

67-F-235M

EAST MARSH DRAIN

MERSEA

BA 2736
S.F. 2-262

E. M. PETO ASSOCIATES LTD.

SUB-SURFACE INVESTIGATION
EAST MARSH DRAIN

for

TOWNSHIP OF MERSEA
c/o C.G. Russell Armstrong Assoc. Ltd.
Windsor, Ontario

DISTRIBUTION:

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JOB NO. 67 F250

DECEMBER, 1967

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE.. 67 P250

1287 caledonia road.

TORONTO 12, ONTARIO

Telephone: 789-1128

December 29, 1967.

Township of Mersea,
c/o C.G. Russell Armstrong Associates Ltd.,
76 University Avenue West,
Windsor, Ontario.

Attention: Mr. E.O. LaFontaine, P.Eng.

Dear Sirs:

Re: Sub-surface Investigation
East Marsh Drain
Township of Mersea

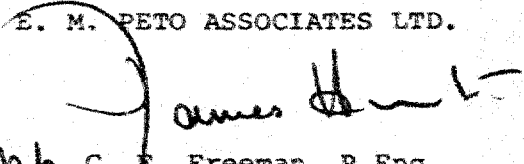
We are pleased to submit herewith six copies of our report on this investigation.

We believe that it should be possible to place the bridge structure on spread footings at about 8 ft. below existing ground surface, provided that settlement of approximately 2 inches can be tolerated. However, we have included, for your consideration, some thoughts on both friction and end-bearing piles.

Whilst we believe this report to be complete within our terms of reference, we should be happy to discuss any queries which might arise from it.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

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

ANSB/jw

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1. INTRODUCTION

1.1 Purpose and Background: It is proposed to replace the existing single lane, beam and plank bridge by a wider, modern structure better capable of carrying today's traffic. The bridge carries Concession B road over the East Marsh Drain, a man-made channel used to drain the surrounding land which lies below the level of Lake Erie.

The investigation reported herein consisted of two boreholes put down on opposite sides of the channel in order to evaluate conditions for the foundations of the new structure.

1.2 Authority: The authority to proceed with this work is contained in a letter dated November 10th, 1967 from E.O. LaFontaine of C.G. Russell Armstrong Associates Ltd., Consulting Engineers, acting on behalf of the Client, the Township of Mersea.

2. FIELD WORK

2.1 Locations and Elevations: The elevations of the boreholes were taken relative to a temporary bench mark, which was a nail established in a pole supporting overhead electricity wires. This was located some 20 ft. south-east of the existing east bridge abutment. The nail was marked with a red flag and was taken to have an elevation of 100 ft. The relationship between this and geodetic datum is not known. The lake level is thought to approximate to geodetic elevation 572 ft.

2. FIELD WORK - cont'd

It had been proposed to locate the holes diagonally opposite each other, however, access problems resulted in both holes being carried out on the west side of the bridge, although on opposite sides of the drainage channel.

2. Field Work: Two boreholes were put down. Initially these were planned to terminate at 40 ft. and to be augmented by two cone probes to 40 ft. at the remaining diagonal locations. In fact this course of action did not prove possible and the two holes were extended from 40 ft. until grinding refusal to the augers was reached at depths of 54 ft. 6 inches and 56 ft. 9 inches in boreholes 1 and 2 respectively.

The holes were put down by one of our field crews using a self-propelled, track mounted, 3 inch diameter, continuous flight auger. Standard penetration tests were carried out at regular intervals to a depth of 40 ft.

3. SUB-SURFACE CONDITIONS

At depths greater than 20 ft. the two boreholes were very similar, however, the upper 20 ft. exhibited important differences. In borehole 2 there is a stiff crust which, in borehole 1, is virtually non-existent, being replaced by stiff cohesive fill, having some organic content.

3. SUB-SURFACE CONDITIONS - cont'd

3.1 Topsoil: At borehole 1 only, on the western or landward side of the bridge, was found 3 ft. 4 inches of rich, black, loar organic topsoil in a loose condition. This material appears to be well suited to agricultural purposes.

3.2 Fill: In borehole 2, from ground surface to a depth of 13 ft. 0 inches was found grey-brown to grey silty clay fill with layers or pockets of organic matter. This material is initially in a stiff state having "N" values of 10 and 11, and softens to firm with depth ("N"=6). It is thought that this material was probably placed at the time the east drainage channel was constructed.

3.3 Grey Brown Silty Clay Till: Beneath the strata described in paragraph 3.1 and 3.2, was found, in both boreholes, a grey-brown silty clay till, wetter than the plastic limit. In borehole 2, this stratum exists from 13 ft. 0 inches to 18 ft. 6 inches. Although no seepage seams were observed, it appears that the upper part of this weathered till is softened and has a higher water content than the same stratum at a little greater depth. The two penetration tests in this stratum in borehole 2 gave values of 13 and 28 respectively. Similarly, in borehole 1, the upper portion is soft, having "N" = 2 to 7 after which the average "N" is about 30 to 16 ft. 0 inches. The softening is probably associated with the drainage channel water level, noted at elevation 90 ft.

3. SUB-SURFACE CONDITIONS - cont'd

- 3.4 Grey Silty Clay Till: This is found below 16 ft. 0 inches in borehole 1 and 18 ft. 6 inches in borehole 2 and is the principal stratum found. The grey-brown material above it is the weather or oxidised upper portion of the same material.

This stratum has an average "N" value of 8, which allows its consistency to be described as stiff to firm. It was found down to 54 ft. 6 inches and 56 ft. 9 inches respectively, at which depths refusal to the augers was encountered.

- 3.5 Refusal Stratum: The stratum encountered at elevation 42 ft. to 43 ft. was not proved, but from the fact that it was found at the same elevation, from the firm grating nature of refusal and from the gas release and oil film found in borehole 1, it is believed that the refusal is on bedrock. Information available on the geology of the area indicates that bedrock can be expected to be limestone.

- 3.6 Water Conditions: No seepage was observed in either hole and both were open and dry on completion. Borehole 1, which was drilled first, was also observed to be dry and open on the day following its completion.

4. LABORATORY TESTING

No laboratory testing was undertaken other than the determination of natural moisture content on samples recovered from each standard penetration test. These values are plotted on the appended borehole logs.

5. TYPE OF FOUNDATION

5.1 Spread Footings: If spread footings were used, bearing pressure would have to be limited to $1\frac{1}{2}$ tons/sq.ft. In this case, in the area of borehole 2, the fill would have to be excavated to 13 ft. 0 inches and a granular mat laid on the softened surface of the crust. Well graded granular material could then be compacted in rises not exceeding 9 inches finished height until the finished surface is a distance equal to the proposed abutment footing width above depth 17 ft. 6 inches. In this way the stresses reaching the lower clay would be limited to about .83 tons/sq.ft. and consolidation settlement would be approximately 2 inches per 10 ft. width of abutment within the likely design range. If it is possible to restrict the abutment width to 10 ft. or less, then the granular fill could be brought up to 7 ft. 6 inches below existing ground level and the abutment founded upon it. In this way settlement of the natural ground could be reduced to negligible proportions. For the western abutment (borehole 1) placement at depth 8 ft. in the crust would allow 2 tons/sq. ft. to be used on footings up to 8 ft. in width with a maximum predicted consolidation settlement of 2 inches. Abutment footings should be kept to the minimum width consistent with the ability to

5. TYPE OF FOUNDATION - cont'd

carry the required loads. When the design loads are known, it will be possible to proportion the footings at appropriate elevations to minimize risk of differential settlement.

5.2 Friction Piles: Provided that it is not intended to place significant quantities of fill to raise approach embankments, it would be possible to use friction piles. The use of fill results in consolidation settlement, causing negative skin friction or downward drag on the piles.

A shear strength of 1000 lbs/sq.ft. could be ascribed to the softer lower clay below about 13 ft. in depth. This figure is taken from the correlation between "N" values and shear strength given by Terzaghi and Peck ("Soil Mechanics in Engineering Practice", page 430) and is substantiated by laboratory findings on similar material from other job sites. A value of 70% of cohesion is taken for adhesion, following Tomlinson (1957). Hence the ultimate load on an individual pile is given by:

$$Q_{ult} = .7 \times A_p \times l \times s.l. + 9 \times A_e \times s_e$$

where A_p = peripheral area per unit length
 l = effective embedded length
 s = shear strength average over length l
 A_e = end area
 s_e = shear strength at pile toe

A factor of safety of . . . could be used to reduce the ultimate load to the working load. In addition notice should be taken of the reduction

5. TYPE OF FOUNDATION - cont'd

in allowable load when the piles are sufficiently close to act as a group. This occurs when the centre to centre spacing becomes less than 8 times the pile diameter. Spacing should not be less than $2\frac{1}{2}$ diameters, centre to centre, at which spacing a square grid has an allowable load of 65% of the sum of the constituent individual pile loads.

Ignoring any effects above elevation 80 ft. and assuming 20 ft. of effective penetration below that depth, an individual 19 inch diameter pile has an allowable working load of 18.1 tons. Thus, ten such piles under a 25 ft. x 6 ft. cap could be expected to have a working load of the order of 135 tons.

The use of friction piles would involve some settlement, whose magnitude would depend on the length of the piles and the width of the abutment. It is thought that if the piles need to penetrate below elevation 60 ft. in order to develop enough capacity, it would be better to use end bearing piles taken to supposed bedrock at elevation 42 ft.

5.3 End Bearing Piles: End bearing piles could be used to transfer the load down to supposed bedrock at 54 ft. to 57 ft. below existing ground surface. If the bedrock is massively bedded intact limestone, a bearing capacity of 40 tons/sq. ft. will be available. However, in the event of the use of this type of pile, one of the piling formulae should be used to determine the prescribed acceptable set for the last inch of driving, or a field loading test should be carried out.

5. TYPE OF FOUNDATION - cont'd'

Should end bearing piles be selected, it is thought that a small displacement driven pile such as a steel "H" pile might be used, or alternatively a Johnson Herkules pile.

It would be quite possible to use a pre-bored cast-in-place concrete pile, but it is thought that this would probably be more expensive in the prevailing soil conditions.

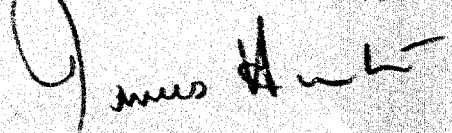
6. SUMMARY

It would be possible to support the structure on either spread footings, or friction piles, if settlement of the order of 2 inches were acceptable. If settlement were an important criterion, end bearing piles to assumed bed-rock at a depth of 54 to 57 ft. might be the favoured solution.

No ground water problems are envisaged and no unusual constructional difficulties anticipated.

Should spread footings be adopted, the question of scour should be examined, although this is not thought to present a serious problem in a ditch of this nature.

E. M. PETO ASSOCIATES LTD.



pb A. N. S. Beaty, P.Eng.
Senior Soils Engineer

ANSB/jw

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 (DPR) -- 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:--

CONSISTENCY	N' BLOWS/FT.	c LB./SQ. FT.	DENSENESS	N' BLOWS/FT.
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

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H.	SAMPLE ADVANCED HYDRAULICALLY	
	P.M.	SAMPLE ADVANCED MANUALLY	

SOIL TESTS

Q _u	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Q _{cu}	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q _d	DRAINED TRIAXIAL		

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_c	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
C_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{C_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
C_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_r	SENSITIVITY

GENERAL

π	≈ 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e \sigma$ OR $\ln \sigma$	NATURAL LOGARITHM OF σ
$\log_{10} \sigma$ OR $\log \sigma$	LOGARITHM OF σ TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
e	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
ρ	ANGLE OF SLOPE TO HORIZONTAL

JOB NO. 67 F250

JOB NAME East Marsh Drain

TECHNICIAN E.S.

BORING DATE Nov. 27/67

CLIENT Township of Mersea, c/o C.G. Russell Armstrong Assoc. Ltd.

ENGINEER A.N.S.B.

GROUND ELEV. 96.4

BOREHOLE TYPE 3" Flight auger

TYPED BY J.W.

SOIL PROFILE			SAMPLES		DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT					LIQ/10 LIMIT _____ W _L PLASTIC LIMIT _____ W _P WATER CONTENT _____ W			REMARKS	
DEPTH ELEV.	DESCRIPTION	LEGEND	NUMBER	TYPE	BLOWS/FOOT	SHEAR STRENGTH C _u LB/SQ. FT					W _P W W _L WATER CONTENT %			
						10	20	30	40	50	10	20		30
0'0"	Black organic topsoil. Moist, very loose		1	SS	2									
3'4"	Brown to light grey brown silty CLAY TILL with grit to pebble content and hair root content to organic silt pockets Above 7' depth wetter than plastic limit and firm grading at 7' to drier than plastic limit and hard. (Softening at transition to grey)		2	SS	7									
			3	SS	21									
			4	SS	39									
			5	SS	32									
16'2"			6	SS	14									
	Grey silty CLAY TILL Wetter than plastic limit Firm to stiff		7	2"mm	Push									
			8	SS	9									
			9	2"mm	Push									
			10	SS	9									
	After 41'6" depth no further samples were taken and just the depth of refusal on augers was proved. This refusal occurred at 54'6" along with the tapping of a gas pocket immediately above assumed bedrock refusal.		11	2"mm	Push									
54'6"	Refusal at 54'6"													No seepage seams noted. Hole dry and open on com- pletion. Next day hole open and dry.

JOB NO. 67 P250

JOB NAME East Marsh Drain

TECHNICIAN

E.S.

BORING DATE Nov.28,29/67

CLIENT Township of Mersea, c/o C.G.Russell Armstrong Assoc.

ENGINEER

A.N.S.B

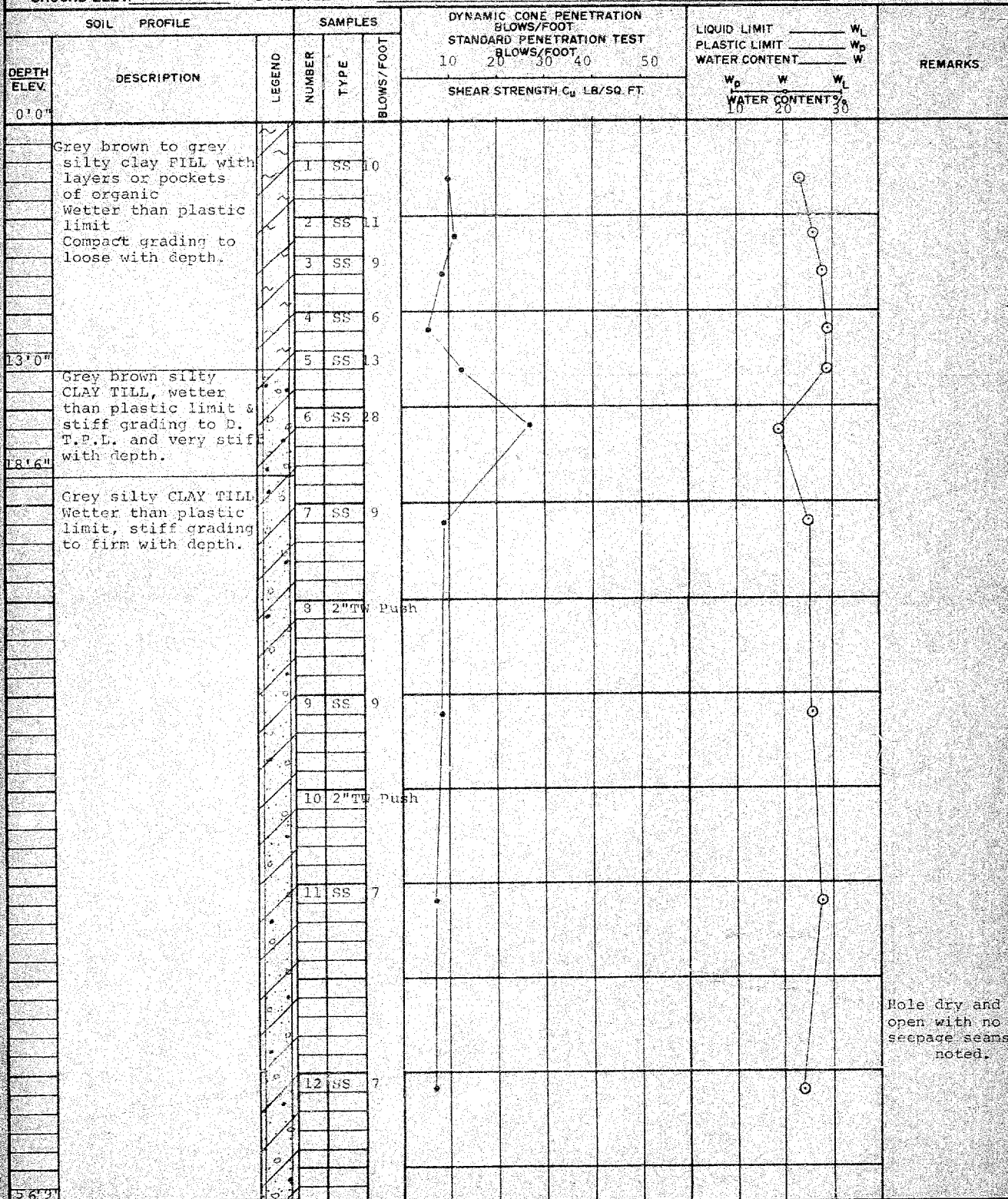
GROUND ELEV. 99.5

BOREHOLE TYPE

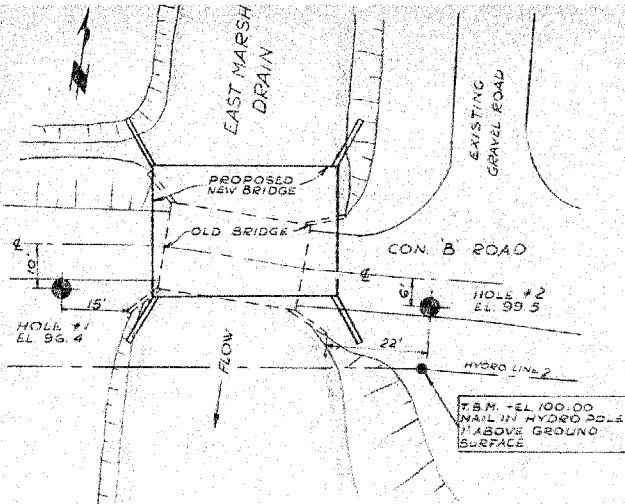
3" Flight auger

TYPED BY

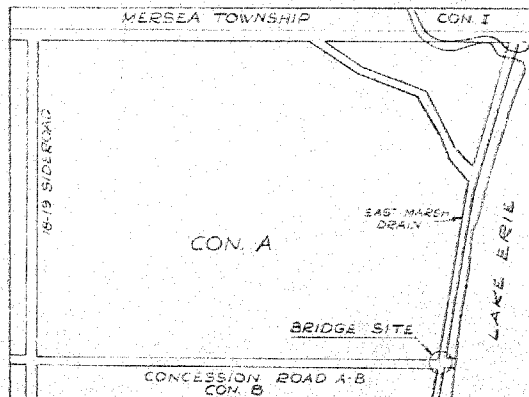
T.W.



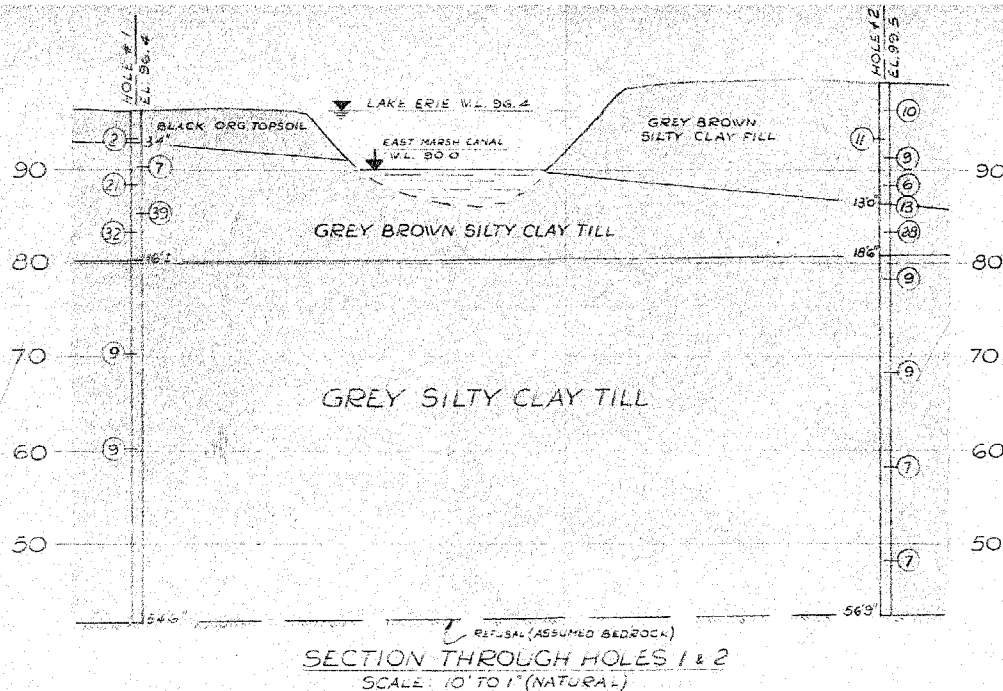
Refusal at 56'9"



SITE PLAN
SCALE: 20' TO 1"



KEY PLAN
SCALE: 1" = 20 CHAINS



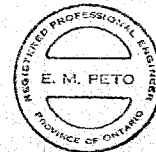
LEGEND

- BOREHOLE
- (7) BLOWS/FOOT

NOTE:

SEE BOREHOLE LOGS FOR
COMPLETE SOIL DETAILS.

NOTE: The actual soil stratification has been verified from data obtained at the borehole locations only. The inferred contacts shown are based on geological evidence and these may vary from those shown between borings.



TOWNSHIP OF MERSEA			
c.g. RUSSELL ARMSTRONG ASSOCIATES LTD.			
PROPOSED BRIDGE OVER EAST MARSH DRAIN			
PREPARED BY e.m. peto associates ltd.			
JOB NO. 67-F250	DATE DEC. 1967	DWN BY K.K.	CHECKED BY Ans