#67-F-256M

TWELFTH LINE

BRIDGE

TECUMSETH TWP.

W.P. 20-74
Site 30-430

AINLEY AND ASSOCIATES
CONSULTING ENGINEERS
105 HURONTARIO STREET
COLLINGWOOD, ONTARIO

TOWNSHIP OF TECUMSETH

TWELFTH LINE BRIDGE

FOUNDATION CONDITIONS

SUBMITTED BY:

DOMINION SOIL INVESTIGATION LIMITED
77 CROCKFORD BOULEVARD
SCARBOROUGH, ONTARIO.

MARCH, 1967.

OUR REF. NO: 7-1-4

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0. INTRODUCTION

At the request of Messrs. Ainley and Associates, Consulting Engineers, a soil investigation was carried out by Dominion Soil Investigation Limited at the site of a proposed bridge (project no: 6658) in Tecumseth Township, Ontario. The structure will carry the 12th line road over Innisfil Creek.

The purpose of our work was to reveal the soil profile below the site, to determine the engineering properties of each stratum encountered, and to advise on the design and construction of feasible foundation methods.

This report presents our findings and recommendations.

1. SUMMARY

A very thick varved clay deposit underlies the site. The consistency of the material is soft to a depth of about 40 ft., therefore the design of spread foundations is not feasible. Point-bearing piles had to be ruled out also, because no dense stratum was encountered to a depth of 144 ft. The only practicable foundation method is friction piles with an estimated safe load bearing capacity of 9 to 20 tons per pile, depending on type of pile used. The design bearing capacity should be determined by a full scale load test.

The bracing and shoring of the excavation for the construction of the pile cap should be designed using soil engineering methods.

2. THE SITE

The site is on the 12th line about 2 miles west of Highway No. 27 near Cookstown, Ontario (see enclosure no. 2). The existing unpaved road passes westward through flat, forested country to Innisfil Creek, which is to be bridged. The creek is about 25 ft. wide at the site and flows south-west across the alignment at an angle of about 45 degrees.

3. SUBSURFACE CONDITIONS

The site is underlain by a varved silty clay deposit with a desiccated crust and topsoil. The crust is brownish due to weathering, but at a depth of 3 to 5 ft. below ground level the colour changes to a uniform grey. The unweathered silty clay extends to over a depth of 120 ft. and because this is the significant stratum for the present project, its engineering properties will be treated below.

31. Varved silty clay

Remarkably uniform in appearance throughout the explored depth, this stratum consists of alternating layers - "varves" - of silty clay and silt with fine sand. The structure is irregular: the thickness of the clay layers varied between 3/4 and 6-inches, whereas that of the silt with fine sand layers varied between 1 mm. and 2-inches within the recovered samples. Although the orientation of these layers was mainly horizontal, by no means is it certain that they are continuous.

In view of the generally wide spacing between the thin silt seams the deposit can be regarded as a homogeneous clay stratum which derives its immediate shear strength from cohesion. The significant index properties of the clay are listed below:

Liquid Limit	32%
Plastic Limit	19%
Plasticity Index	13%
Unified Soil Classification System Group Symbol	CL
Natural Moisture Content	average: 28%
	range: 22 - 35%
Liquidity Index	average: 0.45
	range: 0.2 - 0.9
Sensitivity	approximately 2 (low to medium)
Unit Weight	average: 120 pcf
Assumed Specific Gravity	2.75
Void Ratio	average: 0.84
	range: 0.72 - 0.92

The undrained shear strength of the clay was measured by field vane and laboratory unconfined compression tests. The results are in good agreement with each other and define a linear relationship between the effective overburden pressure (= p) and undrained shear strength (=c). The calculated c/p ratio is 0.25. As indicated on enclosure no. 4, the measured shear strength values increased from about 200 psf near the bottom of the desiccated crust to about 1350 psf at a depth of 95 ft.

The fact that the extension of the c versus depth line passes through the 0 point indicates that the deposit is normally consolidated, i.e. it never sustained larger loads than existing at present. Some over-consolidation is noticeable in the upper crust due to desiccation. A few field vane tests, especially in borehole no. 1, indicated higher shear strength values, but these are attributed to the presence of silt pockets or layers. The measured moisture contents were also somewhat lower in borehole no. 1 but no regular trend with depth was observed.

The failure strain of the unconfined compression test specimens was between 8 and 20%.

The compressibility of the varved silty clay was measured by a laboratory consolidation test on sample 2 from borehole no. 1, and the modulus of compressibility corresponding to the pressure range to be considered in the settlement analysis, is 60 TSF. The coefficient of consolidation for the same pressure range is 0.52 ft/day.

The soil is practically impervious in the vertical direction, but may be quite permeable in the horizontal direction if the silt and sand layers were continuous.

32. Groundwater

The groundwater table was near the surface at the time of field work. It will probably drop in the summer, and the lowest position may be indicated by the end of zone of weathering where the colour of the soil changes to grey.

4. DISCUSSION

41. General Information

It is understood that the proposed bridge will carry a two-lane road and that the main structure will consist of simply supported concrete beams spanning 60 ft. The centre line of the bridge will form a skew angle of about 45 degrees with the centre line of the creek and the deck will be at around elevation 102.5 ft. According to information obtained from the Consulting Engineers, the total reaction at the base of the abutments will be of the order of 900 kips and the pile caps will be approximately 45 ft. by 12 ft. in plan. Since the present riverbed is at elevation 95 ft., elevation 90 ft. was assumed as the bottom level of the proposed pile cap.

Our analysis will be based on the above assumptions, therefore, if the final design were different from the preliminary one, a review of the conclusions might be advisable.

42. Design of Foundations for the Abutments

From the field and laboratory test results it is obvious that the subsoil has insufficient bearing capacity to a depth of about 40 ft. below ground level to support the proposed structure on spread footings, therefore, piled foundations will have to be considered. No suitable bearing stratum was encountered within the explored depth (144 ft.), consequently end-bearing piles are believed to be uneconomical. The only feasible solution is a foundation "floating" on friction piles.

Two types of friction piles were considered: timber and steel. These will be treated separately in the following subsections.

421. Timber Friction Piles

Untreated timber piles will be suitable because the entire pile will be below the ground water table. Fifty feet long piles, measuring 7-inches in diameter at the tip and 12-inches at 3 ft. from the butt were analysed. With the tip of piles at elevation 41 ft. and using a safety factor of 3, the safe bearing capacity of the pile was calculated to be 8.7 tons. Therefore, approximately 51 piles will be required which could be arranged in three rows, each consisting of 17 piles. The front row of piles was assumed to be battered at an angle of 1 to 3, in order to secure the stability of the abutment.

The shear stress along the total circumferential area surrounding the above described pile group is about 170 psf, therefore no reduction for group action is necessary.

The settlement of the pile group is estimated to be of the order of 2½-inches, and assuming that no drainage can take place through the silt and sand layers, the rate of settlement could be very slow: 50% of the consolidation will take place in 5.2 years and 90% of the consolidation will need 22 years to complete. The presence of continuous silt and sand layers, however, may considerably accelerate the rate of consolidation.

422. Steel Friction Piles

The performance of 60 ft. long, 12BP53 steel H piles was analysed with the tip at elevation 30 ft. The safe load carrying capacity of a single pile was calculated to be of the order of 21 tons, thus 21 piles would be required to support the total reaction of 450 tons. These piles can also be arranged in three rows, each containing seven piles and the front row should be battered (at 1 to 3) to resist the horizontal unbalanced forces.

No reduction for group action is necessary and the settlement of the pile group will be about the same as that for the timber friction piles. The consolidation will take the same long period of time, provided no drainage can occur through continuous, pervious and horizontal layers.

423. Load Test

There are inherent difficulties in determining the actual carrying capacity of friction piles embedded in clay, therefore it is recommended that a full scale pile loading test be carried out. The test should be used to confirm the estimated capacity of the piles or to modify the design if necessary.

Since the shear strength of the clay around the piles could be temporarily reduced by the remoulding action of the pile driving, the load tests should be carried out a minimum of one month after the completion of pile driving.

43. Approach Fill

Since the road grade will be 2 to 3 ft. above the present ground level, the height of the approach fill will be small. From the undrained shear strength of the varved clay it was estimated that the subsoil would be stable provided the height of the embankment did not exceed 6 ft.

When the topsoil is being stripped from the site, the desiccated crust should be left intact to derive maximum benefit from its load distributing characteristics.

44. Construction

The excavation for the pile cap will be in soft clay, therefore bottom heave is anticipated if the depth of the former exceeded about 4 ft. Furthermore, the stability of the walls of excavation have to be preserved during construction. Therefore, we recommend that the bracing and shoring should be designed in advance, using soil engineering methods to avoid costly failures and construction difficulties.

For the sake of completeness, we mention it here, that pile heave may be experienced, especially if timber piles were employed. Pile elevations should be frequently checked during driving and the ones which heaved, should be re-driven to the original level.

REGISTERED PROFESSIONAL ENGINEER
L.S. ROLKO
PROVINCE OF ONTARIO

DOMINION SOIL INVESTIGATION LIMITED

L.S. Rolko

L.S. Rolko, P.Eng., A.M. ASCE.

LSR/me

5. APPENDIX

51. Location and Elevation of Boreholes

The site and boreholes were located with the aid of a plan (No. 6658-P2, Dec. 1966) provided by the Consulting Engineers. Preliminary data re the type of structure, span and road grade were also indicated on this drawing. Elevations were referred to a nail in the root of a 30-inch maple tree, 84 ft. left of station 0 + 00 (= 100.0 ft.).

52. Field Work

Between January 20 and March 15, 1967 two boreholes of a total depth of 181.5 ft. and two dynamic cone penetration tests of a total depth of 95.5 ft. were put down at the locations shown on enclosure no. 2. The holes were advanced by a washboring rig. Four samples were taken with a 2-inch outside diameter split spoon (= split barrel), which was driven into the subsoil with a 140 lb. hammer dropping 30-inches. This energy was used in the dynamic cone penetration test also in which a cone (2" Ø, 60 deg. apex) attached to the end of 1 5/8 inch diameter drilling rods was driven into the soil without casing. From driving the split spoon, the Standard Penetration Resistances (= 'N' values) were derived, which indicate the density of the substrata at the sample locations, whereas the dynamic cone penetration test results produced a continuous record of subsoil density.

The majority of the samples was taken with 2-inch diameter thin walled tubes (= Shelby tubes), which were advanced into the bottom of the borehole with one continuous, rapid movement. By this method relatively undisturbed samples were recovered from frequent intervals of depth, which were sealed with wax in the field.

The in-situ shear strength of the soil was measured with a 2-inch diameter, 4-inch long, four-bladed vane with a wing thickness of 1/8-inch. After the initial testing, the soil was thoroughly remolded by rotating the vane and the shear strength measured again. The ratio of the undisturbed and remolded shear strength of the clay yields the sensitivity index.

53. <u>Laboratory Work</u>

All samples were shipped to our laboratory. The soil was extruded from the thin-walled tubes, thus the nearly intact structure of the material could be examined.

The natural water content of the samples was measured to obtain a full profile of natural moisture content versus depth. The unit weight of five samples was also determined, which enables the computing of void ratio in the knowledge of specific gravity. The latter was assumed on the basis of published data.

The liquid and plastic limits of five samples was measured to classify the soil.

All results are shown on the logs of boreholes, and summarized on the log of borehole no. 2.

The soil samples are available for inspection for three months. Thereafter, they will be disposed of unless otherwise instructed.

54. <u>References and Relevant Standards</u>

1. The Unified Soil Classification System by A.A. Wagner. (ASTM Procedures for Testing Soils, 1964, p. 208).

2. CSA Standard No. A119.1-1960: Code for Split Barrel Sampling of Soils.

3. CSA Standard No. A119.2-1960: Code for Thin Walled Tube Sampling of Soils.

4. CSA Standard No. A119.3-1962: Dynamic-Cone Soil Penetration Test.

5. Terzaghi and Peck: Soil Mechanics in Engineering Practice. John Wiley & Sons Inc., New York, 1948.

6. Tschebotarioff: Soil Mechanics, Foundations and Earth Structures. McGraw Hill, 1953.

7. L. Bjerrum et al. The settlement of a bridge abutment on friction piles. Proc. IV Int. Conf. on Soil Mechanics and Foundation Engineering, London, England, 1957.

8. M.J. Tomlinson: The Adhesion of Piles Driven in Clay Soils. Proc. IV Int. Conf. on Soil Mechanics and Foundation Engineering, London, England, 1957.

9. A.W. Bishop and D.J. Henkel: The Measurement of Soil Properties in the Triaxial Test. London 1957.

10. G.A. Leonards: Foundation Engineering. McGraw Hill, 1962.

E n c l o s u r e s

LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK
		COARSE	FINE	COARSE	MEDIUM	FINE				
Ø > 8"	3"	3/4"	4.76mm	2.0	0.42	0.074	0.002 >		NO SIZE LIMIT	

U.S. Standard Sieve Size: No.4 No.10 No.40 No.200

GROUND WATER LEVEL — DEPTH OF CAVE-IN

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

SAMPLER ADVANCED BY static weight : w
" pressure : p
" tapping : t

OBSERVATIONS MADE WHILE CORING: Steady pressure; No pressure; Intermittent pressure

Washwater returns

Washwater lost

PENETRATION RESISTANCES.

SYMBOL:

DYNAMIC PENETRATION RESISTANCE : to drive a 2"ϕ, 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 — 322

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

SOIL PROPERTIES.

W %	Water content	γ	Natural bulk density (unit weight)	k	Coeff. of permeability
LL %	Liquid limit	e	Void ratio	C	Shear strength — in terms of total stress
PL %	Plastic limit	RD	Relative density	ϕ	Angle of int. friction — in terms of total stress
PI %	Plasticity index	C_v	Coeff. of consolidation	C'	Cohesion — in terms of effective stress
LI	Liquidity index	m_v	Coeff. of volume compressibility	ϕ'	Angle of int. friction — in terms of effective stress

UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL / UNCONFINED COMPRESSION TEST: ◉ ●St

LABORATORY / FIELD VANE TEST: X St + St

POCKET PENETROMETER TEST: St

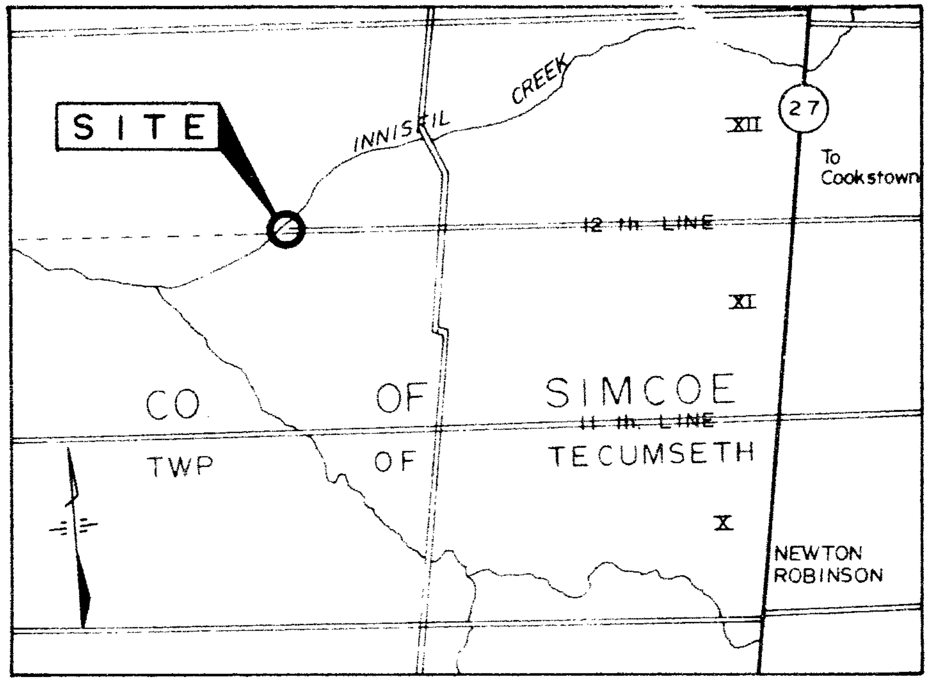
Strain at failure is represented by direction of stem: 20% (up), 5% (right), 10% (down), 15% (left)

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

SOIL DESCRIPTION.

COHESIONLESS SOILS:	RD:
Very loose	0 – 15 %
Loose	15 – 35 %
Compact	35 – 65 %
Dense	65 – 85 %
Very dense	85 – 100 %

COHESIVE SOILS:	C lbs/sq ft
Very soft	less than 250
Soft	250 – 500
Firm	500 – 1000
Stiff	1000 – 2000
Very stiff	2000 – 4000
Hard	over 4000

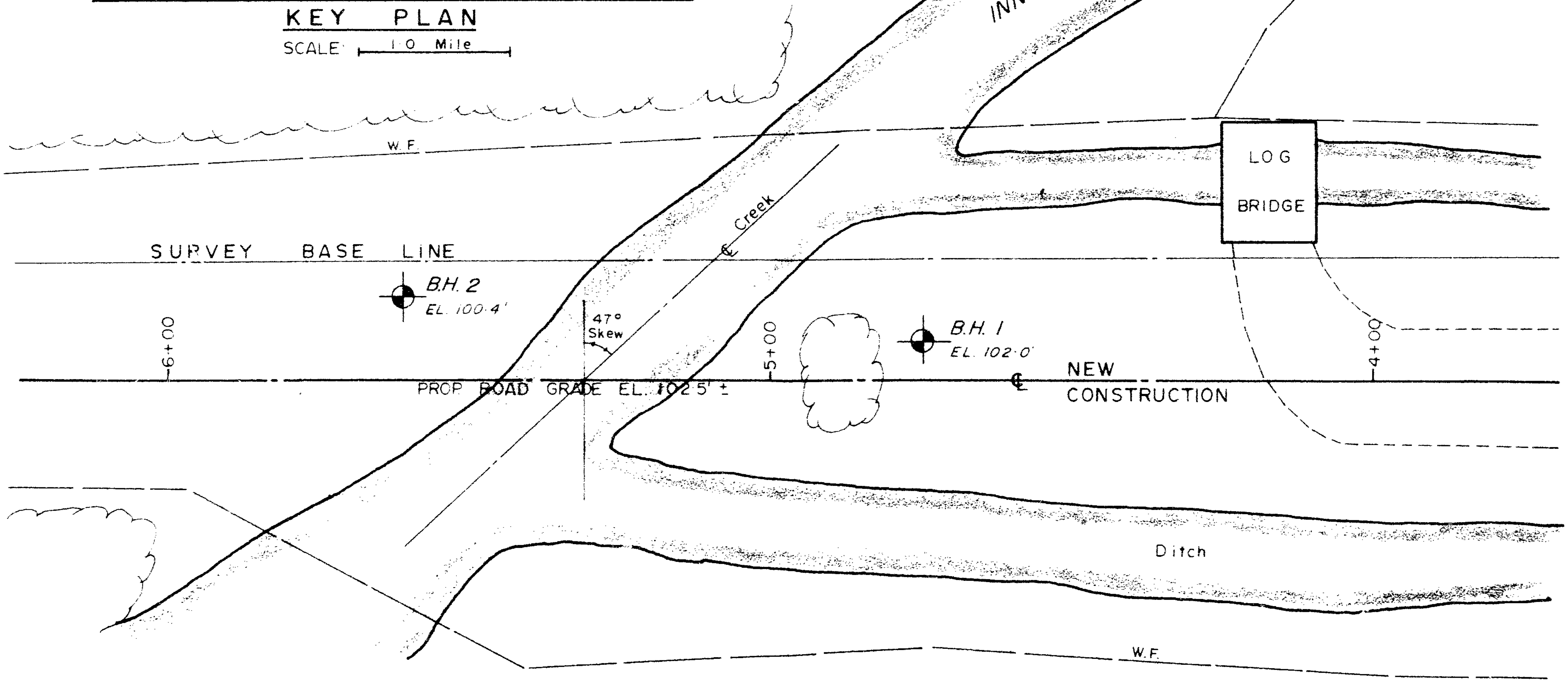


KEY PLAN

SCALE: 1.0 Mile

BOREHOLE LOCATIONS

B.H. Nº 1	STA. 4+75	14' LT. of BASE LINE
B.H. Nº 2	STA. 5+61	6' LT. of BASE LINE



BOREHOLE LOCATION PLAN

SCALE: 1 inch = 20 feet

LOG OF BOREHOLE ......1......

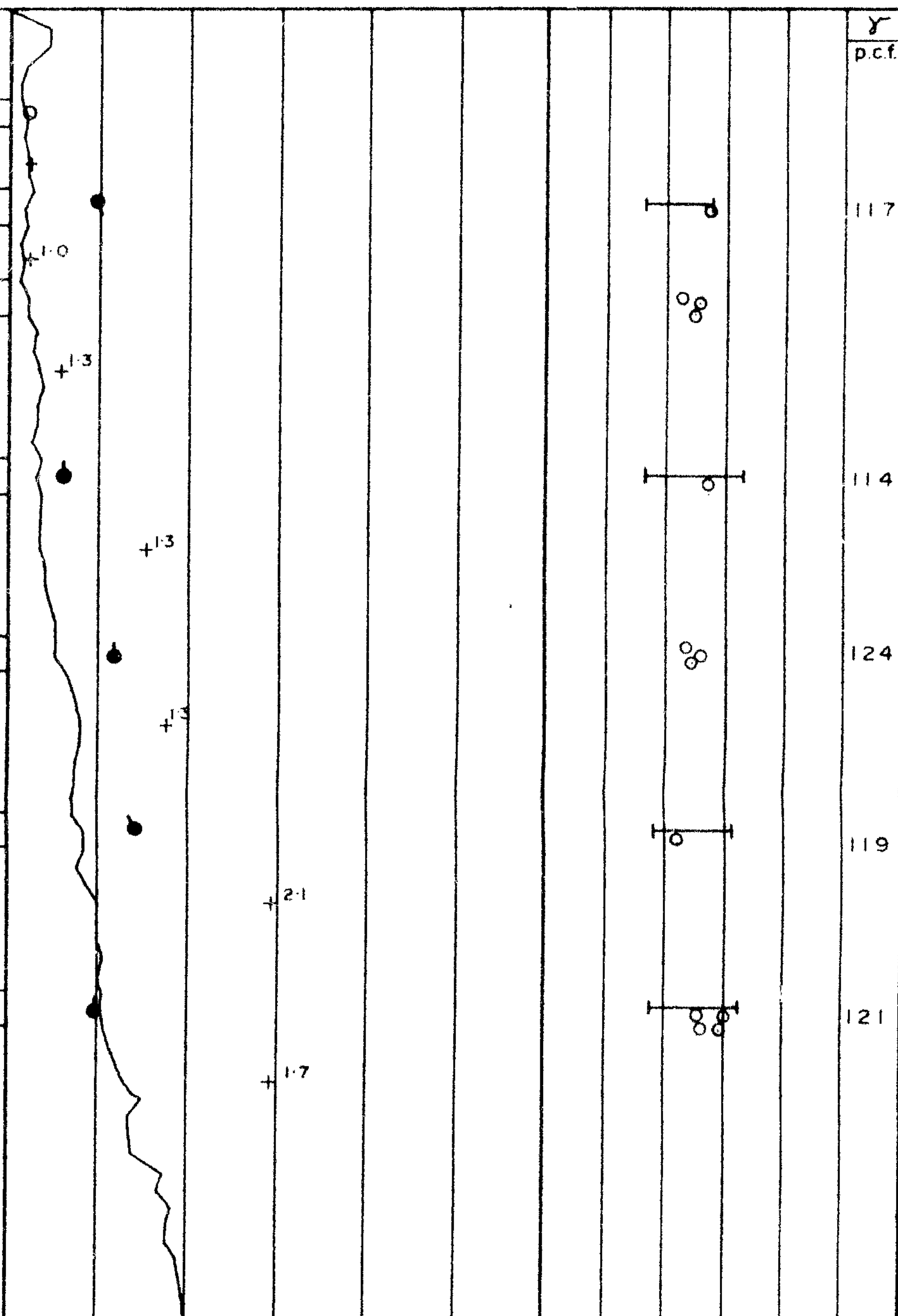
Our Reference Nº 7 - 1 - 4 — Enclosure Nº 3

CLIENT: AINLEY & ASSOCIATES
PROJECT: 12 th. LINE BRIDGE
LOCATION: TWP. OF TECUMSETH, LOT 19, CON. 11 & 12
DATUM ELEVATION: N. & W. - El. 100·0'

DRILLING DATA
Method: WASHBORING
Diameter: 3"
Date: JAN. 20, 1967

SUBSURFACE PROFILE					SAMPLES				
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot	γ p.c.f.	REMARKS
102·0	0	GROUND SURFACE							
100		TOPSOIL weathered desiccated crust, brown							
		grey		W.L. El. 98·0 Jan. 20, 1967	1	SS	4		
90	10	very soft			2	TW	P	117	
					3	TW	P		
80	20	SILTY CLAY with thin silt and fine sand layers.			4	TW	P	114	
70	30	soft			5	TW	P	124	
60	40	(varved deposit)							
		firm			6	TW	P	119	
50	50				7	TW	P	121	
40	60	END OF BOREHOLE							
30	70								
	73	END OF CONE TEST							
20	80								

PENETRATION RESISTANCE Blows / Foot: 20 40 60 80 100
UNDRAINED SHEAR STRENGTH lbs/sq.ft.: + FIELD VANE TEST ● COMPRESSION TEST — 500 1000 1500 2000 2500
WATER CONTENT %: PLASTIC LIMIT W_p, NATURAL W, LIQUID LIMIT W_L — 10 20 30 40 50



VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: D.A.M. CHECKED: Rollis

LOG OF BOREHOLE 2
AND SUMMARY

Our Reference No. 7 - 1 - 4

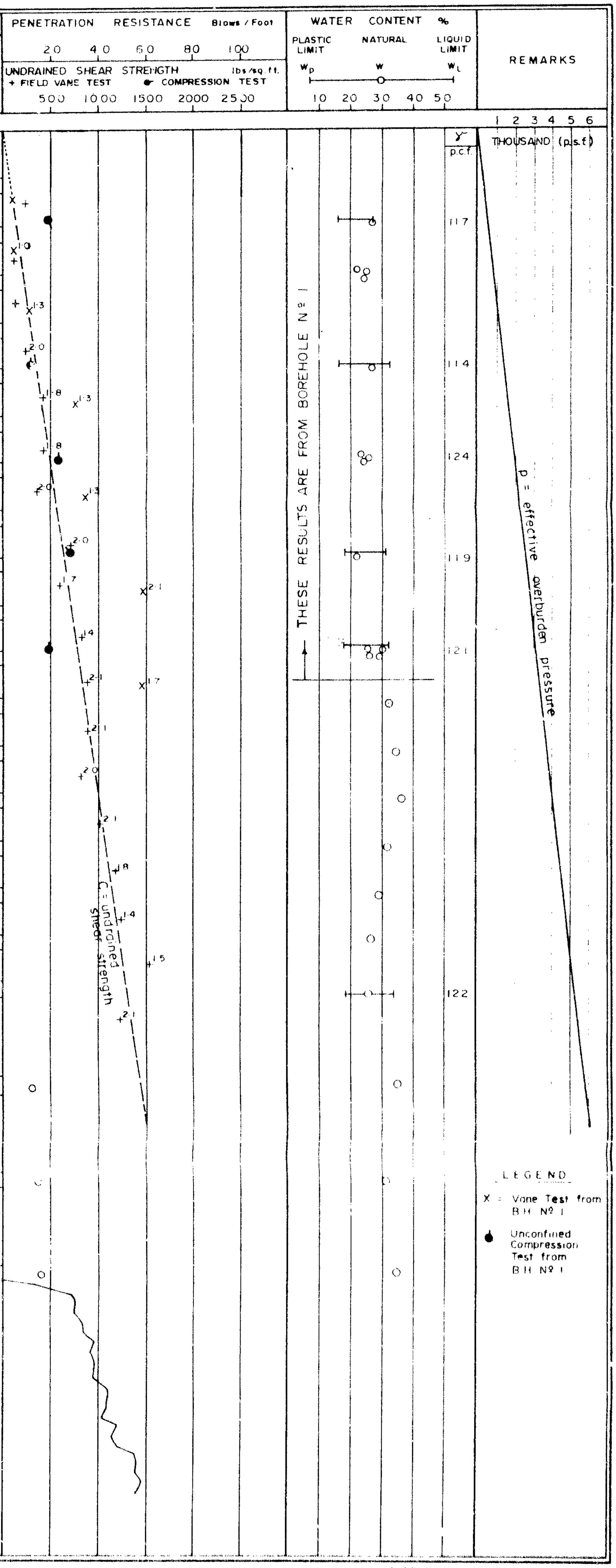
Enclosure No. 4

CLIENT: AINLEY & ASSOCIATES
PROJECT: 12 th LINE BRIDGE
LOCATION: TWP. OF TECUMSETH, LOT 19, CON. 11 & 12
DATUM ELEVATION: N. & W. - El. 100·0'

DRILLING DATA
Method: WASHBORING
Diameter: 3"
Date: MAR 13 - 15, 196?

SUBSURFACE PROFILE / SAMPLES

ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	GROUND WATER	SAMPLE NUMBER	TYPE	'N' Blows/Foot
100·4	0	GROUND SURFACE				
		TOPSOIL — weathered desiccated crust	W.L. El. 98·6' Mar. 15, 1967			
		SILTY CLAY with thin silt and fine sand layers. (varved deposit) — brown / grey, very soft		1	TW	P
90	10			2	TW	P
				3	TW	P
80	20	soft		4	TW	P
				5	TW	P
70	30			6	TW	P
		firm		7	TW	P
60	40			8	TW	P
				9	TW	P
50	50			10	TW	P
				11	TW	P
40	60			12	TW	P
				13	TW	P
30	70	stiff		14	TW	P
				15	TW	P
20	80			16	TW	P
				17	TW	P
10	90			18	TW	P
0	100			19	SS	12
-10	110			20	SS	14
-20	120			21	SS	15
	121·5	END OF BOREHOLE				
-30	130					
-40	140					
	144	END OF CONE TEST				



VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE ... M CHECKED: ...

DOMINION SOIL INVESTIGATION LIMITED

CONSOLIDATION TEST