

#69-F-230 M

BEAR CREEK

BRIDGE.

CONCESSION 5, 6/7

SIDEROAD.





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CONSULTING SOIL & FOUNDATION ENGINEERS

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KINGSTON 5, JAMAICA  
WEST INDIES

MONTEITH - INGRAM ENG. LTD.

PETROLIA

ONTARIO.

69-5-200 M

Report On

SOIL INVESTIGATION

FOR

BEAR CREEK BRIDGE

CONCESSION 5, 6/7 SIDEROAD

TOWNSHIP OF MOORE

by

DOMINION SOIL INVESTIGATION LIMITED.  
369 Queens Avenue

LONDON

ONTARIO.

Ref. No. 9-7-L1

## CONTENTS

	<u>PAGE</u>
SUMMARY.....	1.
I. INTRODUCTION.....	2.
II. FIELD WORK.....	2&3.
III. SUBSURFACE CONDITIONS.....	3,4&5.
IV. GROUNDWATER CONDITIONS.....	5.
V. DISCUSSION AND RECOMMENDATIONS...	5,6,7&8.
Appendix 'A', The Insitu Vane Shear Test.	

## ENCLOSURES

	NO.
LOCATION OF BOREHOLES & SUBSURFACE PROFILE	1.
BOREHOLE LOGS	2&3.
CONSOLIDATION TESTS	4&5.

SUMMARY

The natural soil profile consists of creek deposits, 3 to 4½ feet thick, overlying stiff to very stiff silty clay. Shale bedrock was encountered at EL. 517<sub>+</sub>, about 71 feet below the existing creek bed.

In the prevailing soil conditions the structure can be supported on spread footings or piles. Both types of foundation are technically satisfactory, and the design will be governed by cost, time and availability of materials.

For spread footings, maximum allowable soil pressures of 3000 p.s.f. and 6000 p.s.f. are recommended for the design of footings at EL. 582 and EL. 574 respectively. The estimated consolidation settlements are 2 and 1.5 inches for the respective horizons.

For a piled structure, end-bearing steel H-piles appear to be the most suitable for the site conditions. The working load will depend on the size of pile chosen, and settlement will be limited to the elastic compression of the pile system.

1. INTRODUCTION

In accordance with authorization from Monteith Ingram Engineering Limited, Consulting Engineers, a soil investigation has been carried out in the Township of Moore where it is proposed to replace an existing road bridge with a new structure.

The existing structure is located on the sideroad between lots 6 and 7 of the Township, where the road crosses Bear Creek.

It is understood that the proposed structure will be moved slightly south of the existing bridge, and that the total span will be about 150 feet. The requirements of the project were discussed with Mr. G. W. Ingram, P. Eng., who supplied the foregoing information.

The purpose of the investigation was to reveal the subsurface conditions at the new abutment locations and to determine the relevant soil properties for the design and construction of the new foundations.

II. FIELD WORK

The field work, consisting of two boreholes and two dynamic cone penetration tests, was carried out on July 24th and 25th and August 13th and 14th, 1969, at the locations shown on Enclosure 1. The boreholes were

notes are intended only to amplify this data.

Both boreholes penetrated silty clay fill deposits, which are associated with the construction of the approaches to the existing bridge. Underlying the fill, and between EL. 588 and EL. 583, the boreholes penetrated creekbed deposits consisting of sand, silt and organics.

Below EL. 584.3 at borehole 1 location, and EL. 583.5 at borehole 2 location, both boreholes penetrated an extensive clay deposit which extends to the surface of the shale bedrock at EL. 517<sup>+</sup>. The upper layers of the silty clay stratum have a 'stiff' consistency as indicated by insitu vane shear strengths ranging from 1500 to 1680 pounds per square foot, and unconfined compression shear strengths of 720 and 2250 p.s.f. Below EL. 575 the consistency of the silty clay is described as 'very stiff' based on insitu vane shear strengths ranging from 2400 p.s.f. to 4800 p.s.f., and unconfined compression shear strengths ranging from 3570 p.s.f. to 4400 p.s.f.

A series of laboratory tests were performed on disturbed and undisturbed samples of the silty clay stratum as a means of classification, and to obtain the compressibility characteristics. Four Atterberg Limit tests were performed, which gave values of Liquid Limit between 35% and 52%, and Plastic Limit between 18% and 24%. The

corresponding Plasticity Indices ranged from 14% to 28%, and on the basis of these findings the silty clay may be assumed to have a low to intermediate plasticity.

The moisture content was found to range from 16% to 36% and the Liquidity Indices from -0.1 to 0.4.

Two consolidation tests were performed to determine the compresibility characteristics of the silty clay. The results of these tests are presented in the form of void ratio-log effective pressure curves on Enclosures 4 and 5.

Shale bedrock was encountered at EL. 516.8 in borehole 1 and it was cored to a depth of 5 feet. An examination of the core recovery indicates that the shale is weathered to a depth of about 15 inches, and below the weathered layer the rock is in a sound condition. In borehole 2 refusal was encountered at EL. 518.5 and it may be assumed that bedrock was encountered at this horizon.

#### IV. GROUNDWATER CONDITIONS

The water level in borehole 1 reached equilibrium at EL. 591.0, which was 18 inches above the water level in the creek at the time the field work was carried out.

#### V. DISCUSSION AND RECOMMENDATIONS

The present level of the creek bed is at EL. 587.6, therefore under normal conditions spread footings would be placed at EL. 582<sub>+</sub>. Bedrock was encountered



at a depth of 65 feet below the normal footing grade which makes the use of end-bearing piles worthwhile considering.

(a) Spread Footings

On the basis of the borehole results, a maximum allowable soil pressure of 3000 p.s.f. is appropriate for the design of footings at EL. 582, and this soil pressure incorporates a factor of safety of 3 against shear failure of the underlying soil.

The long-term consolidation settlement which will occur below a 12 foot wide footing, mobilizing a dead load of 3000 p.s.f., is calculated to be 2 inches.

Alternatively, a higher soil pressure may be mobilized by lowering the footing grade to EL. 574. At this level a maximum allowable soil pressure of 6000 p.s.f. is appropriate for the design of footings, and long-term settlement of a 6 foot wide footing is estimated to be 1.5 inches.

The adhesion between the footings and the underlying clay may be taken as 1300 p.s.f. for footings at EL. 582, and 2000 p.s.f. for footings at EL. 574. The factor of safety against horizontal sliding of the abutments must be at least 1.5.

### (b) Piles

The use of friction piles supported in the silty clay stratum will not significantly reduce the settlement estimate of 1.5 inches, therefore end-bearing piles driven to bedrock will reduce the settlement to the elastic compression of the pile and also provide a more rigid foundation.

Due to the length of pile required, it would appear that steel H-piles are the most suitable to use due to their low resistance to penetration, and consequently they will penetrate further into the bedrock.

The following are typical working loads which are appropriate for the design of a steel H-pile foundation.

Size of Pile	Working Load (tons)
8 BP 36	60
10 BP 42	70
10 BP 57	100
12 BP 53	90

The above working loads are based on an allowable stress of 12,000 p.s.i., which is usually applied to piles driven to refusal in medium hard rock.

Many loading tests have proved that when an H-pile is driven to refusal on rock, the load at failure will

correspond to a stress in the steel in the range 35,000 to 40,000 p.s.i., which will provide a generous factor of safety against the 12,000 p.s.i. stress recommended.

The general rule for refusal of a pile is that 5 blows of an adequate hammer produces a total penetration of 0.25 inch. Driving should then cease, provided that the pile has not hit an obstruction and has been driven to a depth at which the borings indicate rock. When piles are driven to refusal on rock, pile loading tests are not generally considered necessary.

#### Construction

It is anticipated that the use of light sheeting will be required to contain the silt and sand strata immediately below the creek bed, and prevent a flow of soil and water into the excavations.

In the case of spread footing design it is recommended that the footing grade be covered with a concrete blanket to prevent disturbance as soon as it has been inspected and approved. Disturbed material below the proposed footing grade should be removed and replaced with lean concrete.



Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

*C.J.W. Atkinson*  
C.J.W. Atkinson, M.Sc., P. Eng.,  
Branch Manager.

## APPENDIX 'A'

### INSITU VANE SHEAR TEST

In soft to stiff clays, and particularly sensitive clay soils such as frequently occur in alluvial deposits, it is difficult to obtain reasonable undisturbed samples for the determination of the undrained shear strength. In order to overcome this difficulty, the vane test was developed as an in-situ method of measuring the shear strength.

The apparatus consists of a 4-inch long by 2-inch wide rectangular 4-bladed rotating vane attached to a thin rod, which is pushed into the undisturbed soil below the bottom of the bore-hole to the depth at which the test is to be made.

A torque is then applied to the vane and the maximum torque when failure occurs is recorded. The vane is then rotated 10 times to remould the soil and after one minute the torque test is repeated. The shear strength of the soil can then be calculated from the torque and the dimensions of the vane, and the sensitivity of the material estimated from the ratio of the original torque to the final torque after remoulding.

# LOG OF BOREHOLE.....!

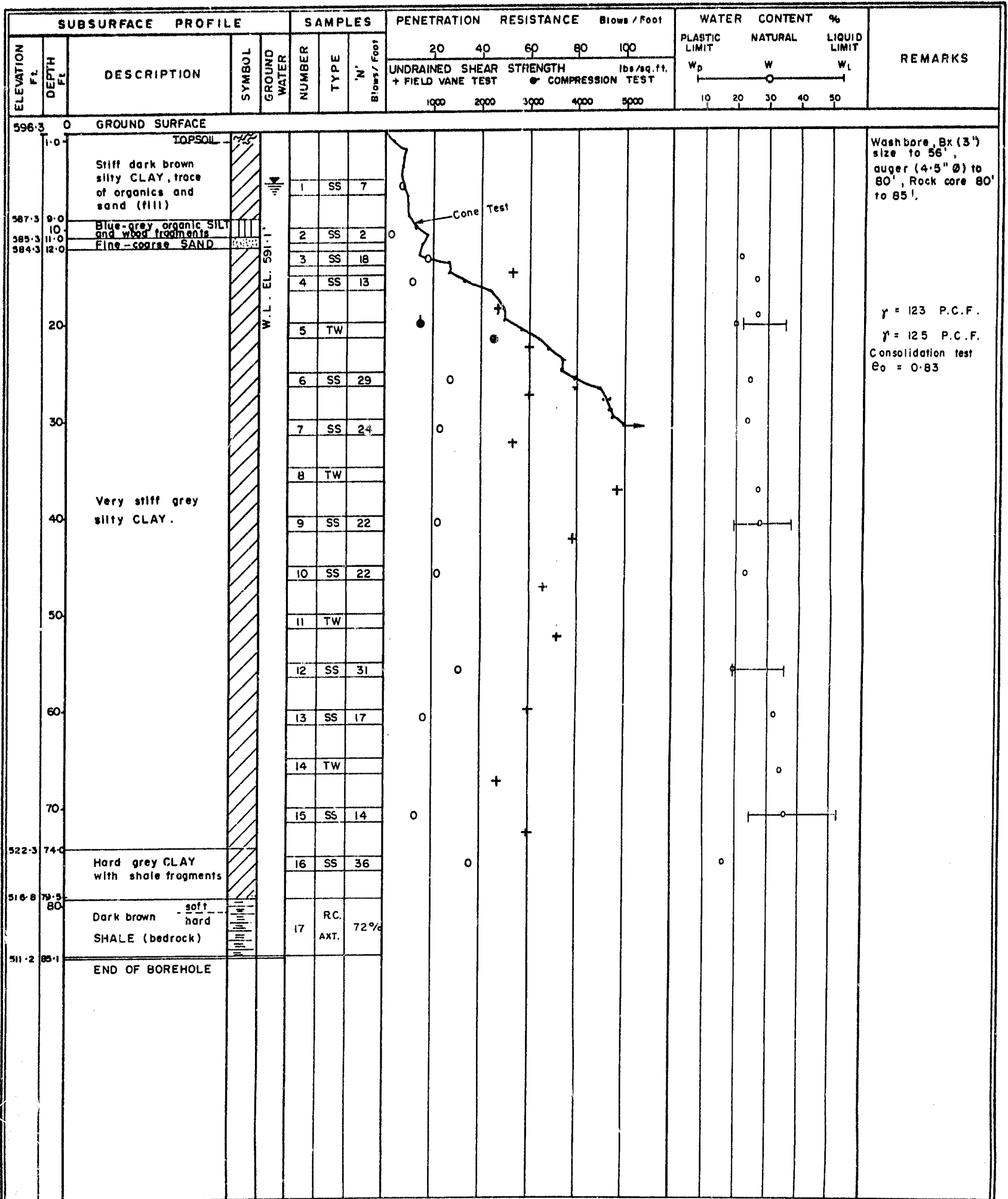
Our Reference No. 9-7-L1

Enclosure No. ....

CLIENT: MONTEITH INGRAM ENGINEERING LTD.  
PROJECT: BRIDGE  
LOCATION: BRIDGEN, ONTARIO.  
DATUM ELEVATION: LOCAL.

## DRILLING DATA

Method: DRY & WASHBORING  
Diameter: 3" (Bx)  
Date: JULY. 24-25, AUG. 13-14, 1969



VERTICAL SCALE: 1 inch to 10 feet

DOMINION SOIL INVESTIGATION LIMITED

MADE: Z. A. CHECKED:

## LOG OF BOREHOLE...2.....

Our Reference No. 9 - 7 - L1

Enclosure №.....

CLIENT: MONTEITH INGRAM ENGINEERING LTD.

PROJECT: BRIDGE

LOCATION: BRIDGEN, ONTARIO.

DATUM: ELEVATION: LOCAL.

### DRILLING DATA

#### Method: AUGERING

Diameter: 4.5 "

Date: JULY . 24 / AUG. 14 , 1969

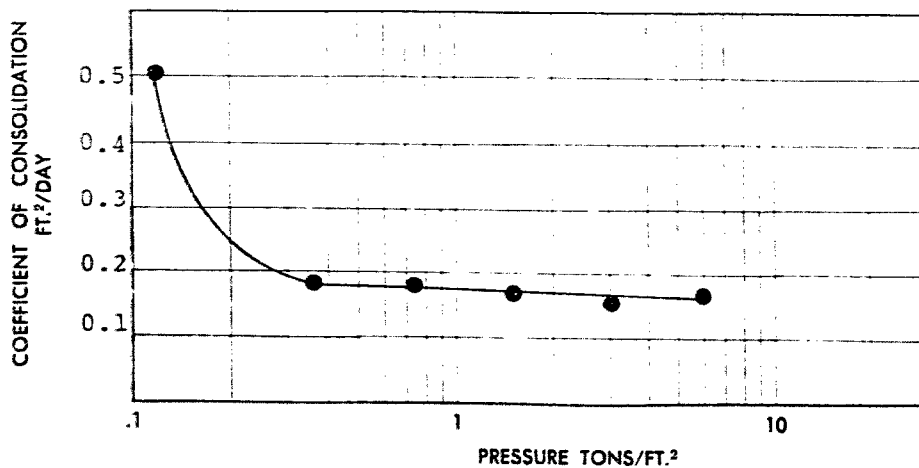
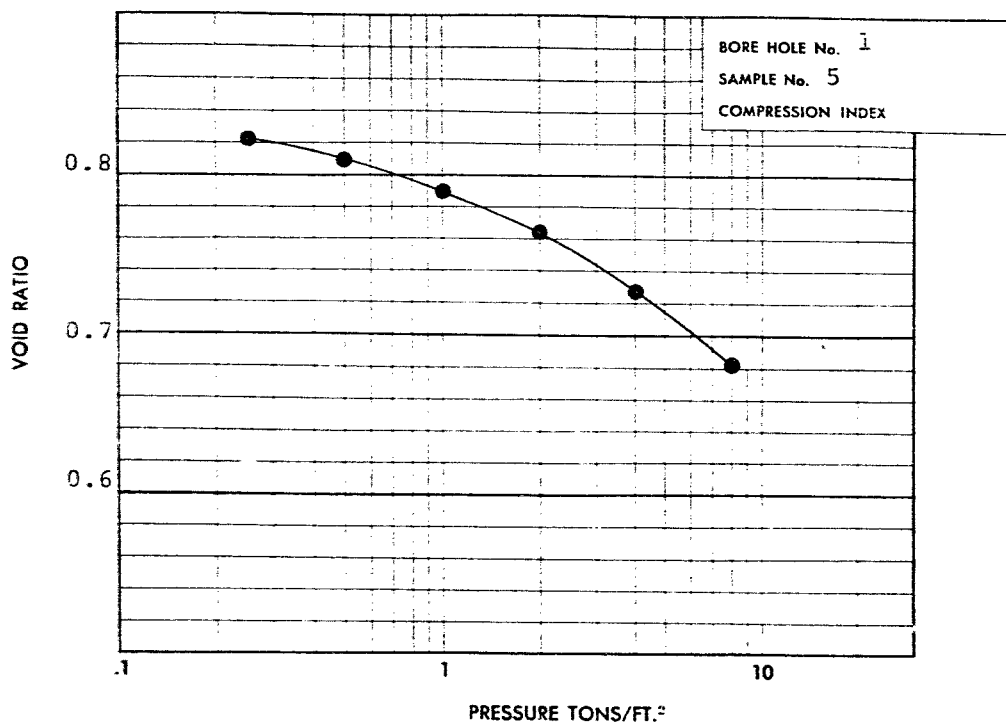
SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			REMARKS						
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows / Foot	Blows / Foot					PLASTIC LIMIT	NATURAL		LIQUID LIMIT					
								20	40	60	80	100	W <sub>p</sub>	W		W <sub>L</sub>					
								UNDRAINED SHEAR STRENGTH									lbs/sq.ft.				
								+ FIELD VANE TEST									● COMPRESSION TEST				
								1000	2000	3000	4000	5000	10	20	30	40	50				
678.0	0	GROUND SURFACE																			
		Stiff grey brown silty CLAY FILL			1	SS	5	0											Auger (4.5" Ø) to 64', washbore without sampling to 89.5'		
					2	SS	13	0													
597.0	10.0	Stiff brown silty CLAY			3	SS	22	0													
					4	SS	14	0													
588.0	20	Dark brown fine coarse SAND and decomposed wood.			5	SS	13	0													
583.5	24.5				6	SS	9	0	+												
	30	stiff v. stiff			7	TW		0	+												
					8	SS	22	0													
	40	Grey silty CLAY			9	SS	31	0													
	50				10	TW															
					11	SS	28	0													
	60				12	SS	28	0													
544.0	64.0				13	SS	30	0													
	70	Clay continues to 89.5' where very dense stratum, assumed to be bedrock, was encountered.																			
	80																				
516.5	89.5	END OF BOREHOLE																			

γ = 128 P.C.F  
γ = 124 P.C.F  
Consolidation test  
e<sub>0</sub> = 0.56

VERTICAL SCALE: 1 inch to 10 feet

**DOMINION SOIL INVESTIGATION LIMITED**

MADE: Z. A.      CHECKED:

**Dominion Soil Investigation Ltd.****CONSOLIDATION TEST**

**Dominion Soil Investigation Ltd.****CONSOLIDATION TEST**