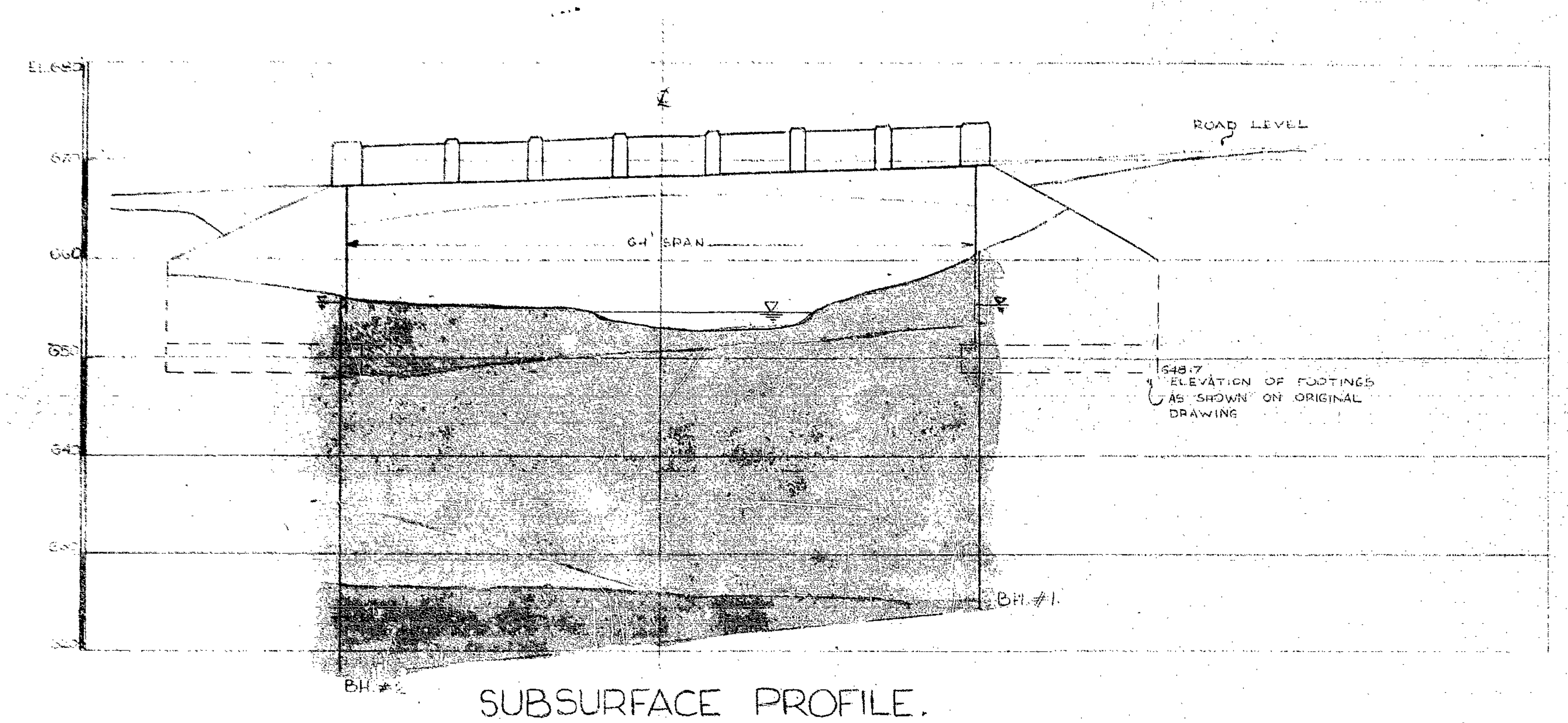
#60-F-223
W.P.#301-60-3
PROP. BRIDGE
WIDENING
CON. #4 & #5
TUSCARORA TWP.

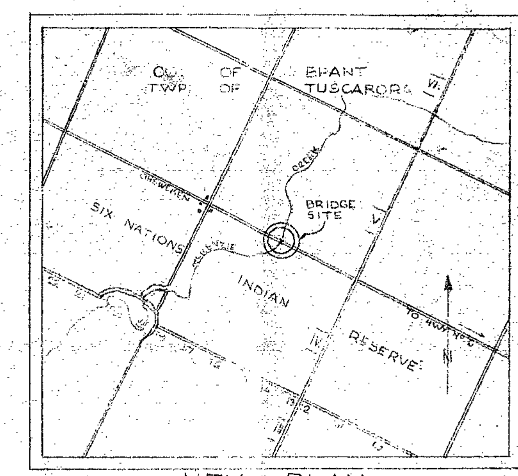
572900 E
4768050 N

17

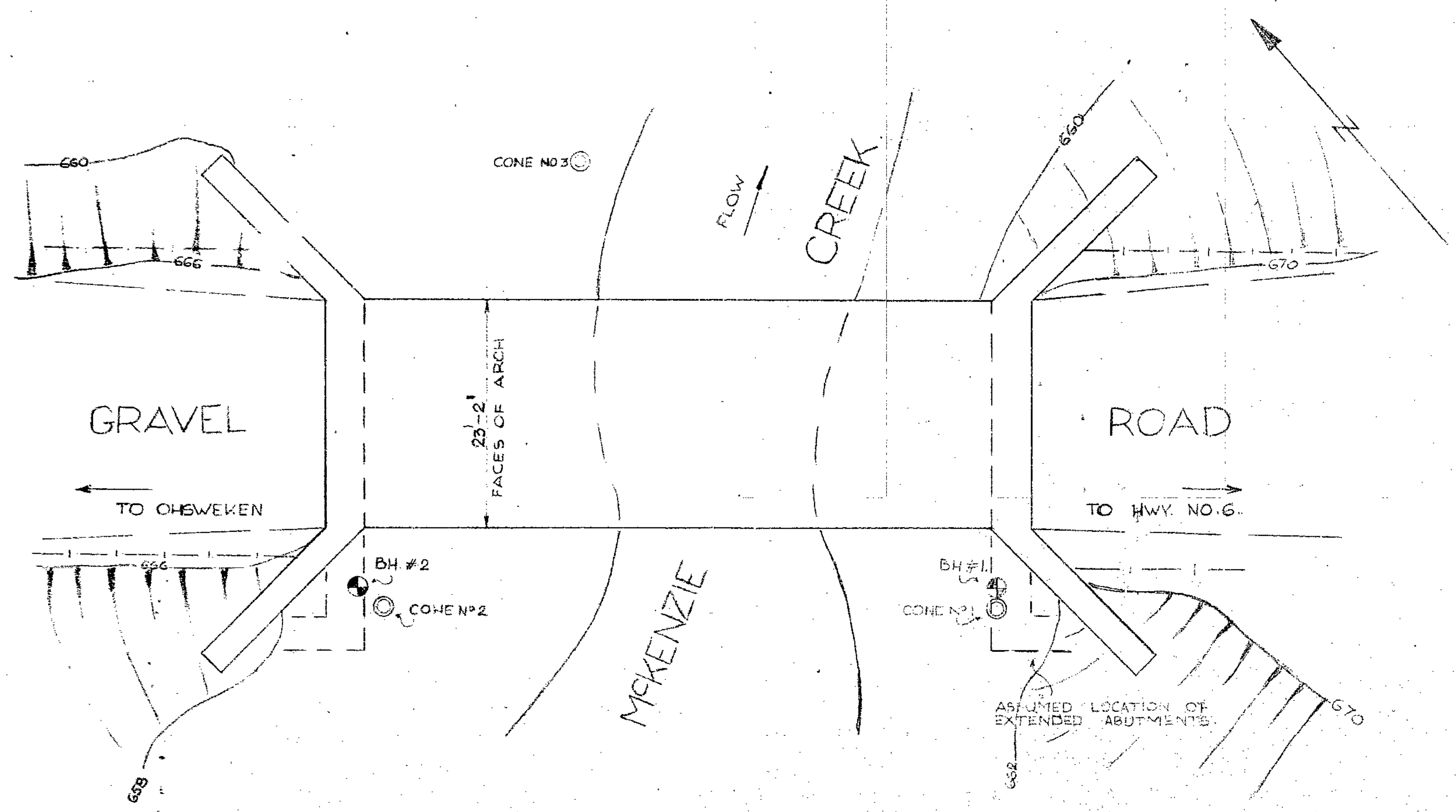
40P1 E



SUBSURFACE PROFILE.
SCALE: 1 INCH TO 10 FEET



KEY PLAN



LOCATION PLAN
SCALE: 1 INCH TO 10 FEET

LEGEND

- BOREHOLE
- CONE
- BROWN CLAY, SOME SAND SILT AND ORGANICS.
- SOFT TO MEDIUM STIFF BROWNISH GREY CLAY
- GRAVEL, SHALE AND CLAY
- BLUE SHALE
- WATER LEVEL, NOVEMBER, 1960.

OUR REF. No. Q-10-21	DEPARTMENT OF HIGHWAYS MATERIALS AND RESEARCH SECTION. DOWNSVIEW ONTARIO
ENCL. No. 1. DATE NOV. 1960	PROPOSED WIDENING OF OBEDIAH BRIDGE OVER McKENZIE CREEK, IN SIX NATIONS INDIAN RESERVE LOT 15 & 16 CON. IV & V. TWP. TUSCARORA IND.RES. 40 COUNTY OF BRANT
DRAWN BY E.L. Checked BY J.P.	DOMINION SOIL INVESTIGATION LIMITED 88 EGLINTON AVENUE EAST TORONTO 12 ONTARIO

cc: Gen. Files

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

December 19, 1960.

FOUNDATION INVESTIGATION REPORT

by: Dominion Soil Investigation, Ltd

Attention: Mr. S. McCombie.

Re: Proposed Widening of Obediah Bridge,
Lots 15 & 16, Con. IV & V, Tuscarora Twp.,
Brant Co., District No. 4 -- (W.P. 301-60-3.)

Attached to this memo, we are forwarding to you the above mentioned report submitted by the Consultant, Dominion Soil Investigation, Ltd. We have reviewed the presented field and laboratory data, and also the conclusions and recommendations, which we find correct and satisfactory.

However, we are of the opinion that spread footings could be used for the new structure. The computed settlement values present the upper limit and most probably, will be less and as such, will not be detrimental for the existing structure.

When the old structure will start to tilt, it will lean against the new adjacent one and resistance will be encountered, and some of the tilting prevented. The overall effect on the old structure will be small and therefore, spread footings are recommended.

Should there be any other questions that you would like to discuss, please feel free to contact our Office.

AGS/MdeF
Attach.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
I. C. Campbell
E. E. Richardson
T. J. Kovich
A. Watt

Foundations Office
Gen. Files.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.
Per:

Stermac

(A. G. Stermac,
FOUNDATION OFFICE ENGR.)

23-61-106-L
62-69

DEPARTMENT OF HIGHWAYS
MATERIALS & RESEARCH SECTION
DOWNSVIEW ONTARIO

REPORT ON FOUNDATION INVESTIGATION FOR PROPOSED WIDENING OF OBEDIAH BRIDGE

LOTS 15 & 16, CON. IV & V TUSCARORA TWP., BRANT CO.
DISTRICT NO. 4
W.P. 301 - 60 - 3

Submitted by

DOMINION SOIL INVESTIGATION LIMITED
88 EGLINTON AVENUE EAST
TORONTO 12 ONTARIO

REFERENCE NO. 0-10-31
NOV. 1960

CONTENTS

ENCLOSURES

HUdson 7-3633

88 Eglinton Ave. East
Suite 312

DOMINION SOIL INVESTIGATION LTD.

SOIL MECHANICS • FOUNDATION ENGINEERING

TORONTO 12, ONTARIO

INTRODUCTION

Authorization was received in a letter dated 28th October, 1960 from the Department of Highways, Materials & Research Section, to conduct a soil investigation adjacent to Obediah Bridge, 0.8 miles east of Ohsweken. The location of the site is shown in the Key Plan on Encl. No. 1. The work is connected with the proposed widening of the 64' span concrete bridge which carries a gravel road across McKenzie Creek. A drawing of the site showing the position of the new section was supplied by the Client (D.H.O. Plan E-3878-1).

The purpose of the investigation was to reveal the subsurface conditions and determine the necessary soil properties for the design and construction of foundations.

I. DESCRIPTION OF SITE & GEOLOGY

The site lies 14 miles south-east of Brantford and 4 to 5 miles south of the Grand River in the Tuscarora Indian Reserve. The surrounding country undulates between elevations 650' and 700'. It is covered with thin forest and scrub, and there is some farming. The McKenzie Creek, which meanders through the area, drains eastwards into the Grand River in the adjacent Haldimand County.

The site lies at the western extremity of the Haldimand clay plain which was submerged by the previous Lake Warren and is underlain by sedimentary rocks of the Salina Formation. The bedrock contains seams of gypsum which is mined in nearby Caledonia and Hagersville.

II. FIELD WORK

Field work was carried out during the period 1st to 5th November, 1960, and comprised 2 boreholes and 3 dynamic cone penetration tests at the locations shown on Enclosure No.1. The holes were wash bored and lined with Bx casing.

The position of samples and the results of standard penetration and dynamic cone tests are recorded on the Engineering Data Sheets, Enclosures 2 to 4. The samples comprised the following:

(a) Split-spoon samples using a standard 2" O.D. sampler.

(b) Undisturbed samples using 2" diameter Shelby tubes which were pushed into the clay without difficulty.

(c) Ax core from the bedrock in Borehole No. 2. The hardness of the bedrock varied considerably due, it is believed, to the presence of seams of gypsum, and it is probably for this reason that the core recovery did not exceed 35%.

The insitu and remoulded strength of the clay was measured with a 2" diameter vane, and the unconfined compressive strength of samples was checked with a pocket penetrometer.

It was not possible to wash bore through the 9 ft. thick layer of dense granular material overlying the bedrock in Borehole No. 2 so that the Ax core barrel had to be run before advancing the casing. The presence of this layer, or its absence in Borehole No. 1, prompted the performance of a third dynamic cone test on the other side of the bridge (see location on Enclosure No. 1.)

III. LABORATORY WORK

A laboratory testing programme was set up to provide data for the computation of bearing capacity and settlement for spread footings, and to assist in classifying the strata.

Atterberg limits were determined for 4 samples and unconfined compression tests were made on 3 of them. The results are shown on the Engineering Data Sheets, Enclosures 2 & 3, and are summarized below. A sieve analysis was made of one sample of the granular material in Borehole No. 1 and the grading curve is given on Enclosure No. 5.

B.H.#/Sample#	1/3	1/6	1/10	2/5	2/8
Elevation (feet)	654.5	645.3	631	641.5	633.7
% Passing No. 200 Sieve	99.9	99.7	99.0	98.9	26.5
Natural Moisture (%)	31.9	28.5	39.5	32.2	10.2
Liquid Limit (%)	46.5	31.1	49.0	46.0	-----
Plastic Limit (%)	22.2	13.4	21.4	20.4	-----
Plasticity Index (%)	24.3	17.7	27.6	25.6	-----
Consistency Index	0.60	0.15	0.34	0.54	-----
Unconfined Compressive Strength (p.s.f.)	-----	2460	1900	1800	-----

Consolidation tests were made on 2 samples of the clay. 24 hour loading cycles were used. The curves for compression and coefficient of consolidation are shown in Enclosures Nos. 6 and 7, and the results are summarized as follows:

B.H#/Sample#	Elev.	Compression Index	e_o	Preconsolidation Load (t.s.f.)
1/6	645.3'	0.149	0.759	2.25
1/10	631.0	0.337	1.072	1.75

IV. SUBSURFACE CONDITIONS

The subsurface profile is shown on Enclosure No. 1, and contains the following layers:

(a) a soft to medium stiff weathered brown clay of intermediate to low plasticity containing fine sand silt and organics. It varies in thickness up to 8 feet.

(b) a soft to medium stiff brownish grey clay of intermediate to low plasticity and low sensitivity. It varies in thickness from 28 feet in Borehole No. 1 to 13 feet in Borehole No. 2. A trend of decreasing shear strength with depth was observed in tests with the vane, pocket penetrometer and unconfined compression apparatus, which is in agreement with the observed higher moisture content and compression index nearer the bottom of the layer.

(c) in Borehole No. 2 only, a very dense layer of gravel, sand and shale fragments in a matrix of clay. This may be the edge of a buried drumlin which is not an uncommon feature in the area.

(d) bedrock consisting of hard blue shale was located around elevation 626'.

Groundwater was encountered in the Boreholes at approximately the level of water in the Creek, viz. El. 655'. In Borehole No. 1 continuous baling could not lower it below this level and in Borehole No. 2 the level was reached about one hour after baling. This demonstrates the perviousness of the upper layers of the soil which is attributable to weathering and the presence of granular and organic material.

V. DISCUSSION AND RECOMMENDATIONS

1. Use of Spread Footings

The existing bridge is of rigid frame construction and is believed to rest on spread footings 5'-6" wide at El. 648.7'. It shows no sign of differential settlement and appears to have functioned satisfactorily since its completion in 1934. The practicability of using spread footings for the new portion has therefore been examined first as the cheapest solution.

(a) Bearing Capacity

In estimating bearing capacity a value for cohesion of 1000 p.s.f. has been used on the basis of vane tests, unconfined compression tests and blow count. (On the basis of vane tests alone this value is conservative).

The ultimate bearing capacity of a 12.55' x 5.5' footing at El. 648.7' is then, according to Meyerhof, 9170 p.s.f. The soil pressure under the existing footing is estimated to be 4500 p.s.f., indicating a factor of safety in the region of 2. For a safety factor of 3, the maximum soil pressure on this size of footing would be 3000p.s.f.

(b) Consolidation settlement

In considering settlement the soil pressure of 4500 p.s.f. under the existing structure has been used as a limiting case. This does not lead to large values, and settlements for lower pressures can be obtained with sufficient accuracy by linear interpolation.

The effect on the existing structure is considered first. Using the Newmark method, the stress under the existing footings due to the adjacent loading has been computed. It is greatest immediately adjacent to the new section, and of negligible amount at the remote end of the footing. The result will therefore be a tilt of the following vertical magnitude:

Computed Consolidation Settlement[1]		2.09 inches
Corrected Settlement[2]		1.0 to 1.3 inches
Time	50%	5 months
	90%	15 months

1 From consolidation test results.

2 Using Skempton & Bjerrum's correction factor modified by Wood. See references.

The overconsolidation ratio lies between 2 and 3. The range of values given for corrected settlement correspond to values for the pore pressure coefficient A between 0.2 and 0.4. The soil is fully saturated so that coefficient B has been taken as 1.

In calculating settlement under the new footings, the thickness of the compressible clay layer has had to be considered. Under the east abutment where it is 24 feet thick the vertical stress under the "characteristic point" (0.74 a/2, 0.74 b/2 as defined by Kany) has been calculated according to Steinbrenner and the following values obtained:

Computed Consolidation Settlement		2.40 inches
Corrected Settlement		1.25 to 1.5 inches
Time	50%	5 months
	90%	15 months

Under the west abutment where the clay is only 13 feet thick it cannot be treated as semi-infinite. The vertical stress has therefore been calculated according to Tzutovich for a layer of finite thickness overlying a hard stratum. The settlement is thus found to be:

Computed Consolidation Settlement		3.22 inches
Corrected Settlement		1.8 to 2.1 inches
Time	50%	5 months
	90%	15 months

The higher settlement values in the thinner layer are a result of a lesser reduction of stress with depth and a higher net soil pressure because there is less overburden on the east side.

(c) Immediate settlement

The immediate settlement under the new footings due to elastic deflection has been calculated using a value for Young's Modulus E of 35 tons per square foot. This is taken from the stress/strain curves of the unconfined compression tests which give the following values:

Sample No.	Unconfined Strength (t.s.f.)	E (t.s.f.)
1/6	1.23	10.5
2/5	0.90	10.8
1/10	0.95	34.5

According to Skempton the value E is very sensitive to sample disturbance and the upper end of a range of scattered values should be taken. In this case, with only 3 results a value of 35 t.s.f. for E is probably low. The resulting average deflection according to formulae given by Timoshenko & Goodier (based on Schleicher) is 3 inches.

This immediate settlement would probably cause a discontinuity in the surface between the old and new footings, but would also tend to increase the estimated tilt of the existing structure caused by consolidation settlement.

The effect of settlement on the new section would probably not be serious, but the possible effects of tilting on the existing rigid frame and differential settlement between the two portions excludes the use of spread footings as a satisfactory solution.

2. <u>Use of Piles</u>

To cause the least possible disturbance to the existing structure and minimize differential movement between the old and new sections, the proposed new construction should be piled. Treated timber piles complying with AASHO specifications will be suitable for 20 ton working loads. This will necessitate approximately 14 piles under each abutment. These will function as point bearing piles and satisfactory sets will be found approximately at the following elevations:

East abutment	El. 625'
West abutment	El. 635'

The pile caps should be located not higher than El. 650'.

3. <u>Construction</u>

At the east end of the bridge the wing walls have rotated forward approximately 2 inches at the top because of pressure from the approach fill behind. If excavation is made in front of them for the pile caps this movement may be accelerated, and so the walls should be braced before excavation begins. If this is done no stability problem is envisaged in the excavation itself, although light bracing may be required to prevent spalling of the top few feet if the walls of the cut are vertical.

It has been observed that the top layer of brown clay is pervious and it is recommended that the creek be diverted through a temporary culvert to carry away the bulk of the water. Pumping may still be necessary to keep the excavation dry.

VI. SUMMARY

1. The strata consist first of a top layer of brown weathered clay up to 8 feet in thickness containing some sand, silt and organics. Below this under the east abutment is a layer of soft to medium stiff brownish grey clay 28 feet thick overlying bedrock which is hard blue shale at El. 626' (approx.). Under the west abutment a layer of dense granular material 9 feet thick lies on top of the bedrock, reducing the thickness of the overlying clay layer to 13 feet.

2. Groundwater was located at about El. 655', and water was observed to move easily through the top layer of brown weathered clay.

3. The existing rigid frame structure on spread footings has functioned satisfactorily, although its factor of safety on bearing capacity for an estimated soil pressure of 4500 p.s.f. appears to be about 2. The use of spread footings cannot be recommended for the new section, however, because of the effect of tilting on the existing rigid frame and differential movements between the two sections.

4. Piles will be the most satisfactory form of foundation for the new section. Treated timber piles complying with AASHO specifications will be suitable for 20 ton working loads. Satisfactory sets will be found near El. 625' under the west abutment and El. 635' under the east abutment.

5. The existing wing walls should be braced before any excavation is made in front of them. Only light bracing will then be required if the sides of the excavation tend to spall.

6. Because of the perviousness of the top stratum the creek should be diverted through a culvert during construction. Pumping may still be necessary to keep the excavations dry.

REFERENCES:

1. Chapman, Putnam: The Physiography of Southern Ontario, Toronto, 1951.

2. Meyerhof: The Ultimate Bearing Capacity of Foundations, Geotechnique, Vol. II, 1950 and 51. p. 301 et seq.

3. Skempton, Bjerrum: A Contribution to the Settlement Analysis of Foundations on Clay. Geotechnique, Vol. VII, 1957, p. 168 - 178.

4. Muir Wood: Correspondence, Geotechnique Vol. IX, March, 1959.

5. Steinbrenner and Kany: From Mitteilungen aus dem Institut fuer Verkehrswasserbau, Grundbau und Bodenmechanik der Technischen Hochschule Aachen. Heft 19, Aachen 1959.

6. Tsutovich: From Szechy, Foundations (in Hungarian) Volume 1, Budapest, 1957.

7. Timoshenko & Goodier: Theory of Elasticity, 2nd Edition, McGraw Hill, 1951.

DOMINION SOIL INVESTIGATIONS LIMITED

James Park

November, 1960.

James Park, P.Eng.

E n c l o s u r e s

Order No. O-10-21 Enclosure No. 2.

No. O-11-21 Enclosure No. 2.

Dominion Soil Investigation Ltd.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Date: 1-2 NOV. 1960. / 1-2 Nov. 1960

Project: McKENZIE CREEK - OBEDIAH BRIDGE
Location: BRANT CO. TUSCARORA TWP. CON. IV & V. LOTS 15 & 16
Hole Location: SEE ENCL. 1.
Hole Elevation and Datum: 660.8
Field Supervisor: J.P. Prep.: E.L.
Driller: N.O Checked: J.P.

LEGEND

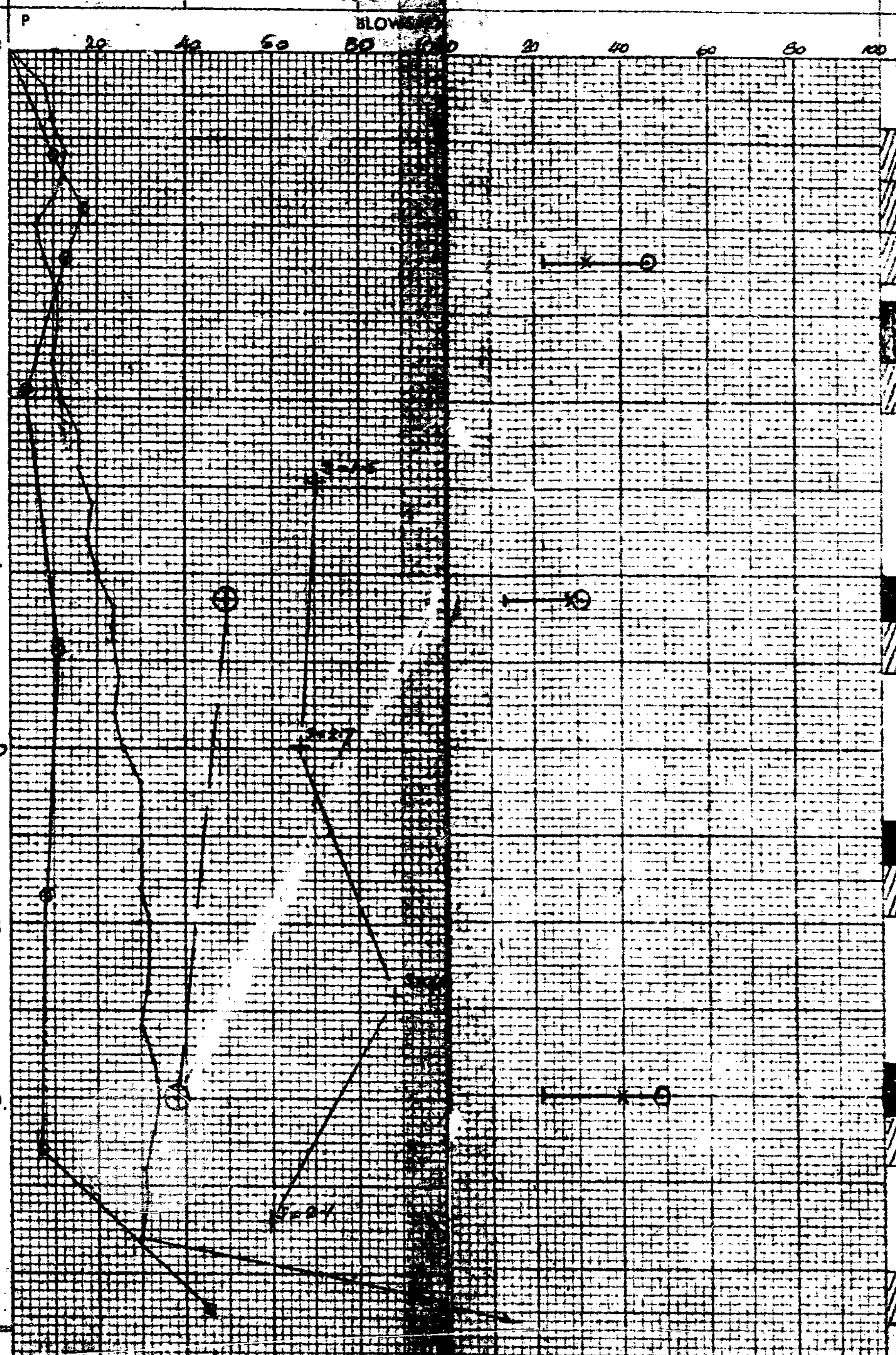
Shear Strength (C)
- Unconfined compression ⊕
- Vane test and sensitivity (S) +S

Penetration Resistance (P)
- 2" Split tube
- 2" Dia. Cone
- Casing

Consistency
- Natural moisture and Liquidity Index (LI) × LI
- Liquid limit
- Plastic limit

Sampling Method
- 2" Dia. split tube
- 2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	CONSISTENCY / MOISTURE CONTENT % DRY WEIGHT	SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS
	GROUND LEVEL	660.8						
	GRASS & ROOTS							
	BROWN CLAY ▽ W.L.		5			1. 2. 3. 4. 5.		SPLIT SPOON AND CONE PENETRATION TEST: HAMMER WEIGHT : 140 lbs. DROP : 30 in.
	BROWNISH GREY CLAY	650	10			6.	124.6	
		640	15, 20			7. 8. 9.		
		630	25, 30			10. 11.	113.7	
	BLUE SHALE.		35			12.		
	END OF BOREHOLE	624.2						



Order No. 0-10-21

Enclosure No. 3 No.

Enclosure No. 3

Dominion Soil Investigation Ltd.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

Date: NOV. 2-4. 1960.

Date: Nov. 2-4, 1960

Project: McKENZIE CREEK OBEDIAH BR.
Location: BRANT CO. TUSCARORA TWP.
Hole Location: SEE ENCL. I. CON, IV & V. LOT 15 & 16
Hole Elevation and Datum: 656.2.
Field Supervisor: J.P. Prep.: E.L.
Driller: N.O. Checked: J.P.

LEGEND

Shear Strength (C)
- Unconfined compression ⊕
- Vane test and sensitivity (S) $+^S$

Penetration Resistance (P)
- 2" Split tube
- 2" Dia. Cone
- Casing

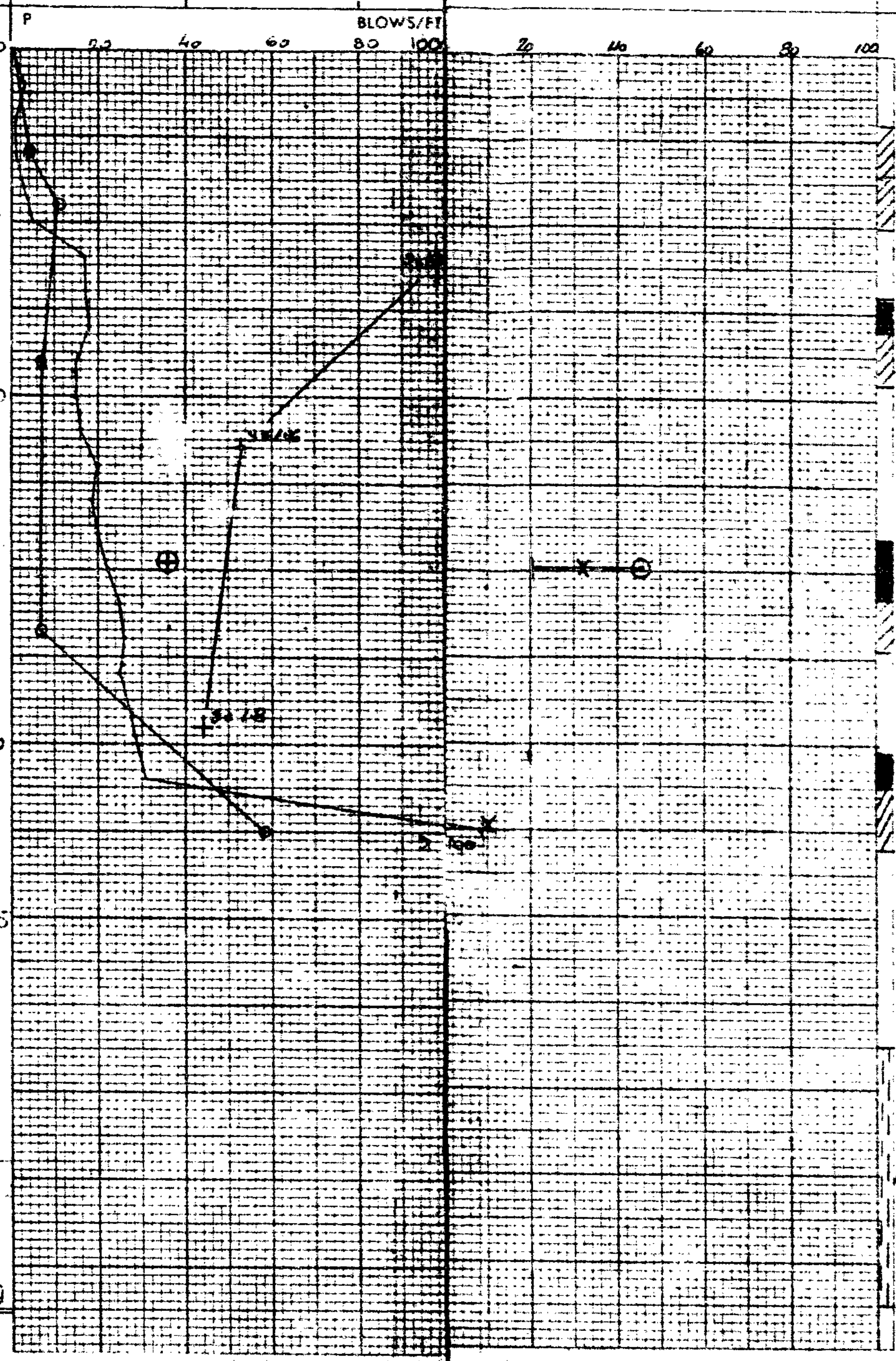
Consistency
- Natural moisture and Liquidity Index (LI) × LI
- Liquid limit
- Plastic limit

Sampling Method
- 2" Dia. split tube
- 2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS
	GROUND LEVEL	656.2	0			
	GRASS & ROOTS ▽ W.L.					
	BROWN CLAY CONTAINING FINE SAND, SILT & ORGANICS			1.		
	BROWN SILTY CLAY	650	5	2.		SPLIT SPOON AND CONE PENETRATION TEST: HAMMER WEIGHT: 140 lbs. DROP: 30 in.
	BROWNISH GREY CLAY	640	10 / 15 / 20	3. 4. 5. 6.	119.9 (sample 5)	
	GRAVEL, SHALE AND CLAY.	630	25	7. 8.		
	BLUE SHALE	626.5		Ax core		
	END OF BOREHOLE.	617.5	38.9			

STRENGTH AND PENETRATION RESISTANCE

CONSISTENCY



Order No. O-10-21 Enclosure No. 4.

Dominion Soil Investigation Ltd.

Engineering Data Sheet for ~~Borehole:~~ CONE TEST 3

Date: NOV. 3. 1960.

Project: McKENZIE CREEK - OBEDIAH BR.
Location: BRANT CO. TUSCARORA TWP CON. IV
Hole Location: SEE ENCL. I. & V. LOT 15&16
Hole Elevation and Datum:
Field Supervisor: J.P. Prep.: E.L
Driller: N.O. Checked: J.D.

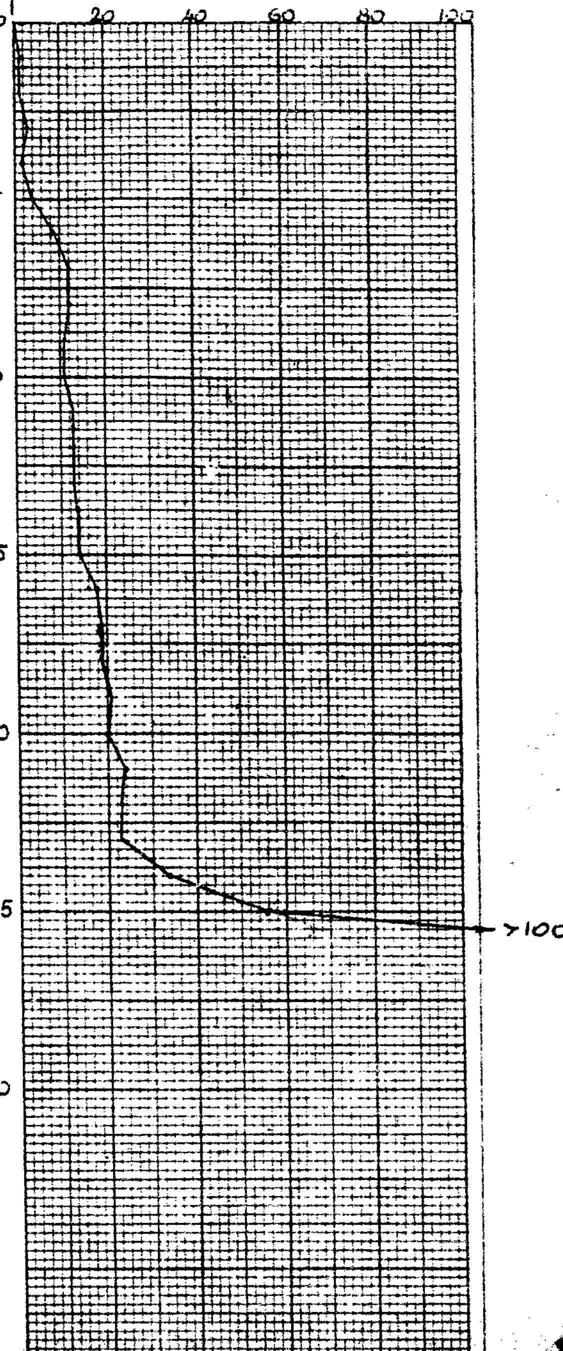
LEGEND

Shear Strength (C)
- Unconfined compression ⊕
- Vane test and sensitivity (S) $+^{S}$

Penetration Resistance (P)
- 2" Split tube
- 2" Dia. Cone
- Casing

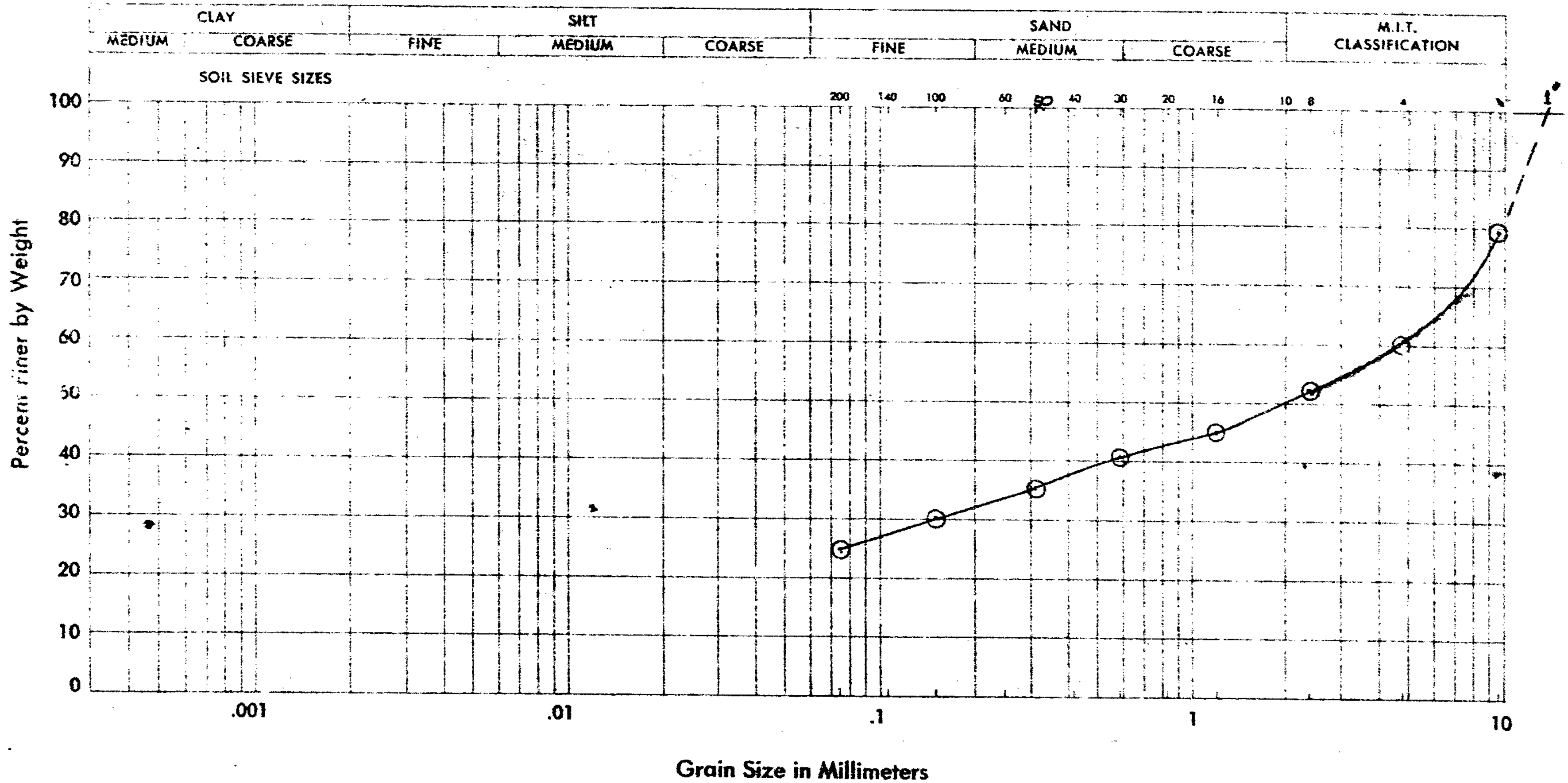
Sampling Method
- 2" Dia. split tube
- 2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE C P.S.F. P BLOWS/FT.
		656.2	0	
			5	
		650		
			10	
			15	
		640		
			20	
			25	
		630		
			30	



Dominion Soil Investigation Ltd.

GRAIN SIZE DISTRIBUTION



Project: OBEDIAH BRIDGE WIDENING.

Order No. O – 10 – 21
BOREHOLE No. 2.
SAMPLE No. 8.
DEPTH 22.5'
ELEVATION 633.7'

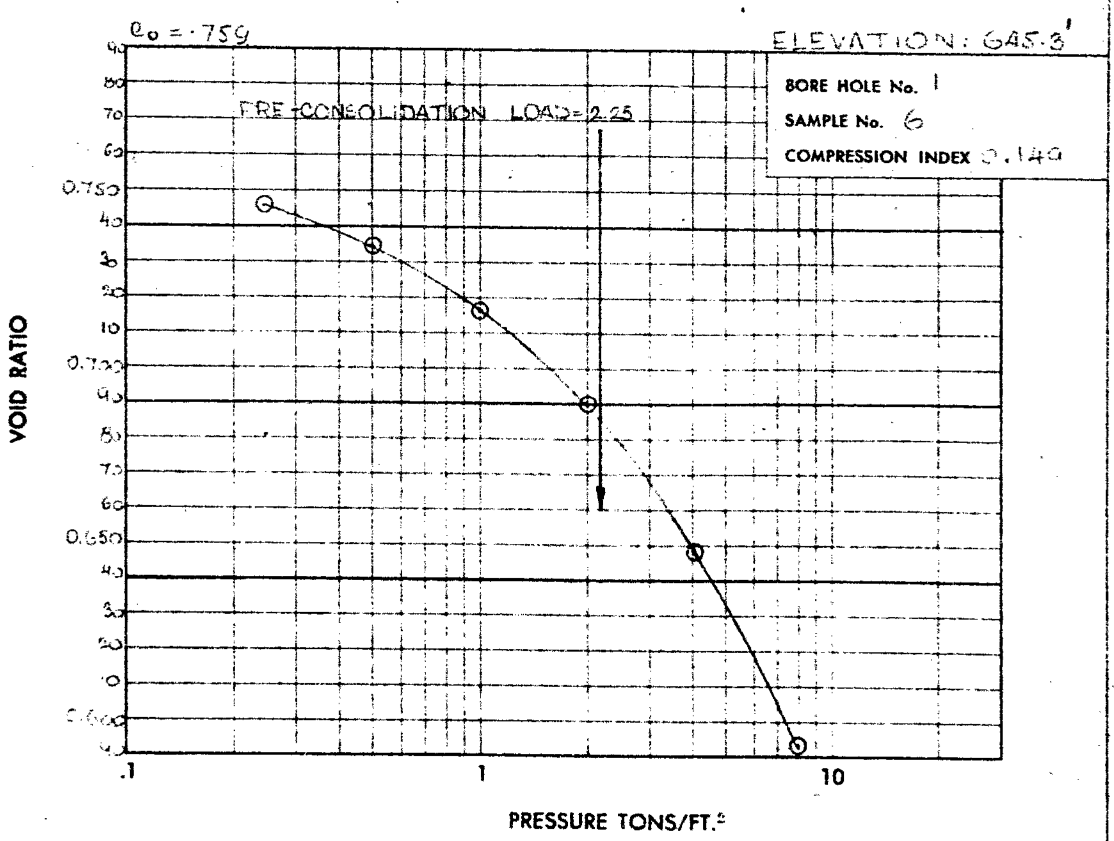
Enclosure No. 5.

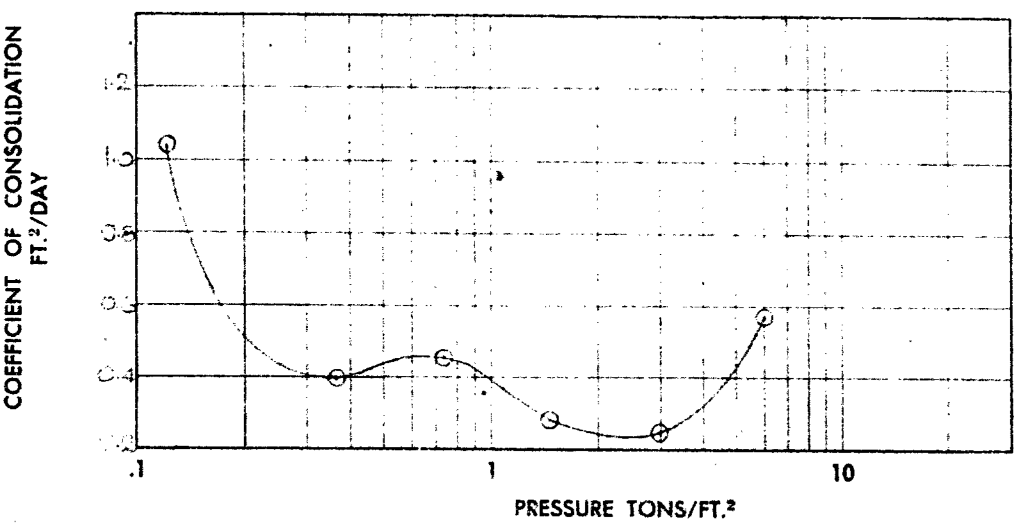
Order No. O-10-21

Enclosure No. 6

Dominion Soil Investigation Ltd.

CONSOLIDATION TEST





Order No. 0-1021

Enclosure No. 7

Dominion Soil Investigation Ltd.

CONSOLIDATION TEST