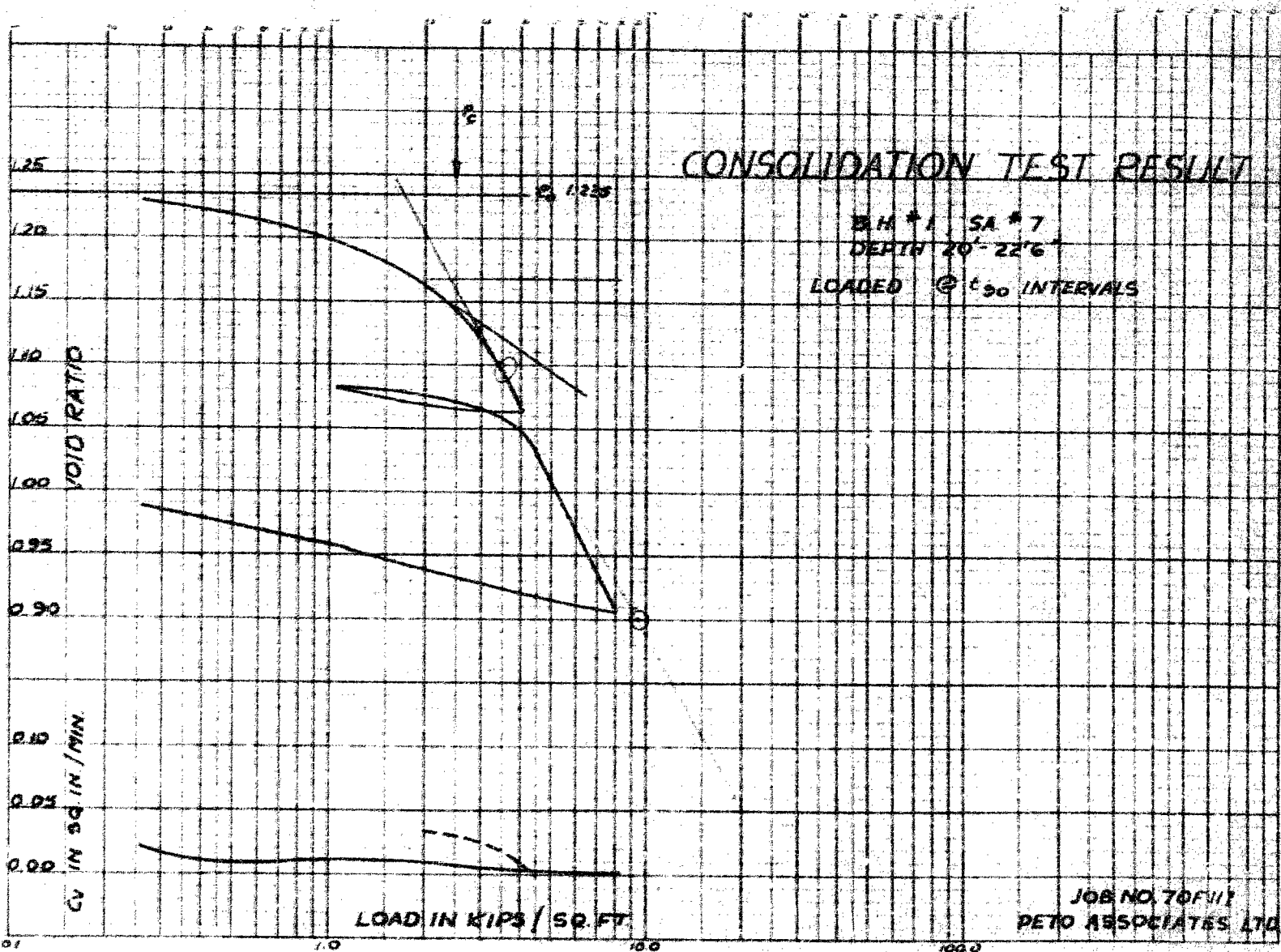
FIGURE 1

LA SUDDEAL

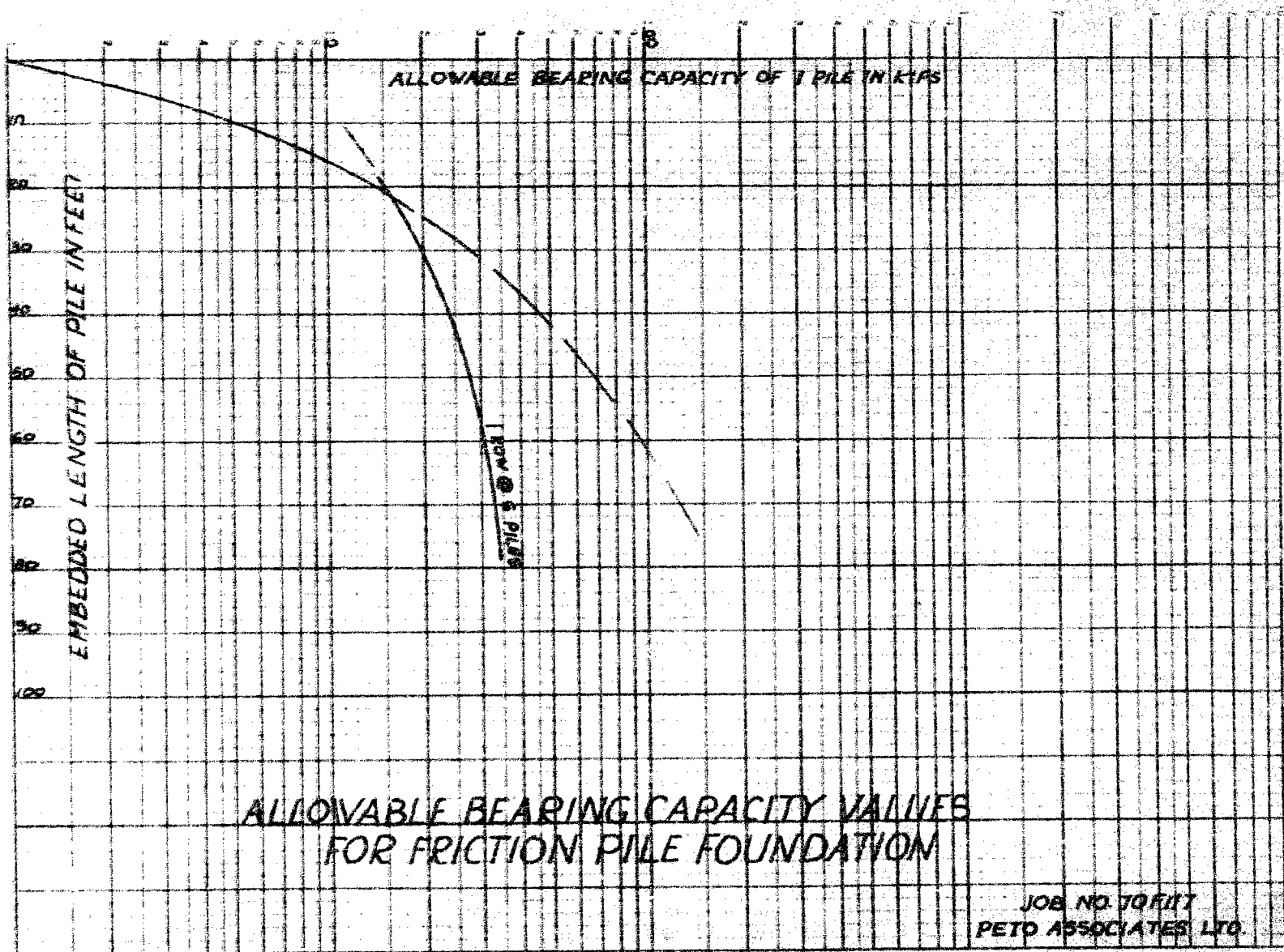
DEPTH	10'-12'	REMARKS	GREY SILTY FINE SAND
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FIG. 3





JOB NO. 70F111
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LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS / FT.</u>	<u>C LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>	
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4	
SOFT	2 - 4	250 - 500	LOOSE	4 - 10	
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30	
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50	
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50	
HARD	> 30	> 4000			
W.T.P.L.	WETTER THAN PLASTIC LIMIT		D.T.P.L.	DRIER THAN PLASTIC LIMIT	
A.P.L. ABOUT PLASTIC LIMIT					

TYPE OF SAMPLE

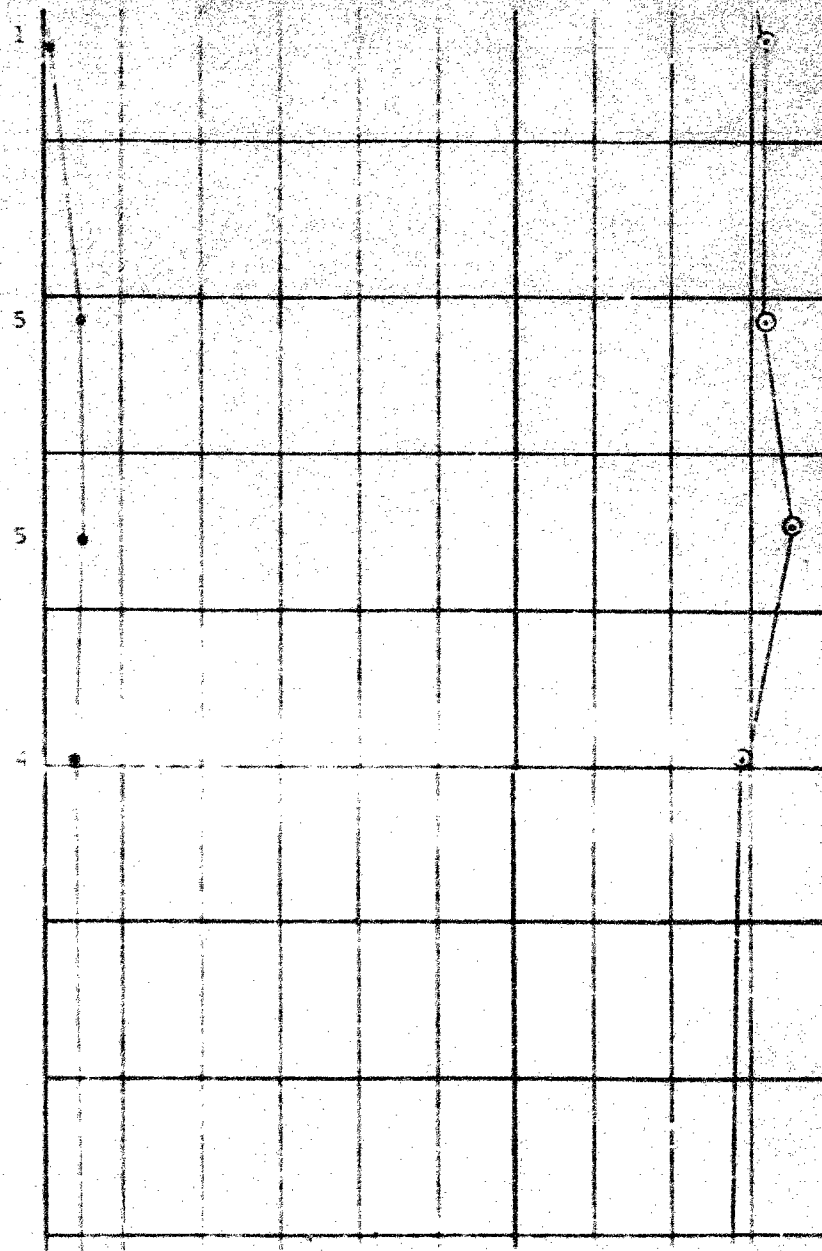
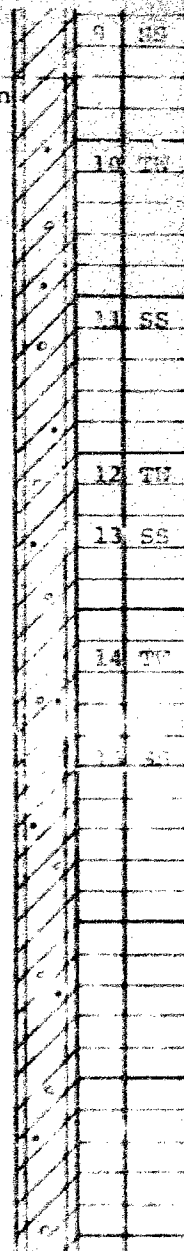
SS	SPLIT SPOON	TW	THINWALL OPEN
WS	WASHED SAMPLE	TP	THINWALL PISTON
SB	SCRAPER BUCKET SAMPLE	OS	OESTERBERG SAMPLE
AS	AUGER SAMPLE	FS	FOIL SAMPLE
CS	CHUNK SAMPLE	RC	ROCK CORE
ST	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Su	UNCONFINED COMPRESSION	LV	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	FV	FIELD VANE
Su	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

Change to gray brown
silty clay, soft
black grits &
les

odd large pieces
of black shale
found



water 14'6"
after 2 min.
14'3"
after 2 min.
14'
after 2 min.
13'9"
after 2 min.
13'6"

hole 96'
PX at 20'
water 13'
no bailing

Sept. 23/70
time 8:00 AM
water 11'3"

BH Terminated at
74'0"

Cone penetration
from 74'0" to
97'0"

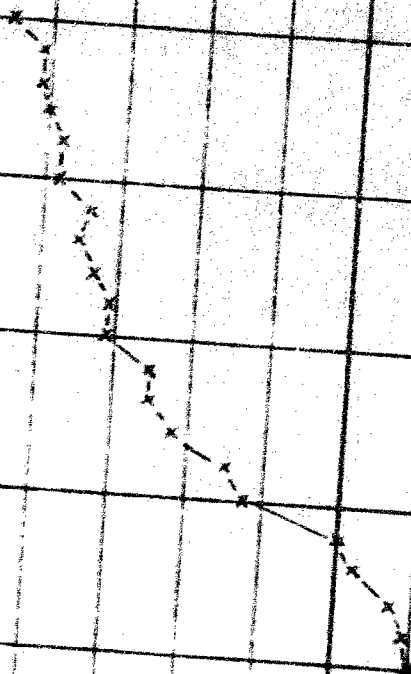
16 SS

4

3

160 *

155 *





PETO ASSOCIATES LTD.

RECORD OF BOREHOLE NO. 1

CONSULTING SOIL ENGINEERS

JOB NO. 70F117

JOB NAME McWilliam Bridge Reconstruction

TECHNICIAN J. J.

BORING DATE Sept.

CLIENT Township of Sebring d/o Tougham & Case

ENGINEER J. J.

GROUND ELEV. 93.55

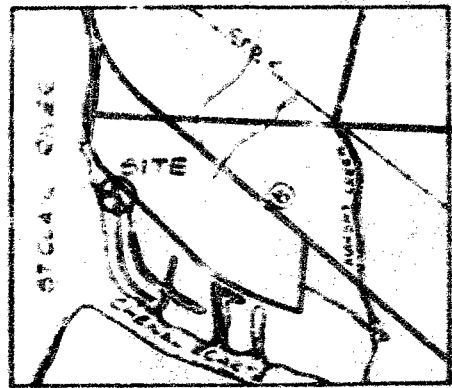
BOREHOLE TYPE Standard Pit

TYPED BY J. J.

SOIL PROFILL		SAMPLES			DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT	LIQUID LIMIT _____ WL PLASTIC LIMIT _____ Wp WATER CONTENT _____ W	REMARKS
DEPTH ELEV	DESCRIPTION	LEGEND	NUMBER	TYPE			
11.0'	TOPSOIL-Bk silty loam		1	SS	3		HX at 9' SS 9' after pulling SS water 3'5" after 2 min. 5'6"
11.5'	SILT-Gr or sandy silty some organic moist		2	SS	2		
12.0'	PEAT-Bk organic clayey silt, 11.0' moist		3	SS	13		after 2 min. 5'5"
12.5'	SILT-Grey brown clayey sandy silt soft, some roots		4	SS	5		
13.0'	SAND-Bk silty fine wet		5	SS	7		Sept. 22/70 time 8:00 AM HX at 9' water 1'5" overnight
13.5'			6	SS	7		
14.0'			7	SS			HX at 10' SS 14" water after 2 min. 9' after 2 min. 8'5" after 2 min. 5'2" after 2 min. 7'8" after 2 min. 7'4"
14.5'	CLAY-Lt brown silty clay, soft		8	SS			
15.0'							HX 15' 10"

HOLE #1
E. 98.6

6'



KEY PLAN
SCALE 1/25000

53'
RIVERSIDE DRIVE

PLANK WALLS

CASPIAN LANE

EXISTING
BRIDGE

TOO OF BANK



SITE PLAN
SCALE 10' TO 1"

JOB NO 70-F117
PETO ASSOCIATES LTD
NOV. 1970 *EX*