

64-F-255M

LOT 11, CON. XXII

STEPHEN TWP

2A.1811

MR. B. M. ROSS  
CONSULTING ENGINEER  
GODERICH ONTARIO

STRUCTURE SITE No. 12-288

64-F-255M

Report on  
SOIL INVESTIGATION  
for  
ROAD BRIDGE  
LOT 11, CONCESSION XXII  
TOWNSHIP OF STEPHEN

by

DOMINION SOIL INVESTIGATION LIMITED  
363 Queens Avenue  
LONDON ONTARIO

Reference No. 4-4-L1  
April 13, 1964

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### SUMMARY

The strata consist of topsoil and fill to 4.5 feet; soft or loose silt deposits containing a high proportion of compressible organic matter to approximately 20 feet and a dense granular stratum 2 to 4 feet thick. Below the granular layer is the hard crust of a clay till which becomes less stiff with depth and subsequently stiffens below about 40 feet.

A spread footing design is recommended with the footings bearing on or near the surface of the till at El.28. The use of a sheet pile cofferdam will be necessary for this design. The maximum soil pressure should not exceed 6000 p.s.f.

As an alternative, timber piles might be used, and theoretical safe loads over a range of elevations are given on page 5. Some difficulty may be encountered in reaching the necessary penetration to give the pile tips adequate scour protection. In this case it may be necessary to prebore.

Other types of pile are not likely to be economic, but of these the Franki-type displacement caisson with an expanded base would probably be the most satisfactory.

## I INTRODUCTION

In accordance with verbal authorization from the office of Mr. B. M. Ross, a soil investigation has been carried out at a site in the Township of Stephen where it is proposed to replace an existing road bridge with a new structure. The site lies 3 to 4 miles south of Grand Bend, and the bridge spans a tributary of the Ausable River.

It is understood that the new structure will be located in the same position as the existing one. The span will depend on the requirements for erosion protection as revealed by the soil conditions and may lie between 26 and 35 feet.

The purpose of this investigation has been to reveal the subsurface conditions and to determine the necessary soil properties for the design and construction of the new foundations.

## II FIELD WORK

Field work was carried out on the 3rd and 4th of April, 1964 and consisted of 2 boreholes at the locations shown on enclosure 2. The holes were advanced by washboring and lined with Bx (3-inch) casing. Standard Penetration tests were performed at frequent intervals to obtain a measure of the relative density or consistency of the soil and to recover disturbed samples. Undisturbed samples were recovered in 2-inch diameter thin-walled tubes. Vane shear tests were performed using a 2-inch diameter vane 4 inches long with a blade thickness of 1/8 inch.

Dynamic cone penetration tests were performed adjacent to each hole, and at the two other diagonally opposite corners of the bridge. This test gives a continuous record of penetration resistance. It enables the detection of abrupt changes in stratification and gives some indication of the resistance which might be encountered in the driving of piles.

The results of the field tests are recorded on geotechnical data sheets comprising enclosures 3 and 4.

## III SUBSURFACE CONDITIONS

Details of the stratification at each borehole are shown on the data sheets and a general picture of the subsurface conditions is given by the profile on enclosure 2. Under a 6-inch layer of topsoil the principal strata are as follows:

- (a) Fill. The fill forms the road embankment and consists mainly of silt with sufficient clay to impart a slight cohesion. Its moisture content is low and its

consistency is *stiff*. The dark brown colour is caused by the presence of a small amount of organic matter.

- (b) Silt or clayey silt with organics. This is a *soft*, unconsolidated mixture of silt, clay and organic material. Between depths of 4.5 and 9.5 feet in both boreholes, the amount of organic matter is small, there are traces of white marine shells, about 1 mm. in diameter, and a definite horizontal lamination is discernible. The proportion of organics increases with depth and the laminated structure disappears. In borehole 2 a layer of compressible black peat was encountered between 12.5 and 14.5 feet.

The density of the deposit is low, and several insitu vane shear strength tests gave values in the range 200 to 500 p.s.f.

This type of condition is not uncommon in the area. Marshland is associated with the formation of lagoons behind the sand dunes which border the Lake Huron shoreline. Extensive marshes have been mapped between Thedford and Grand Bend. The nearby valleys of streams and rivers are sometimes filled with organic deposits which are apparently extensions of the lagoons.

- (c) Gravel and sand. In both boreholes a layer of *dense* granular material was found around a depth of 20 feet. At borehole 1 this layer is 3.5 feet deep and consists mainly of fine to medium sand with traces of clay and gravel. At borehole 2 it is less than 2 feet deep, and consists of all sizes of gravel and sand with a trace of clay. This material probably represents the bed of an earlier river. A stratum of firm to stiff inorganic silty clay 4 feet deep separates the granular layer from the organic deposit in borehole 1 only.
- (d) Silty clay till. Immediately below the gravel layer, the *hard* upper crust of a silty clay till stratum is encountered. The stiffness of the till decreases with depth and in borehole 2, which extends deeper than borehole 1, a subsequent increase was observed below El.10. The till is a cohesive, impervious mixture of clay and silt, containing up to 5% of subangular gravel particles generally less than 1/2 inch in diameter.

#### IV FOUNDATIONS

The highest level common to both boreholes at which a satisfactory bearing stratum was encountered is El.28, which is 9.7 feet below the stream bed (El.37.7). This elevation corresponds to the bottom of the granular layer in borehole 1 and the top of this layer in borehole 2. Most of the material between the bed of the stream and El.28 is a soft wet organic deposit with a low shear strength. In these conditions

several types of foundation have been considered, and will be discussed in turn.

(a) Spread footings

On the basis of the field penetration tests a maximum gross soil pressure of 6000 p.s.f. is recommended for the design of spread footings bearing at El.28\*. The estimated settlement due to consolidation of the till is 1.0 inch. Differential settlement is unlikely to exceed 3/4".

To support the sides of the excavation, and to permit dewatering, it will be necessary to carry out the construction within a steel sheet-pile cofferdam. The sheet piles should be driven into the clay till to form a water seal. The pile tips should be driven to El.25 on the south side of the bridge, and at least to El.26 on the north side. Dewatering can be effected by pumping.

The closely similar results of cone penetration tests 2 and 3 suggest that uniform conditions will be encountered across the excavation on the south side, i.e. El.28 will correspond to the surface of the gravel stratum. At the north abutment the granular layer is thicker and was encountered at a higher elevation. The cone tests reached refusal at 2.5 to 5.0 feet higher than on the south side. The field evidence suggests that El.28 will coincide with the surface of or lie within the hard till layer at the north abutment. Particular care should be taken in both conditions to ensure that all organic or other soft material is removed from the grade before the footings are poured.

If, for reasons such as scour protection or to increase resistance to horizontal sliding, it is necessary to place the footings below El.28, the recommended soil pressure of 6000 p.s.f. is applicable between Els. 28 and 23. Whatever footing level is chosen below El.28, the sheet pile enclosure should be driven at least 2 feet lower.

In computing the resistance to horizontal sliding, the coefficient of friction between the footing and the soil should be taken as 0.35.

\* The diminishing resistance offered to both the split spoon and dynamic cone at El.26 might in some circumstances indicate a decrease in stiffness or density. In the present case it is believed to reflect only a change in the texture of the materials on either side of the interface. Material from the surface of the till layer was recovered in the split spoon, and found to be of very stiff to hard consistency. It yielded pocket penetrometer readings for unconfined compressive strength between 3.5 and 4.0 t.s.f.

## (b) Timber Piles

The use of timber piles is considered as a possible alternative to the foregoing arrangement of deep footings. Certain problems may arise in the driving of this type of pile because of the soil stratification. The piles will penetrate easily to the surface of the granular layer. In being driven through this layer into the till there is a possibility that they may suffer damage, and for this reason should be fitted with protective steel shoes. For the conditions encountered at borehole 1 it may not be possible to drive the piles into the till below El.28 unless the soil is prebored. Consideration should be given to whether this is deep enough for scour protection. At borehole 2 the piles can probably be driven through the hard upper crust of the till. The soil decreases in stiffness with depth so that the end bearing resistance also decreases. The skin friction on the sides of the pile does not increase sufficiently to compensate for the reduction. This is illustrated by the following table which gives the theoretical ultimate bearing capacity of 10-inch and 12-inch piles of constant cross-section, over a range of elevation. The calculations are made for the conditions at borehole 2. The assumed soil parameters are given on enclosure 5.

Elevation (feet)	Ultimate bearing capacity (tons)	
	10-inch pile	12-inch pile
25	13.5	19.5
24	13.5	19.5
23	13.5	19.5
22	10.0	14.0
21	11.1	15.5
20	12.4	17.0
19	13.6	18.5
18	11.7	15.3
17	12.9	16.8
16	14.2	18.3
15	15.4	19.8
14	16.7	20.3
13	17.9	21.8
12	19.2	23.3
11	20.4	24.8
10	21.7	26.3
0	36.6	46.9

It may be assumed, conservatively, that equivalent values apply at borehole 1 at the same elevations. A factor of safety not less than 2 should be applied to the foregoing figures. Thus, a pile with a working load of approximately



10 tons can be supported at El.25, but it must be driven much deeper before any substantial increase in bearing capacity is achieved. It is important to note that a substantial part of the theoretical bearing capacity is derived from end bearing. For this reason the diameter of the pile tips should not be less than that assumed.

If the design requires that the piles will lie partly above and partly below the water table, pressure-impregnated creosote-treated piles should be used.

In conclusion, the following approach to the use of timber piles is recommended:

- (i) piles should be used only if they offer a substantial economic advantage over the footing design which requires a sheet pile cofferdam.
- (ii) if the requirements of erosion protection or horizontal resistance to sliding dictate that the pile tips must penetrate below El.28, facilities should be provided for preboring - at least on the north abutment.
- (iii) if at all possible (with reference to (ii) above) the pile tips should bear in the hard upper crust of the till, taking advantage of its greater strength, or in the granular layer above it.

#### (c) Steel or Concrete Piles

A 12-inch diameter steel pipe pile driven to El.0.0 would have a theoretical ultimate bearing capacity of 46.9 tons, and thus a safe working load of about 23 tons. This would not be an economic value for a steel pile. It would be necessary to drive a steel member an undetermined distance below the depth of exploration before an economic safe working load could be reached. By comparison with timber piles, this type of pile has no apparent advantages and would be much more costly.

The Franki type displacement caisson with an expanded base would probably be more economic than a steel pipe. Safe working loads of 50 to 60 tons per pile could be achieved with 30-inch diameter bulbs formed between Els. 10 and 5. It would be necessary to encase that part of the shaft passing through the soft organic strata in a permanent thin-walled steel shell such as a helcore pipe.

In conclusion, the use of spread footings is considered to be the most straight forward method. If it is necessary to use piles, timber will probably be the least expensive although some problem may arise in reaching the required penetration.

V

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4. The Ultimate Bearing Capacity of Foundations by G. G. Meyerhof Geotechnique, Vol. II, 1950 and 1951.
5. M. J. Tomlinson, The Adhesion of Piles Driven in Clay Soils, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, 1957.
6. Some Pile Loading Tests in Stiff Clay: by K. Y. Lo and A. G. Stermac, Canadian Geotechnical Journal, Volume I, Number 2, March 1964.



DOMINION SOIL INVESTIGATION LIMITED

*James Park*

James Park, M.Sc., P.Eng.

## LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

### SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
		COARSE	FINE	COARSE	MEDIUM	FINE						
φ > 8"	3"	¾"	4.75mm	2.0	0.42	0.074	0.002	>				
U.S. Standard Sieve Size :		No.4	No.10	No.40	No.200							

### SAMPLE TYPES.

AS Auger sample	RC Rock core	TP Piston, thin walled tube sample
CS Sample from casing	% Recovery	TW Open, thin walled tube sample
ChS Chunk sample	SS Split spoon sample	WS Wash sample

<b>SAMPLER ADVANCED BY</b>	static weight : w	<b>OBSERVATIONS</b>		Steady pressure
"	pressure : p	<b>MADE WHILE CORING</b>		No pressure
"	tapping : t			Intermittent pressure

	Washwater returns
	Washwater lost

### PENETRATION RESISTANCES.

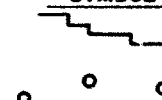
**DYNAMIC PENETRATION RESISTANCE** : to drive a 2"  $\phi$ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

**STANDARD PENETRATION RESISTANCE, -N-** : to drive a 2" outside dia, split spoon sampler 1 foot into the ground, expressed in blows per foot.

#### EXTRAPOLATED -N- VALUE

The energy for the penetration resistances is supplied by a 140 lb. hammer falling 30 inches

**SYMBOL :**



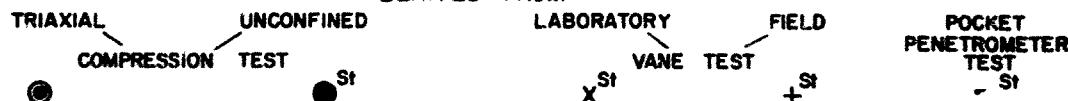
**322**

### SOIL PROPERTIES.

W % Water content	$\gamma^s$ Natural bulk density (unit weight)	k Coeff. of permeability
LL % Liquid limit	e Void ratio	C Shear strength
PL % Plastic limit	RD Relative density	$\phi$ Angle of int. friction
PI % Plasticity index	$C_v$ Coeff. of consolidation	C' Cohesion
LI Liquidity index	$m_v$ Coeff. of volume compressibility	$\phi'$ Angle of int. friction

### UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —



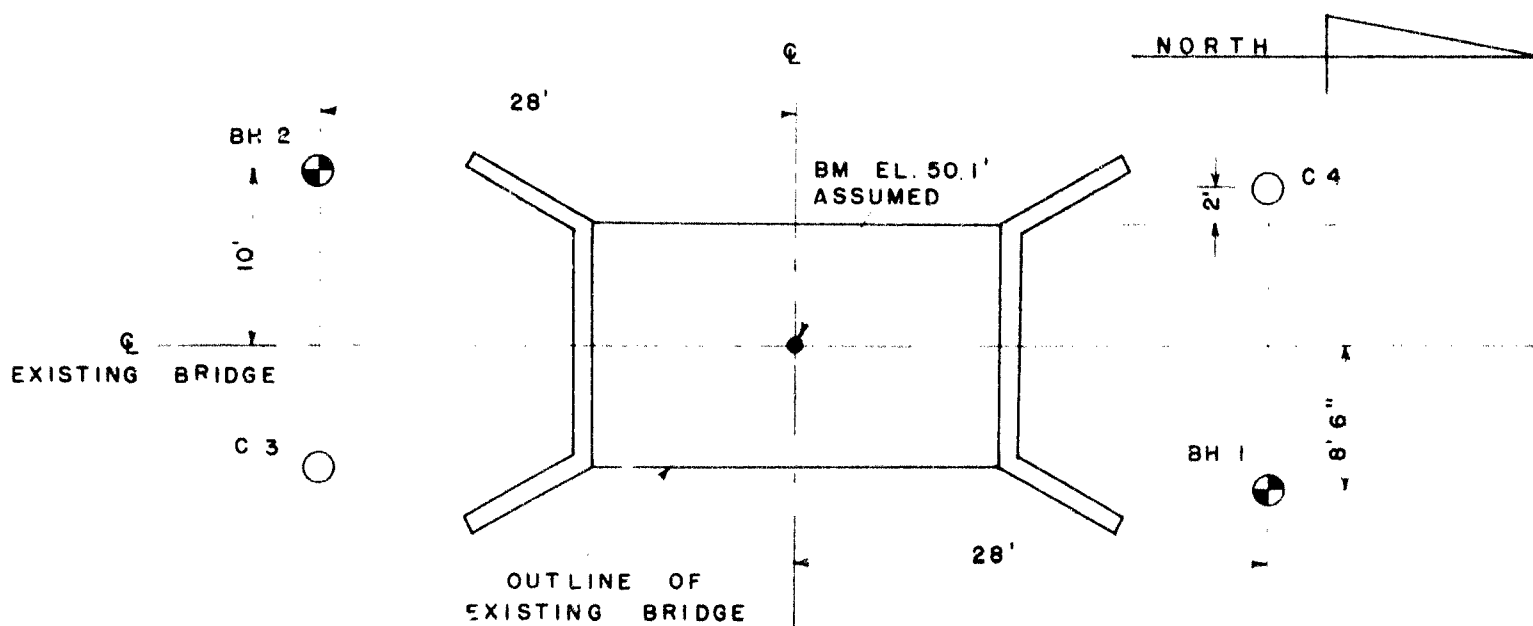
Strain at failure is represented by direction of stem

$20\%$   
 $15\% \quad 5\%$   
 $10\%$

$St$  : sensitivity =  $\frac{\text{shear strength in undisturbed state}}{\text{shear strength in remoulded state}}$

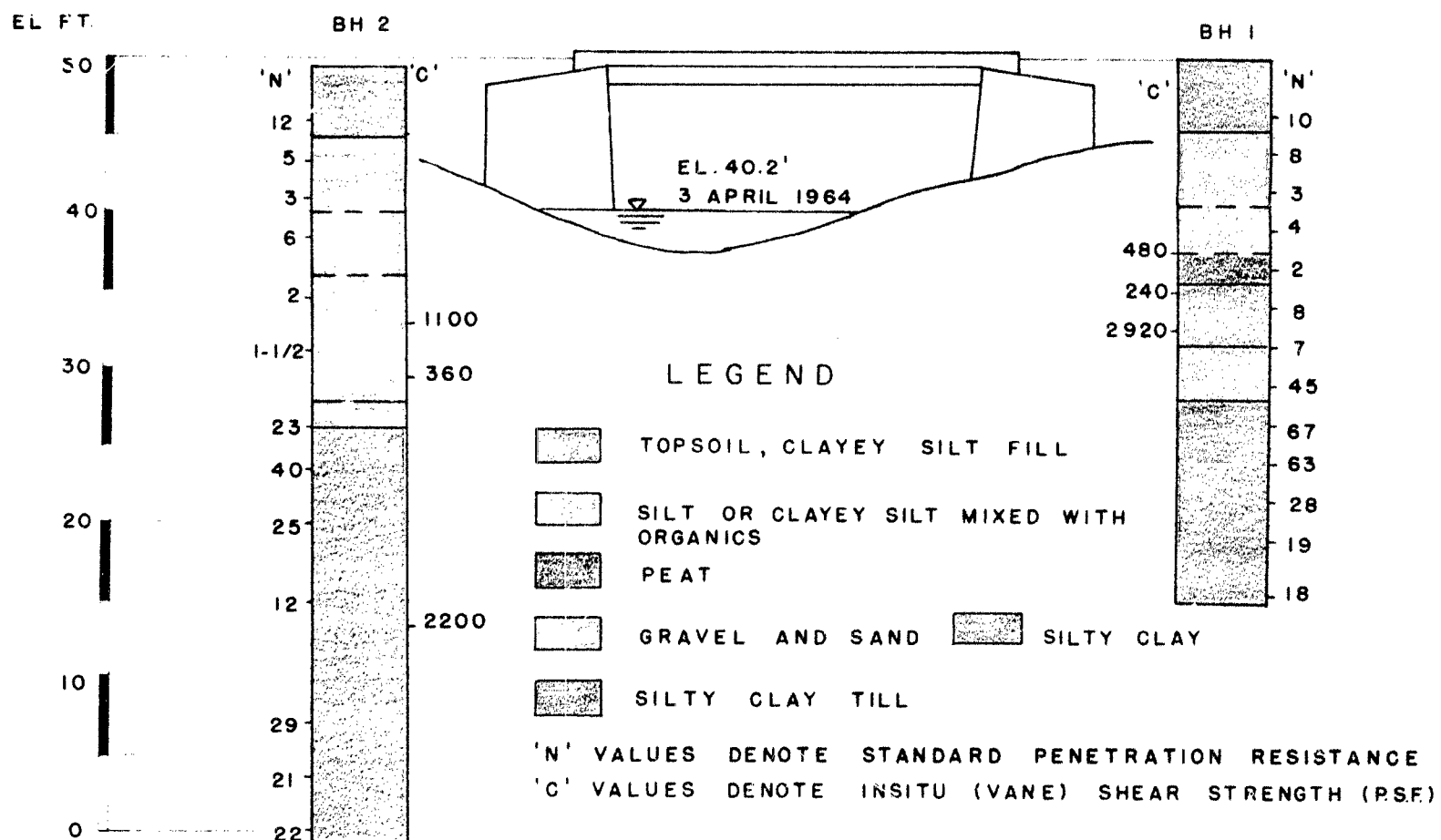
### SOIL DESCRIPTION.

COHESIONLESS SOILS :	RD :	COHESIVE SOILS :	C lbs/sq.ft.
Very loose	0 - 15 %	Very soft	less than 250
Loose	15 - 35 %	Soft	250 - 500
Compact	35 - 65 %	Firm	500 - 1000
Dense	65 - 85 %	Stiff	1000 - 2000
Very dense	85 - 100 %	Very stiff	2000 - 4000
		Hard	over 4000



## LOCATION OF BOREHOLES

SCALE: 1 INCH TO 10 FEET



## SUBSURFACE PROFILE (LOOKING EAST)

SCALE: 1 INCH TO 10 FEET

OUR REFERENCE NO 4-4-1.1

## GEOTECHNICAL DATA SHEET FOR BOREHOLE .1 . . . .

CLIENT: Mr. B. M. Ross

PROJECT: Road Bridge

LOCATION: Township of Stephen

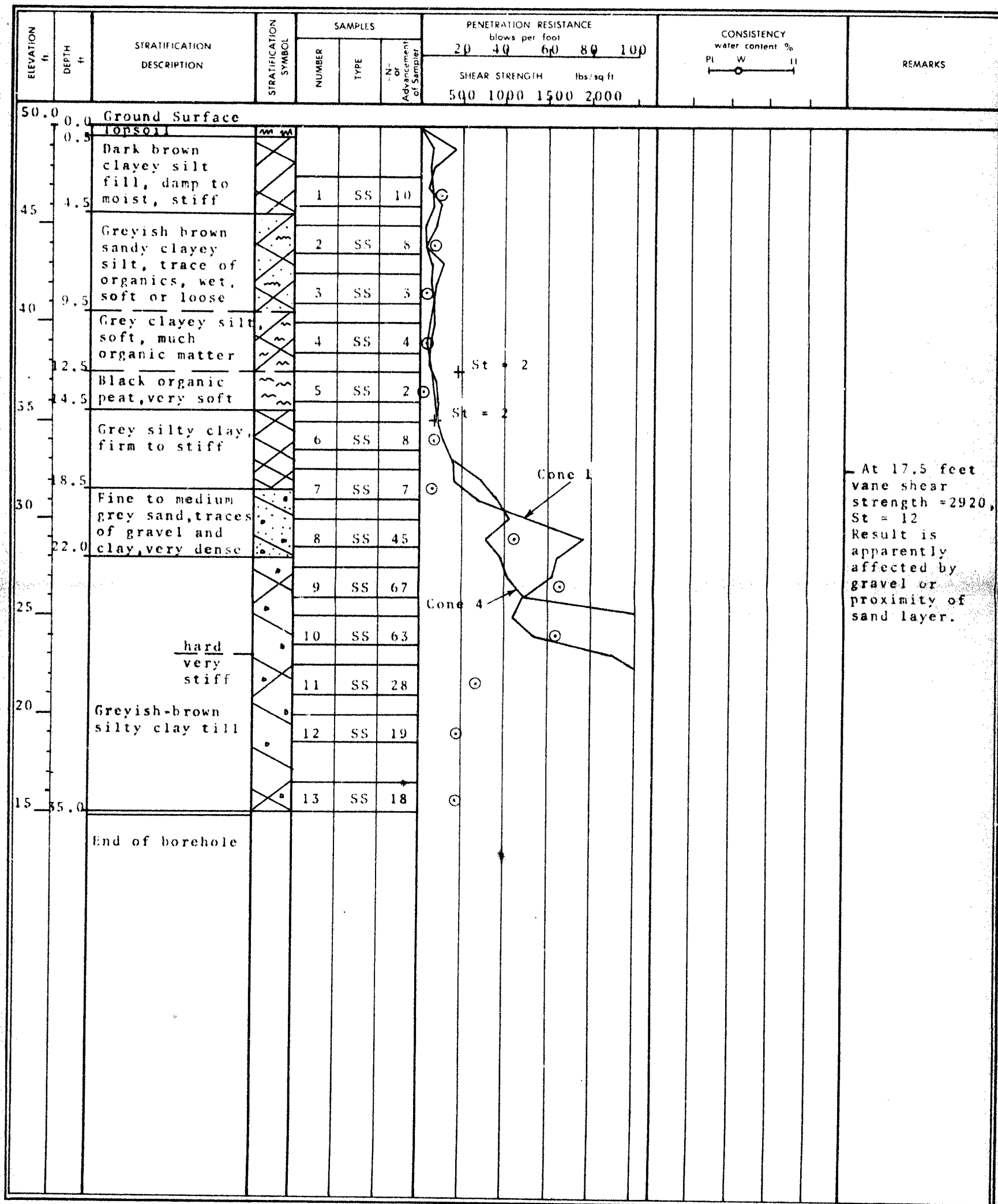
DATUM ELEVATION: 50.1 feet (see encl. 2)

METHOD OF BORING: Washboring

DIAMETER OF BOREHOLE: Bx (3-inch)

DATE: April 4, 1964.

ENCLOSURE NO. 3



VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: SB

CH'D: JP

# GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . . .

OUR REFERENCE NO 4-4-11

CLIENT: Mr. B. M. Ross  
 PROJECT: Road Bridge  
 LOCATION: Township of Stephen  
 DATUM ELEVATION 50.1 feet (see encl. 2)

METHOD OF BORING Washboring  
 DIAMETER OF BOREHOLE 8x (3-inch)  
 DATE: April 3, 1964.

ENCLOSURE NO. 4

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot					CONSISTENCY water content %			REMARKS
				NUMBER	TYPE	N- or Advancement of Sampler	20	40	60	80	100	PL	W	LI	
							SHEAR STRENGTH      lbs/sq ft								
							500	1000	1500	2000					

49.7	0.0	Ground Surface																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														</
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\*\* Sample 8:  
 Sampler advanced  
 by tapping below  
 21'-8". End of  
 tube damaged.

\* Sample 9:  
 19/6"  
 14/6"  
 9/6"

Data used in calculating the ultimate bearing capacity of piles.

Elevation	Undrained shear strength (p.s.f.)	Adhesion (p.s.f.)
25 to 22	5500	1000
22 to 18	3500	1000
18 to 8	2200	1000
8 to 0	3500	1000