

68 - F-224 M

DE E RIVER

BRIDGE

MUSKOKA

BA 2841
Site 42-29

PETO ASSOCIATES LIMITED

DEE RIVER BRIDGE
MUSKOKA

for

TOWNSHIP OF WATT, DISTRICT OF MUSKOKA
c/o GREER GALLOWAY & ASSOCIATES LTD.

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JOB NO. 68-F67

MAY, 1968.

*Received
May 13/68*



PETO ASSOCIATES LTD.
CONSULTING SOIL ENGINEERS
1287 Caledonia Rd. Toronto 19 · Ontario · Phone (416) 789-1126

JOB NO. 68-F67

May 6, 1968.

Township of Watt,
District of Muskoka,
c/o Greer Galloway & Associates Ltd.,
973 Crawford Drive,
Peterborough, Ontario.

Attention: Mr. C.P. Maitra, P. Eng.

Dear Sirs:

Re: Dee River Bridge,
Muskoka.

In connection with the above project, the following is our report of the work carried out and our observations and recommendations for the construction of a bridge at the site.

Authority to carry out the investigation was received from Mr. C.P. Maitra, P. Eng., in a letter dated April 2nd 1968. It is proposed to construct a bridge at an existing bridge site over the River Dee in the Township of Watt, District of Muskoka. No details are available as yet for the design of the bridge or the anticipated loads.

Two boreholes, as shown on the attached site plan, were sunk by standard rig to depths of 85 ft. 0 ins. and 75 ft. 8 ins. with dynamic core penetration resistance being measured in borehole 1 from 85 ft. 0 ins. to refusal at 108 ft. 0 ins.



Disturbed samples were obtained from the standard penetration test where taken, and shelby tube samples taken and vane tests carried out as indicated on the appended borehole logs. A careful check was kept on ground water conditions during drilling operations and subsequently until leaving the site.

In addition to the moisture contents determined for each standard penetration test sample, liquid and plastic limits were carried out for two samples with corresponding unconfined compression tests from the same samples. An additional unconfined compression test was also carried out. The laboratory results and geotechnical properties determined are presented in Table 1.

In general, the boreholes encountered similar conditions. Fill was present in both boreholes to depths of 7 ft. 0 ins. and 6 ft. 0 ins. respectively. Borehole 1 showed very soft organic silt to underly fill from 7 ft. 0 ins. to 11 ft. 0 ins. before a grey brown becoming grey silty clay was encountered. This material was also encountered in borehole 2 immediately underlying the fill. This persisted to depths of 41 ft. 6 ins. and 41 ft. 0 ins. respectively. This clay material changed with depth to an interlayered grey and reddish brown slightly silty clay. The material was sensitive, the sensitivity decreasing with depth. *the clay.* The shear strength of this material also increased with depth with a corresponding decrease of moisture content. The maximum shear strength obtained from the vane tests taken in borehole 1 was 1200 p.s.f. at a depth of 21 ft. 6 ins. with a minimum of 500 p.s.f. at a depth of 12 ft. 0 ins.

The unconfined compression tests on samples taken from borehole 2 showed the shear strength to be of the order of 250 p.s.f. It would appear that these unconfined samples had suffered disturbance owing to these low shear strength values compared to the vane test results.

The standard penetration values obtained bear no relation to shear strength of the material as the blows disturb the structure of the material on impact. Below the clay material a transitional zone of interlayered clay and sand was encountered in both boreholes. This material was very varied in composition with thin layers of silty sand, silty clay and clayey silt being present. The clay where present was similar to the overlying material. This zone persisted to depths of 51 ft. 6 ins. and 50 ft. 0 ins. in boreholes 1 and 2 respectively, when a grey slightly silty very fine sand was encountered. Borehole 1 was sampled to a depth of 67 ft. 6 ins. and then washed and cased to a depth of 85 ft. 0 ins. with no change in material. Below this depth dynamic cone penetration values were taken to 108 ft. 0 ins. when virtual refusal was obtained. The density of this material varied between very loose and compact with a final reading of dynamic cone resistance of 168 blows. This may be a change in density to a very dense material or a boulder.

Borehole 2 was sampled to 61 ft. 6 ins. when dynamic cone penetration resistances were measured to refusal at 74 ft. 9 ins. Core drilling was commenced at 73 ft. 4 ins. the difference in depth is assumed to arise as a result of the cone sliding down the boulder which was proved to a depth of 75 ft. 6 ins. Below this depth a standard penetration test was carried out which gave a value of 200 blows for 2 ins. penetration. The sample obtained from this test showed brown coarse sand with some fine gravel, but the high value obtained may also indicate the presence of another boulder.

A perched water table was encountered above the impervious grey silty clay at depths of 4 ft. 0 ins. and 4 ft. 4 ins. in boreholes 1 and 2 respectively. Below the interlayered clay and sand an artesian water condition was encountered which gave a maximum reading of 12 ft. 0 ins. above ground level. This value was obtained in borehole 2. In borehole 1 the value obtained was approximately ground level. This water tended to push the sand up the casing and made drilling operations very difficult. The standard penetration values

will have been affected by this water and the values obtained will have underestimated the density of the sand.

From the results of the boreholes, the only feasible method of supporting a new bridge on this site is by the use of piles. Piles have been used in the existing bridge and it is assumed that these are friction piles set in the clay material. The loads exerted by the existing bridge however, will be much lower than the loadings produced by a new structure unless it is of a similar design. Whilst values of unconfined shear strength as indicated by the vane increase to 1200 p.s.f. the value of unit friction on the pile in the clay can not be counted as to equal to the cohesion i.e. 1200 p.s.f. owing to the fact that this is a sensitive material. Hence, on driving a pile the soil close to the pile will be remoulded and will lose its structure and hence, its cohesion.

In time there may be a regain in strength, but the only way to estimate this reliably is to carry out a pile loading test over an appropriate period of time.

Assuming that the shear strength regained to a value of 500 p.s.f. and the pile adhesion was assumed to be 250 p.s.f., then for a rectangular grouping of 14 piles driven to a depth of 30 ft. in three rows within an area of 30 ft. X 10 ft., then the allowable load would be 150 tons on the pile group with a factor of safety of 2.

Consolidation of the clay may occur, but would depend in the lengths of pile, loads on the pile etc. and estimates of settlement would require consolidation tests. If approach embankments were anticipated above the existing level, depending on the increase in load consolidation of the clay would occur and transfer loading onto the piles with further possible settlement of the piles.

*Not necessarily shear failure
but rather greater load on the piles*

Piles driven through the clay into the sand encounter the artesian water and will, therefore, be subject to uplift pressure. Initial seepage around the pile may reduce skin friction to less than hydrostatic uplift, however, it is anticipated that this would seal owing to sand rising up around the piles and skin friction in the sand would mobilize. A test pile would be necessary to establish definitely the loads to be carried by a pile and the depths to which they could be driven and the length of time required to mobilize skin friction. If the piles cannot be driven to a point bearing refusal on the very dense material indicated in the boreholes, the probable safe load for a one ft. diameter pile will be of the order of 25 tons.

The comments made previously with respect to the placing of embankments also apply here. If the piles can be driven to this depth, then the type of pile and hence, the strength of the pile will determine the load to be carried.

In summary, this is a very difficult site for the location of a bridge, other than the type already in existence.

At the time of the sinking of the first borehole, the approximate river boundary was as shown in the site plan, but this subsided to the boundary shown on the site plan supplied to us, by the finish of the investigation.

Yours very truly,
PETO ASSOCIATES LTD.

DJB/jc

D. J. Belshaw
D.J. Belshaw, P. Eng.

TABLE I

GEOTECHNICAL PROPERTIES

BH #	Sample No.	Depth	W%	L.L.	P.L.	I.P.	Shear(1) Strength p.s.f.	Remoulded Shear Strength p.s.f.	Shear(2) Strength p.s.f.	Failure Strain	γ W
1	Vane test	12'-12'8"					500	75			
		18'0"-18'9"					800	75			
		18'9"-19'6"					600	75			
		21'6"-22'3"					1200	200			
		31'6"-32'3"					1050	200			
2	4	10'-12'	82.1	88.2	30.1	58.1			300	3%	98.6
	7	20'-22'	90.1	84.0	28.3	55.1			230	2%	92.9
	4	10'-12'	82.1						240	10%*	96.2
	9	30'-32'	69.3						255	4%	59.3

1. Vane test

2. Unconfined compression test

* Disturbed sample.

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

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
WS	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_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a 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
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	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_o	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
β	ANGLE OF SLOPE TO HORIZONTAL

e.m. peto associates ltd.

RECORD OF BOREHOLE 107.

Consulting soil engineers

JOB NO. 68P67

JOB NAME 100' Long Bridge

TECHNICIAN

BORING DATE Apr. 16-18

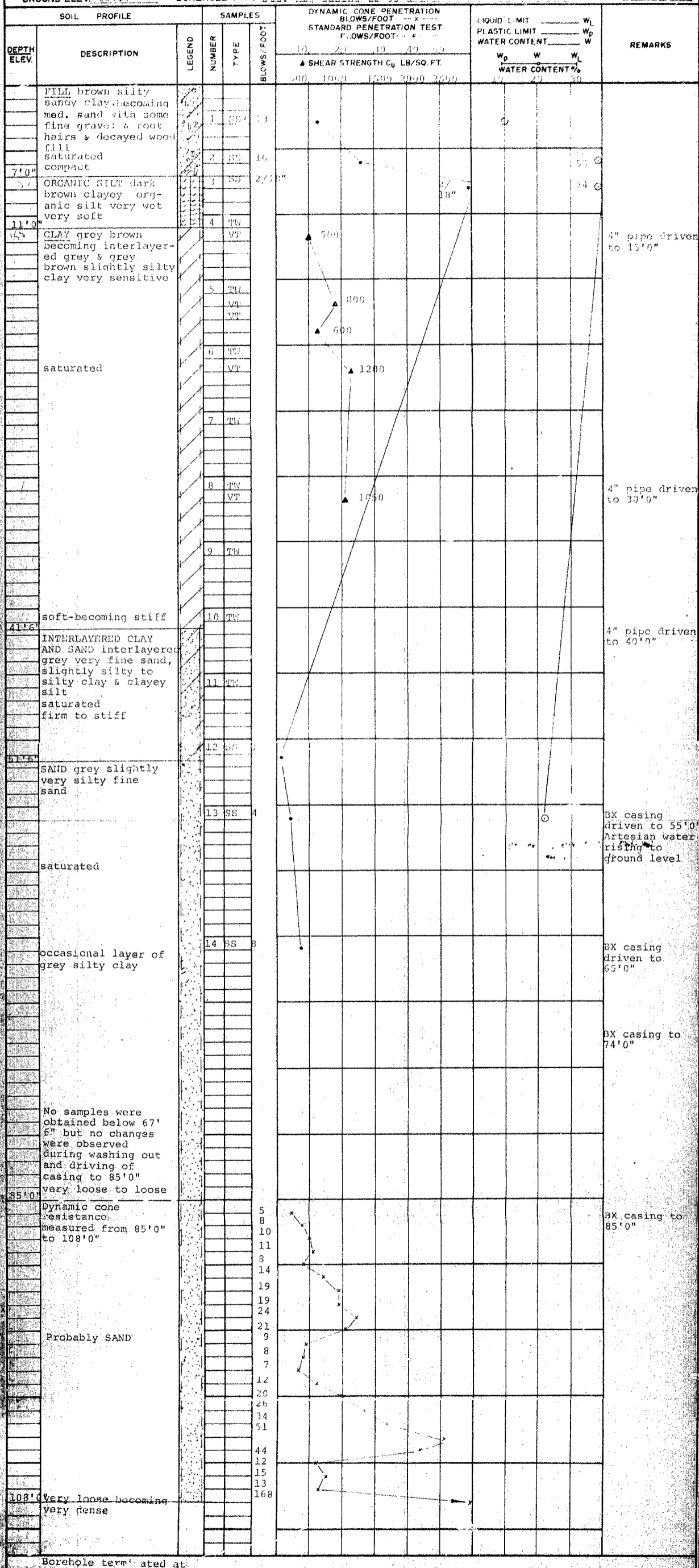
CLIENT TARRANT & SONS, 100' Long Bridge, 100' Long Bridge, 100' Long Bridge

ENGINEER

GROUND ELEV. 169.1

BOREHOLE TYPE 100' Long Bridge

TYPED BY



e.m.peto associates ltd.

RECORD OF BOREHOLE NO. 2

Consulting soil engineers

JOB NO. 68P67

JOB NAME Dog River Bridge

TECHNICIAN WT

BORING DATE Apr. 10, 11, 16

CLIENT Township of Watt, Dist. of Muskoka, c/o GOREY Galloway & Assoc. Ltd.

ENGINEER BH

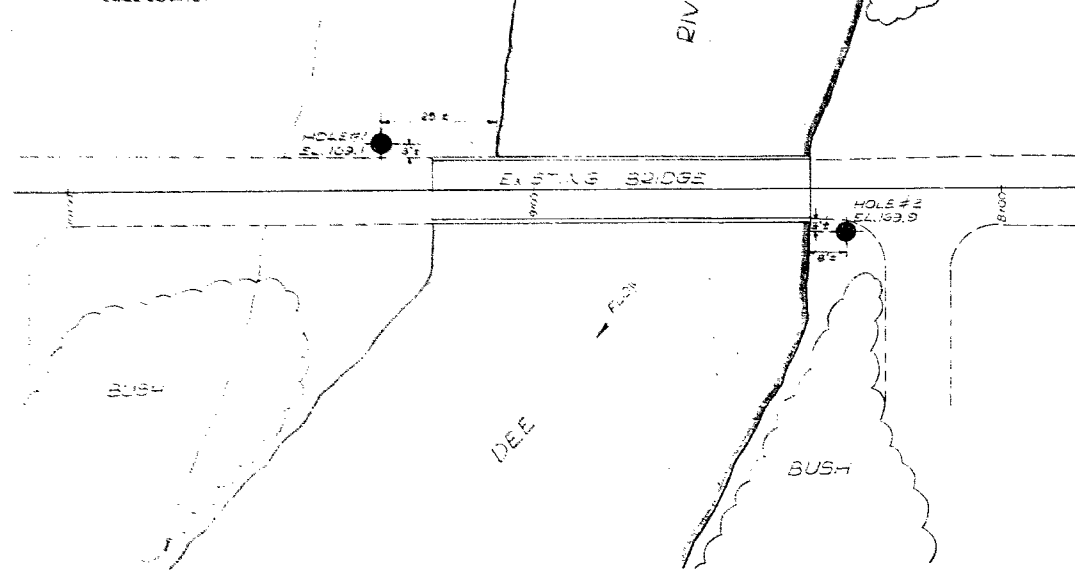
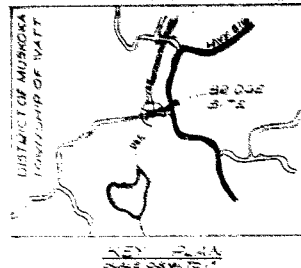
GROUND ELEV. 169.9

BOREHOLE TYPE Standard Rig - Casing to 75'0"

TYPED BY JC

DEPTH ELEV.	SOIL PROFILE DESCRIPTION	LEGEND	SAMPLES		BLOWS/FOOT	DYNAMIC CONE PENETRATION BLOWS/FOOT STANDARD PENETRATION TEST BLOWS/FOOT					LIQUID LIMIT _____ W _L PLASTIC LIMIT _____ W _P WATER CONTENT _____ W			REMARKS
			NUMBER	TYPE		10	20	30	40	50	W _p	W	W _L	
						SHEAR STRENGTH C _u LB/SQ. FT.					WATER CONTENT % 10 20 30			
	FILL mottled grey brown, yellow brown and dark brown clayey silt, organic silt and decayed wood		1	SS	1/12"									
6'0"	CLAY grey silty clay becoming brown slightly silty clay sensitive.		2	SS	3									4" Casing to 5'0" water level 4' 4" when borehole at 5'0"
			3	SS	2									42
			4	SS	-									77
	becoming grey brown in colour becoming interlayered reddish brown & grey slightly silty-clay very sensitive		5	SS	2/20"									30
			6	SS	1/18"									88
			7	TW										casing to 10'0" hole dry
	saturated		8	SS	1/18"									90
			9	TW										84
			10	SS	1/18"									75
	very soft to soft													
41'0"	INTERLAYERED clay & SAND interlayered grey very fine silty sand slightly silty to silty clay & clayey silt		11	SS	4									
			12	SS	9									
	saturated													BX casing following hole by own weight
50'0"	firm to stiff SAND grey slightly silty very fine sand.													
	saturated													
			13	SS	2									
	very loose		14	SS	4									BX casing to 60'0" sand backing up hole to 54'0"
	Dynamic cone resistance measured from 62'0" to 74'9" Probably SAND terminated at a refusal of 74'9"													
	very loose becoming compact													
73'4"	Boulder													
74'9"														
75'8"	SAND brown coarse sand with occasional piece of fine gravel very dense		15	SS	200/2"									BX casing to 70'0" sand backing up hole to 44'0" and water rising to ground surface BX casing driven to 73'4" AX casing drilled through boulder from 73'4" to 75'6" BX casing driven to 75'0" very hard and difficult to drive. Sample 15 then taken
	Borehole terminated at 75'8"													
	Artesian water rising to 12'0" above grade at termination of borehole.													
	N.B. the standard penetration results in the sensitive clay underestimate the strength of the material and likewise the artesian water present in the sand will have affected the standard penetration resistances measured in this material.													

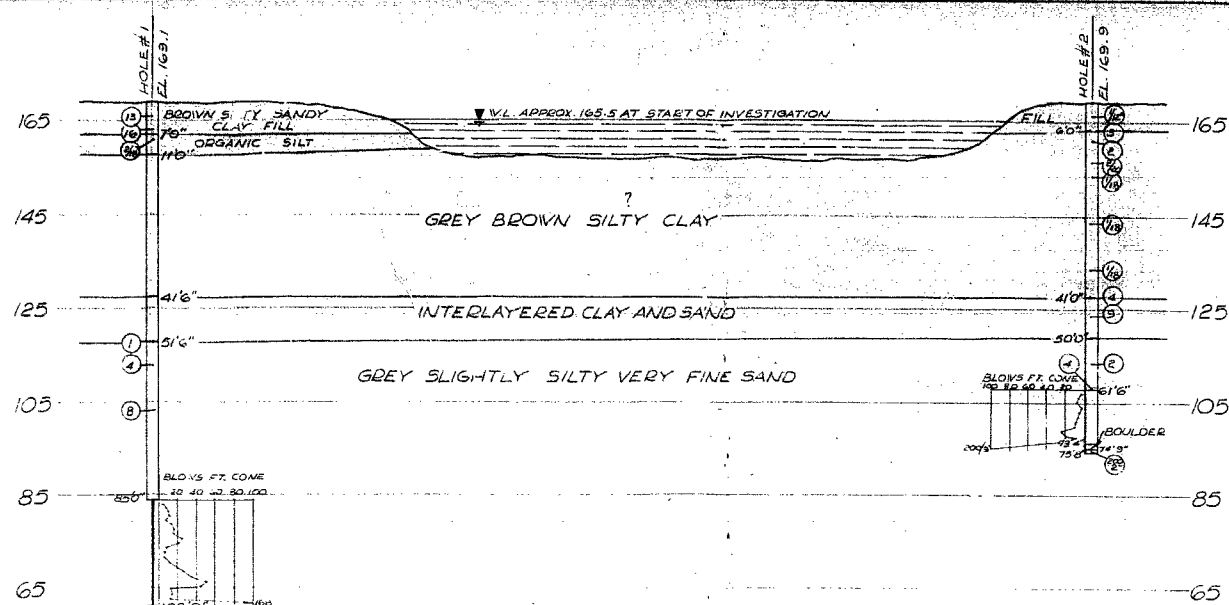
DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



SITE PLAN
SCALE 20' TO 1"

B.M. EL 17784
N.M. IN TOP OF O.S. ELM STR.
165' LT. OF STATION 6+62

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



SECTION THROUGH HOLES 1 & 2
SCALE: HOR. 100' TO 1", VERT. 20' TO 1"

LEGEND

- BOREHOLE
- BLOWS / FT.
- ▽ WATER LEVEL

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 WATT, DISTRICT OF MUSKOKA
% GREER, GALLOWAY & ASSOCIATES LTD.

DEE RIVER BRIDGE

PREPARED BY
PETO ASSOCIATES LTD.

JOB NO.	DATE:	SCALE	DRAWN BY	CHECKED BY
68 F67	MAY 1968	AS SHOWN	LL	KK