

64-F-258 m

DEV. ROAD #733

AUSABLE RIVER

Hwy. 21

BOSANQUET

TWP.

GIVEN BY JOHN ROY
NOV 13. 1964

DEPARTMENT OF HIGHWAYS, ONTARIO
REGIONAL MATERIALS & TESTING DIVISION
LONDON ONTARIO

Report on
SOIL INVESTIGATION
for
DEVELOPMENT ROAD 733
AUSABLE RIVER TO HIGHWAY NO. 21

CO. LAMBTON
Twp. BECAUQUE

(Consultant)

64 - F 258 M

by
DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO
Reference No. 4-9-L2
November 6th, 1964

CONTENTS

	<u>Page</u>
SUMMARY.....	1
I INTRODUCTION.....	2
II SITE AND GEOLOGY.....	2
III FIELD AND LABORATORY PROCEDURES.....	2 and 3
IV SUBSURFACE CONDITIONS.....	3 and 4
V GROUNDWATER CONDITIONS.....	4
VI DISCUSSION.....	4 to 6
VII REFERENCES.....	7

ENCLOSURES

	<u>No.</u>
SYMBOLS, ABBREVIATIONS AND NOMENCLATURE	1
SUBSURFACE PROFILE	2
GEOTECHNICAL DATA SHEETS	3 to 6
CONSOLIDATION TESTS	7 and 8
SUMMARY OF LABORATORY TEST DATA	9
SUMMARY OF SHEAR STRENGTH VALUES	10

SUMMARY

Four exploratory boreholes revealed the presence of very soft and highly plastic clay deposits varying in thickness from 8.5 to 28.0 feet. The clay is underlain by a compact to dense fine sand.

The depth to groundwater varied from 0 to 3 feet below the ground surface at the time the borings were made.

The shear strength of the soil is such that considerable sinkage is anticipated below the proposed road embankment. It is estimated that the embankment will settle to the top of the sand layer, displacing the clay, over most of the length of the road, and that even where the clay is deepest the embankment can be induced to settle to the bottom with relatively little additional effort. This method of construction, i.e. complete displacement of the soft clay deposits, is recommended for the project.

Some aspects of removing the soft clay by excavation are discussed briefly.

I INTRODUCTION

In accordance with a letter of authorization dated 3rd of September, 1964 from the Department of Highways, Materials & Testing Division, a soil investigation has been carried out on the line of proposed Development Road 733 between the Ausable River and Highway No. 21. The part of the road under consideration is approximately 1 mile long and traverses the now drained Smith Lake in the extreme northeast part of Lambton County. At present, the route consists of a narrow dirt road used mainly by farm vehicles.

This investigation was intended to augment a previous exploration carried out by the Department of Highways, consisting of a number of shallow auger holes spaced at close centres along the route. The purpose of the present work was to explore in greater depth the subsurface conditions at four selected points, to determine the relevant properties of the subsoil and to make recommendations concerning the construction of the proposed new road.

II SITE AND GEOLOGY

The line of the proposed road traverses Smith Lake which was drained in relatively recent times to provide rich agricultural land for corn, celery, onions and other special crops.

Chapman & Putnam¹ refer to the many lagoons which were formed behind the sand dunes of the Huron Fringe by the glacial Lake Algonquin, or by the present Lake Huron. Smith Lake is near the centre of one of the larger such features.

III FIELD AND LABORATORY PROCEDURES

The field work was done on the 10th, 11th and 12th of September, 1964 and consisted of four boreholes at locations specified by the Department of Highways along the proposed road centre-line. The holes were advanced by washboring using a skid-mounted diamond drill, and were lined with Bx (3-inch) casing.

Dynamic cone penetration tests were performed adjacent to each borehole. In the very soft soil conditions which prevail immediately below the surface, it was generally found that the cone would drop under the weight of the hammer and drill rods alone for some depth. It was subsequently driven until appreciable resistance was encountered. The purpose of this test was mainly to establish the boundary between the highly compressible upper deposits and the denser deposits below.

¹ numbers refer to references.

Standard penetration tests were performed using a 2-inch O.D. split spoon in locations where the soil was sufficiently firm to yield meaningful results. In many cases the spoon was merely pushed into the soil for the purpose of recovering a representative sample.

Insitu vane shear tests were performed using a 2-inch diameter 4-bladed vane. The vane was turned by means of a torque wrench having a sensitivity of approximately ± 30 pounds per square foot. In many cases no reading was obtained and in such cases the shear strength is recorded as "less than 30 p.s.f.".

A laboratory testing programme was carried out to determine the index properties, unconfined compressive strength and compressibility characteristics of the soft upper strata.

The results of the field tests and most laboratory results are recorded on geotechnical data sheets comprising enclosures 3 to 6. Elevations have been referred to the level of the top of a red and white survey peg located at the base of a mound of rubble near the west extremity of the road. This level is assumed to be El. 100.0. Consolidation test results are shown on enclosures 7 and 8 and a summary of all laboratory results is given on enclosure 9.

IV SUBSURFACE CONDITIONS

Details of the stratification at each borehole are shown on the data sheets and a summary of field test results is given in the form of a subsurface profile on enclosure 2.

Each hole encountered some depth of black or brown organic topsoil varying from 5 feet to less than 1 foot in thickness. This material has a very soft, sometimes fibrous texture and consists mainly of organic matter.

Below the topsoil there is a very soft greenish grey clay deposit of varying thickness. In boreholes 1 and 3 it extends to 8.5 and 11.8 feet respectively and in boreholes 2 and 4 to 22.8 and 28.0 feet respectively. Traces of brown fibrous organics and small white calcareous shells were frequently found within this stratum, especially in the upper 10 to 12 feet. Laboratory tests showed the following range of properties: moisture content 66% to 195%; plasticity index 31% to 87%; liquidity index 0.8 to 1.6; unit weight 75 to 99 p.c.f.; undrained shear strength 83 to 198 p.s.f.; void ratios of 1.76 and 3.3 and compression indices of 0.9 and 0.4.

At all levels within the clay stratum, the 'N'-value is less than 1. The undrained shear strength values measured in the field and in the laboratory are plotted on enclosure 10, and show a general trend of increasing strength with depth. The results vary from a maximum of 300 pounds per square foot to less than 30 p.s.f. The c/p characteristic which has been drawn through these points is in good agreement with the Skempton equation for normally consolidated clays, viz. $c/p = 0.11 + 0.0037 (PI)^*$. It is noted that the laboratory shear strength results are generally higher than the field vane results, suggesting that the latter may not have recorded the full shear strength of the material. In this connection it would be useful to make further borings of a larger diameter and using a larger vane. In this way a more accurate shear strength profile might be determined.

Each borehole was terminated in a compact to very dense stratum of fine silty sand which was explored to a maximum depth of 35.5 feet below the ground surface.

V GROUNDWATER CONDITIONS

The natural water table was encountered at depths varying from ground surface to 3.0 feet. It is understood that this level is constantly controlled by pumping and, for the purpose of the following analysis it will be assumed that the water table coincides with the ground surface.

VI DISCUSSION

It has been shown that the soil, for a considerable depth below the surface, is a very soft and highly compressible organic clay. The ability of such a material to carry a road foundation has been studied, and it is concluded that a considerable amount of sinkage would occur. For example a 5-foot depth of granular fill placed on the surface would impose a pressure of 500 to 600 pounds per square foot. The shear strength of the underlying soil is 50 pounds per square foot or less so that clearly the soil would be displaced, and a "mud wave" would form on either side.

A proposed method of estimating the sinkage based on past experience² is to assume that the sinkage will stop when the ultimate bearing capacity of the soil is equal to the applied weight of fill. This can be expressed by the equation proposed by Sinacori² et al:-

*Where c is the undrained shear strength, p is the overburden pressure and PI is the plasticity index.

$$\text{Safety factor} = \frac{5.7c + \gamma_o D + \gamma_o M}{\gamma_f H + \gamma_f' D} \dots\dots\dots(1)$$

where c = undrained shear strength of soil

γ_o = unit weight of soil

γ_o' = unit weight of soil (submerged)

γ_f = unit weight of fill

γ_f' = unit weight of fill (submerged)

D = depth of fill below ground surface

H = depth of fill above ground surface

M = height of "mud-wave" above ground surface

and it is assumed that the water table coincides with the ground surface.

Experience reported at the Rainy Lake Causeway³ indicated that the actual sinkage was somewhat greater than that described above.

In the conditions prevailing at boreholes 1 and 3 where the soft clay is about 10 feet deep, the fill would sink to the level of the sand. For the conditions at borehole 4, equation (1) suggests that stability might be reached at a depth of about 18 feet.

For example:

if c = 150 p.s.f.

γ_o = 87.5 p.c.f.

D = 18 feet

γ_o' = 25.0 p.c.f.

H = 5 feet

γ_f = 120.0 p.c.f.

M = 3 feet

γ_f' = 57.5 p.c.f.

then

$$\text{Safety factor} = \frac{5.7 \times 150 + 25 \times 18 + 87.5 \times 3}{120 \times 5 + 57.5 \times 18} \dots\dots (2)$$

$$= \frac{1566}{1633} = 0.96$$

The safety factor can be varied above or below unity by slight adjustments to the assumed soil parameters.

On the basis of the above criteria it appears that over most of the length of the road where the depth of soft clay is 15 feet or less, the clay will be displaced entirely if the upper elevation of the fill is maintained. In the deeper areas such as at boreholes 2 and 4, there is a possibility that sinkage may stop at about 18 feet, leaving 5 to 10 feet of soft clay below. If the fill were allowed to remain at this level it is estimated that about 12 inches of consolidation settlement would occur and that the time periods for 50% and 90% consolidation would be 250 and 1100 days respectively.

However, considering the relatively small proportion of material which might remain undisplaced, it would seem logical to encourage the embankment to settle to the bottom of the soft stratum, especially if this can be done without much additional effort and thereby obviate the problems of long-term differential settlement. On the basis of the previous observation reported from the Rainy Lake project the embankment might continue to settle in any case. Referring to the numerical values in equation (2), removal of the mud wave would reduce the safety factor to 0.8, and the addition of an extra 5 feet of surcharge @ 120 p.c.f. would reduce it further to 0.58.

From the foregoing analysis it appears that it would not be difficult to form the entire length of road by displacing the soft clay. If this method is adopted the fill should be placed by continuous end-dumping. Probing should be carried out to ensure that soft material does not become trapped below the fill, and in such an event if the problem cannot be over come by removing the mud wave or applying surcharge, blasting can be used as a final resort.

Some thought has been given to partial or complete excavation, but the practical problems involved appear far greater than in the method just described. The physical act of digging the material out would present difficulties such as the stability of the digging machines and the trucks required to carry the material away. The side slopes would need to be very flat to be stable, the amount of land used would be correspondingly large, as would be the amount of fill required, and disposal of the excavated soil might present further problems.

This report has been prepared by Mr. J. Park, P. Eng., who also supervised the field work, and has been reviewed by Mr. K. H. King, P. Eng.

VII REFERENCES

1. The Physiography of Southern Ontario by L. J. Chapman and D. F. Putnam of the Ontario Research Foundation - University of Toronto Press 1951.
2. M. N. Sinacori, W. P. Hofmann and A. H. Emery, "Treatment of Soft Foundations for Highway Embankments", Highway Research Board, New York State, Department of Public Works, Proceedings, Volume 31, 1952.
3. Matich, Rutka and Anderson, "Foundation Aspects of the Rainy Lake Causeway", The Engineering Journal, November 1963.



DOMINION SOIL INVESTIGATION LIMITED

James Park

JP/mkf

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

ENCLOSURES

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
$\phi > 8"$	3"	COARSE	FINE	COARSE	MEDIUM	FINE	0.074	0.002	>	NO SIZE LIMIT		
U.S. Standard Sieve Size		No. 4		No. 10		No. 40	No. 200					

SAMPLE TYPES.

AS Auger sample
CS Sample from casing
ChS Chunk sample

RC Rock core
% Recovery
SS Split spoon sample

TP Piston, thin walled tube sample
TW Open, thin walled tube 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.

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

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

SYMBOL :

322

SOIL PROPERTIES.

W % Water content
LL % Liquid limit
PL % Plastic limit
PI % Plasticity index
LI Liquidity index

γ_s Natural bulk density (unit weight)
e Void ratio
RD Relative density
C_v Coeff. of consolidation
m_v Coeff. of volume compressibility

k Coeff. of permeability
C Shear strength in terms of total stress
 ϕ Angle of int. friction
C' Cohesion in terms of effective stress
 ϕ' Angle of int. friction

UNDRAINED SHEAR STRENGTH.

- DERIVED FROM -

TRIAXIAL COMPRESSION TEST

UNCONFINED TEST

LABORATORY

VANE TEST

FIELD

POCKET PENETROMETER TEST - St

Strain at failure is represented by direction of stem

20%
15% + 5%
10%

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

SOIL DESCRIPTION.

COHESIONLESS SOILS :

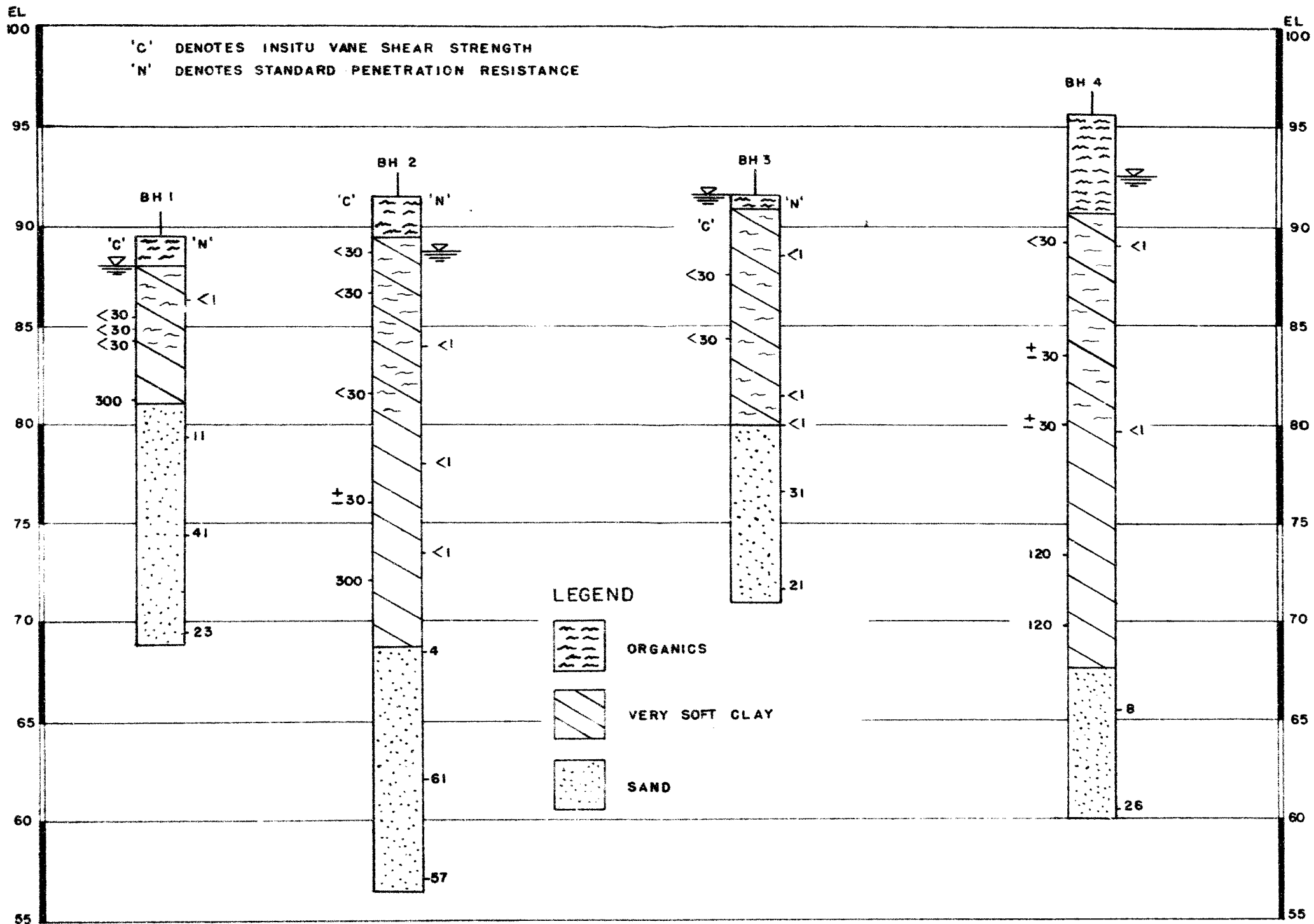
RD :

COHESIVE SOILS :

C lbs/sq ft

Very loose 0 - 15 %
Loose 15 - 35 %
Compact 35 - 65 %
Dense 65 - 85 %
Very dense 85 - 100 %

Very soft less than 250
Soft 250 - 500
Firm 500 - 1000
Stiff 1000 - 2000
Very stiff 2000 - 4000
Hard over 4000



SUBSURFACE PROFILE

HORIZONTAL SCALE: 1 INCH TO 400 FEET
VERTICAL SCALE: 1 INCH TO 5 FEET

GEOTECHNICAL DATA SHEET FOR BOREHOLE 1.

OUR REFERENCE NO 4-9-12

CLIENT Department of Highways Ontario
PROJECT Development Road No. 733
LOCATION Smith Lake
DATUM ELEVATION 100.0 feet (see text) Borehole chainage 237 + 00 11th, 1964

METHOD OF BORING Wash boring
DIAMETER OF BOREHOLE 8x (3-inch)
DATE September 10th, 1964 and 11th, 1964

ENCLOSURE NO 3

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE					CONSISTENCY					REMARKS
				NUMBER	TYPE	N ₆₀ or Advance- ment of Sampler	blows per foot					water content %					
							SHEAR STRENGTH 10 lbs/sq ft										
							10	20	30	40	50	50	100	150	200		
89.5	0.0	Ground surface															
	1.5	Black organic muck, very soft		1	CS											WL El. 88.0 September 11/64	
		Greenish grey clay, very soft		2	SS	1											
85		Some fibrous materials and shells														*Vane shear strength less than 30 p.s.f.	
	6.8			3	TW	p											
	8.5															2" Ø cone penetrated to 8'-6" under weight of 140 lb. hammer and drill rods.	
80		Very fine brown silty sand, compact to dense		4	SS	11											
				5	SS	41											
75																V denotes insitu vane shear test	
70				6	SS	23											
	20.5	End of borehole															

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: MKF CHD: JP

OUR REFERENCE NO. 4-9-12

GEOTECHNICAL DATA SHEET FOR BOREHOLE . 2. . . .

CLIENT: Department of Highways Ontario
 PROJECT: Development Road No. 733
 LOCATION: Smith Lake
 DATUM ELEVATION: 100.0 feet (see text)

METHOD OF BORING: Wash boring
 DIAMETER OF BOREHOLE: 8x (3-inch)
 DATE: September 11th, 1964
 Borehole chainage 246 + 50

ENCLOSURE NO. 4

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot		CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	4 or Advancement of Sampler	20	40	60	80	
91.4	0.0	Ground surface									
90	2.0	Black organic muck, very soft		1	CS						
				2	CS						
					V						
85		some fibrous material and shells		3	TW	p					
				4	SS	p					
					V						
80	11.0	Greenish grey clay, very soft		5	TW	p					
				6	SS	p					
					V						
75				7	SS	p					
					V						
70	20.0	shells									
				8	SS	4					
	22.0	Very fine brown silty sand, compact to dense									
65				9	SS	61					
60											
				10	SS	57					
55.0		End of borehole									

2" dia. cone

WL El. 88.7 September 11/64

*Vane shear strength less than 30 p.s.f.

$\delta = 85.0$

$\delta = 98.0$

Occasional seams of coarse sand, traces of clay and wood fragments below 50 feet.

2" ϕ cone penetrated to 16' under weight of 140 lb. hammer and drill rods

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE: MKF

CH'D: JP

GEOTECHNICAL DATA SHEET FOR BOREHOLE 3

OUR REFERENCE NO. 4-9-L2

CLIENT: Department of Highways Ontario
 PROJECT: Development Road No. 533
 LOCATION: Smith lake
 DATUM ELEVATION: 100.0 feet (see text)

METHOD OF BORING: Wash boring
 DIAMETER OF BOREHOLE Bx (3-inch)
 DATE: September 12th, 1964
 Borehole chainage 261 + 00

ENCLOSURE NO. 5

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE		CONSISTENCY		REMARKS
				NUMBER	TYPE	Advancement of Sample	blows per foot	lb./sq. ft.	PI	W	
91.4	0.0	Ground surface									WL
90.8	0.6	Black organic muck, very soft		1	CS						E1. 91.0 September 12/64
89.0	1.4	Greenish grey some clay, black very organics		2	SS	p					*Vane shear strength less than 30 p.s.f.
85.0	5.0	soft, some fibrous material and shells		3	TW	p					
80.0	11.8	Very fine brown silty sand, compact to dense		4	SS	p					2" Ø cone penetrated to 11' under weight of 140 lb. hammer and drill rods
75.0				5	SS	p					
70.0	20.5	End of borehole		6	SS	31					
				7	WS						
				8	SS	21					

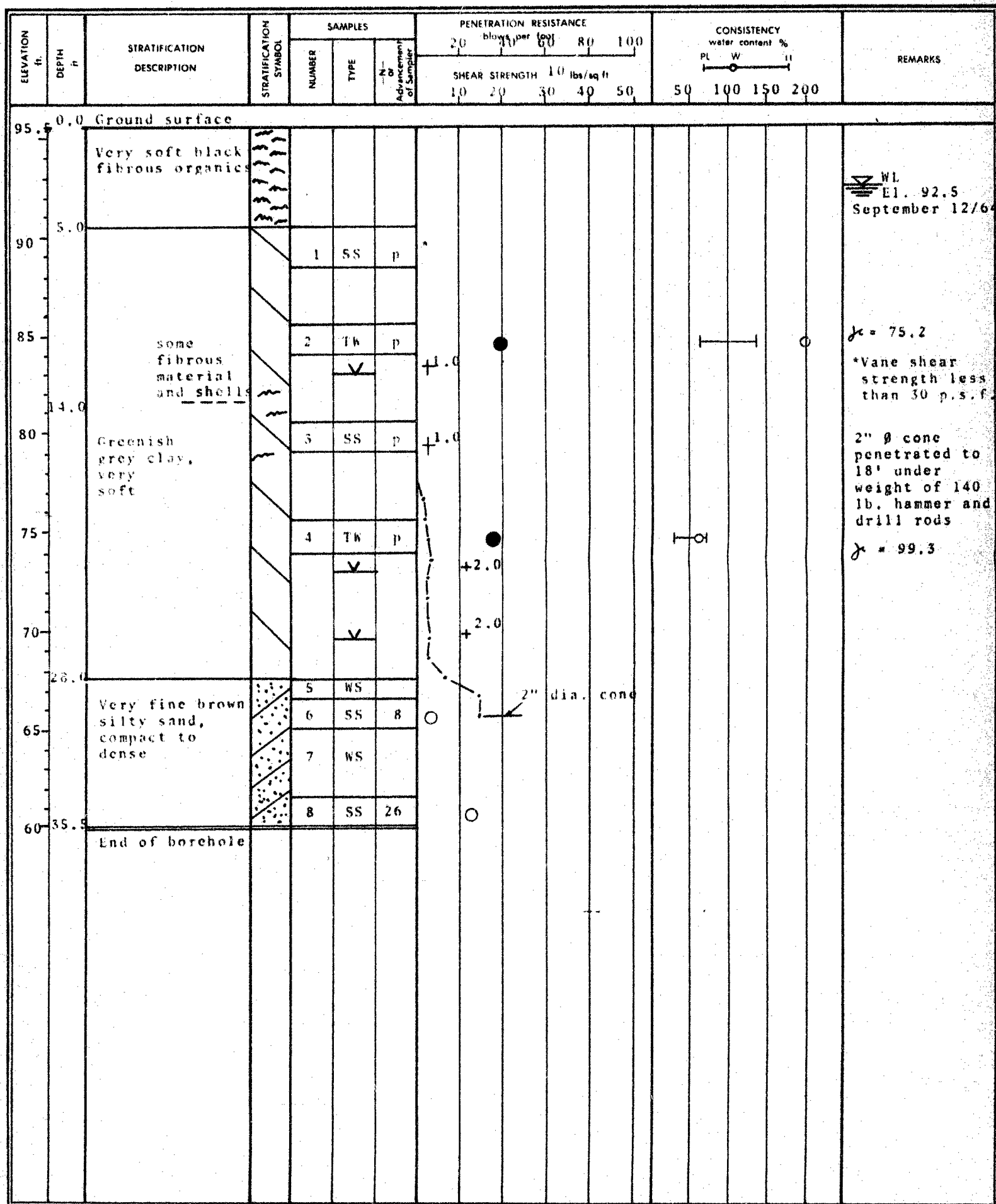
OUR REFERENCE NO. 4-9-L2

GEOTECHNICAL DATA SHEET FOR BOREHOLE 4....

CLIENT: Department of Highways Ontario
PROJECT: Development Road No. 733
LOCATION: Smith Lake
DATUM ELEVATION: 100.0 feet (see text)

METHOD OF BORING: Wash boring
DIAMETER OF BOREHOLE: Bx (3-inch)
DATE: September 12th, 1964
Borehole chainage 274 + 50

ENCLOSURE NO. 6



VERTICAL SCALE: 1 IN. TO 5 FT.

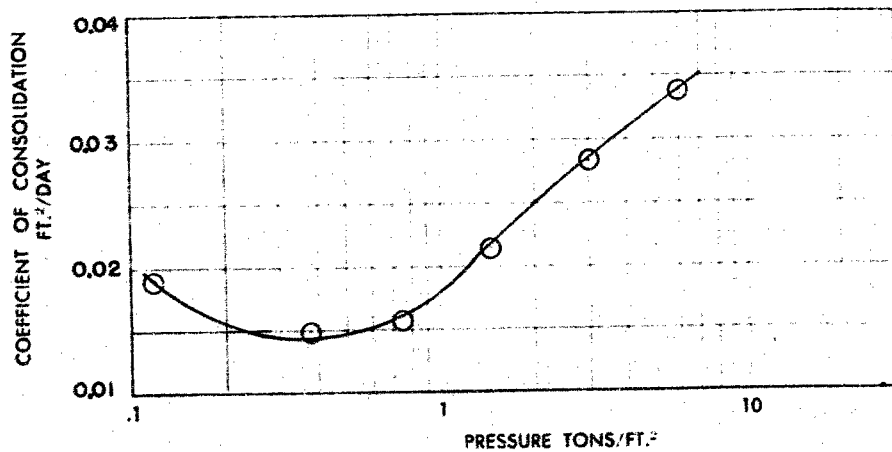
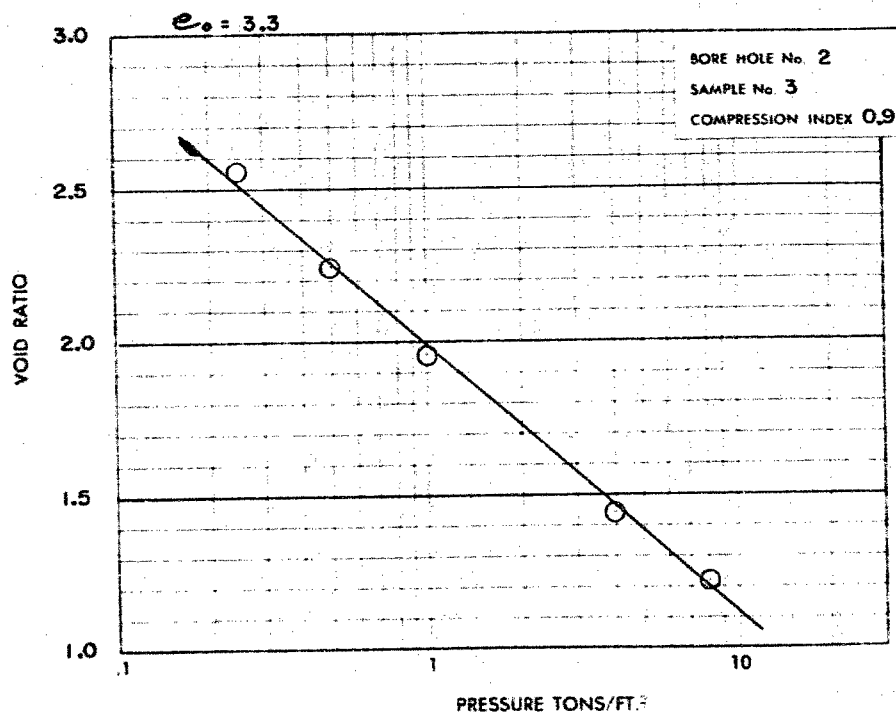
DOMINION SOIL INVESTIGATION LIMITED

MADE: MKF

CHD: JP

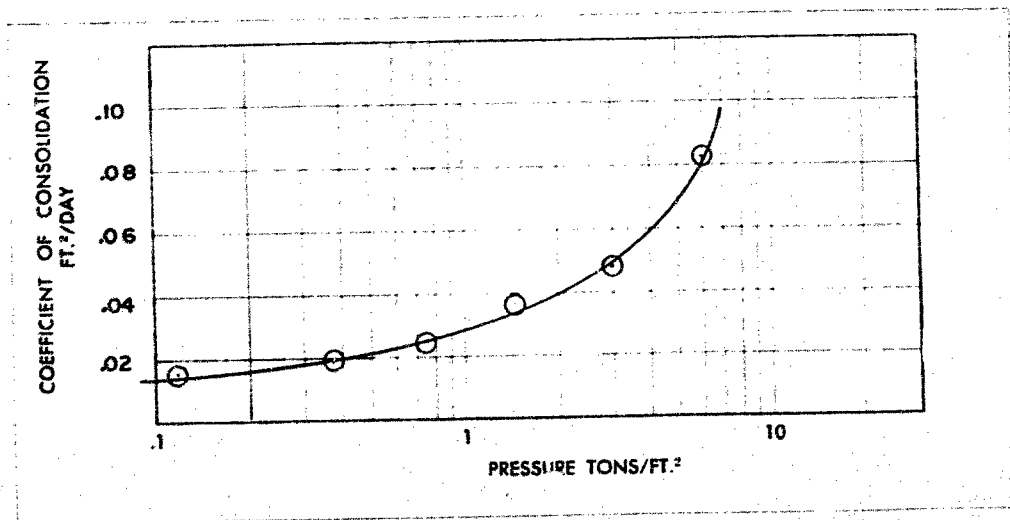
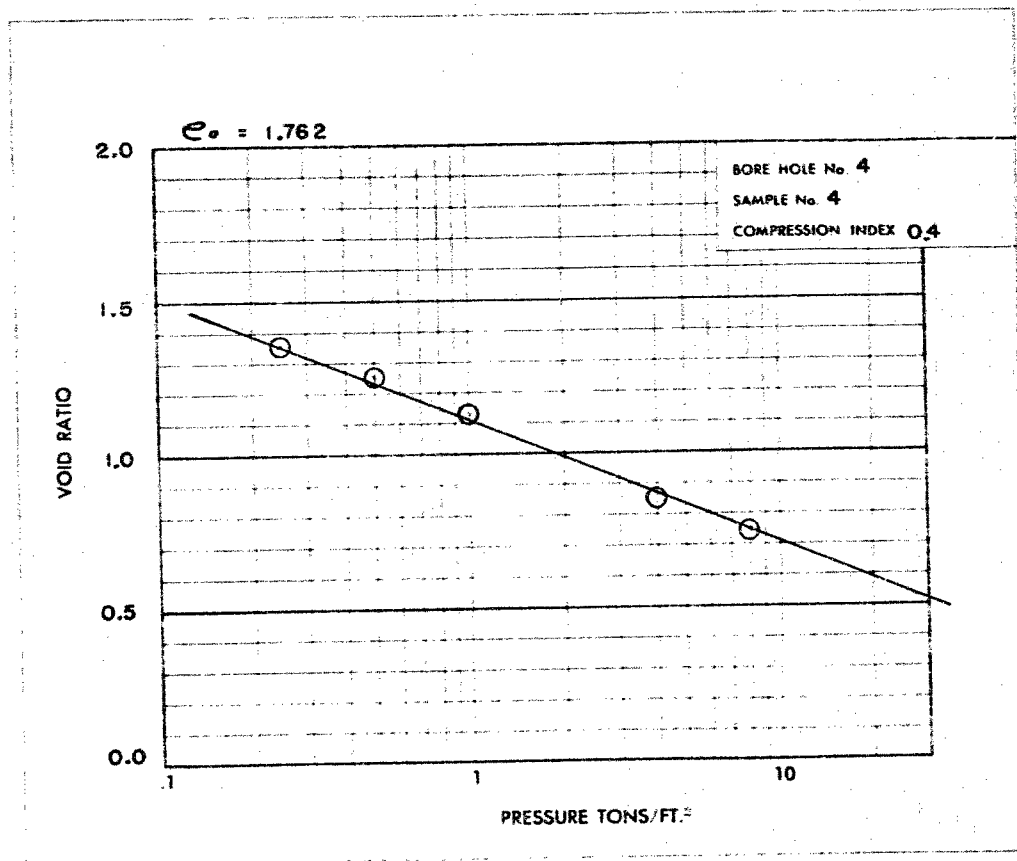
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CONSOLIDATION TEST



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CONSOLIDATION TEST



SUMMARY OF LABORATORY TEST DATA

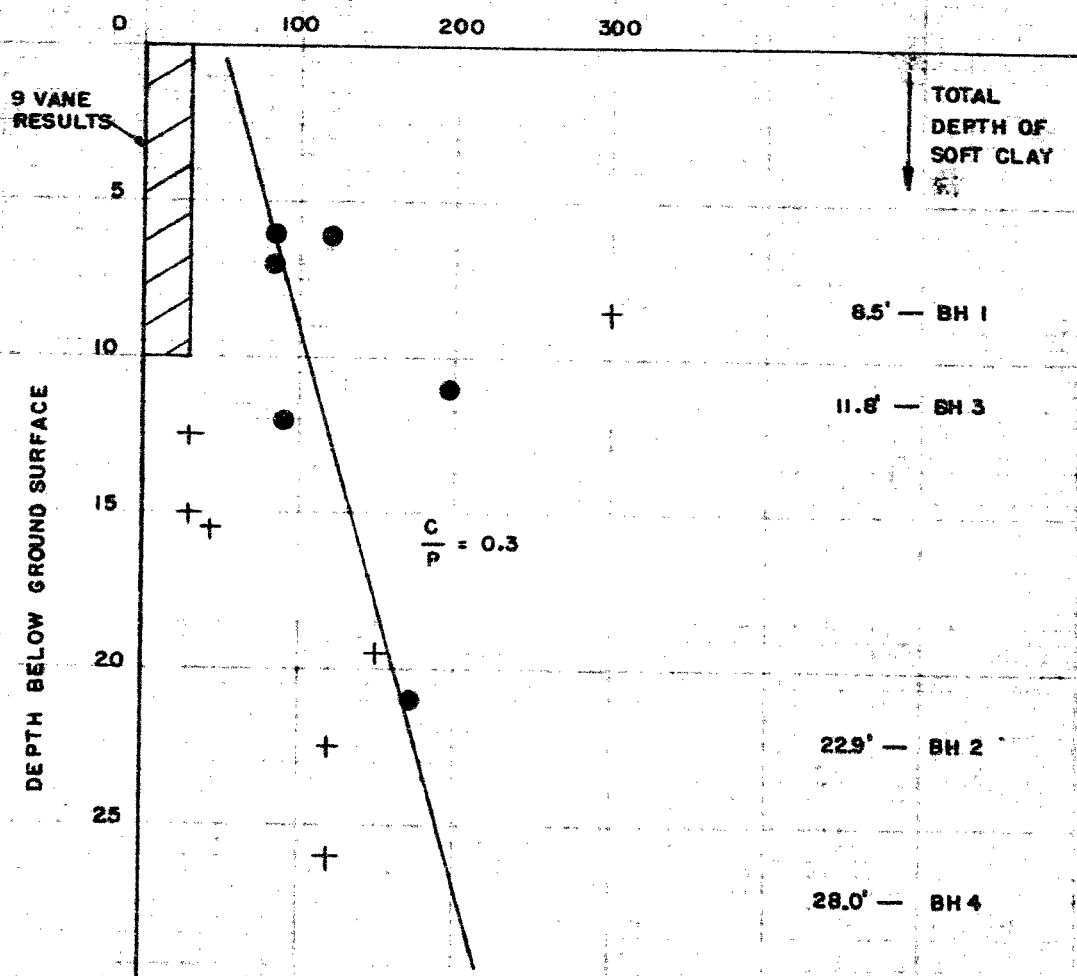
Borehole	1	2	2	3	4	4
Sample	3	3	5	3	2	4
Depth (feet)	7	6	12	6	11	21
Natural water content (%)	66.7	133	67	153	195	62.2
Liquid limit (%)	63.4	126	57.5	116	148	71.1
Plastic limit (%)	27.8	40.4	26.7	50.5	61.4	28.9
Plasticity index (%)	35.6	85.5	30.8	65.3	86.6	42.2
Liquidity index	1.1	1.1	1.3	1.6	1.5	0.8
Shear strength (p.s.f.)*	83	85	89	121	198	175
Unit weight (p.c.f.)	98	79	98	82	75	99
Void ratio	-	3.3	-	-	-	1.76
Compression index	-	0.9	-	-	-	0.4
Group symbol	CH	CH	CH	OH or MH	OH or MH	CH

*Taken as 1/2 unconfined compressive strength.

JOB NO 4-9-L2

ENCLOSURE NO 10

UNDRAINED SHEAR STRENGTH (PSE)



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Mr. Frank Gormic,
Bridge Engineer,
Bridge Design Section.

Materials and Testing

June 22nd, 1967

Re: W.F.203-66 District 17. Aux Sables River Structure
Lands and Forests Department, Massey, Ontario

The bridge site for the above structure is underlain by quartzite rock. This is taken from the geological map of the area. Examination of a polished section of the specimen submitted by you from the site shows under microscpic viewing the rock type classed as a quartz gneissoid metasediment. This rock is fractured with small cracks filled with calcite. The texture is fine grained. Mineral components are quartz, biotite mica and trace amounts of feldspar and garnet. It is dense and homogenous in structure and of a medium grey colour. This rock appears to be extremely hard and well composed in texture and structure. Bearing and crushing strengths should be quite high.

In this immediate area there are at least two major faults and other secondary breaks. An examination of the rock structure at this site will be made during the next month.

B. A. Glassford

B. A. Glassford,
Geologist.

MBB/jz

cc: W. R. Bennett
E. R. Saint
Z. Katona
M. Devata
B. A. Glassford

Mr. Frank Cormie,
Bridge Engineer,
Bridge Design Section.

B. K. Glassford

July 20th, 1967

S.P. 203-66 District #17, Aux Sables River Structure
Lands and Forests Dept., Massey, Ontario

I examined the rock at the site of the proposed bridge structure on July 7th. As stated in the memo to you on June 22nd, the descriptions and comments remain the same.

Two photographs are attached showing the bridge site.

There is one feature that should be brought to your attention. This rock being of a gneissic type has a banded or lineated structure, which in this specific area dips approximately 60° west. In blasting procedures for foundation footings for the piers this rock structure would tend to carry the blast along lineations and hence shatter and weaken the adjacent rock to the footings area. This would leave it open to water, ice and frost weathering actions. A certain amount of detailed care and attention by the contractor in his blasting method should be brought to his attention.

B. K. Glassford,
Geologist.

BKG/jm

cc: W. R. Bennett
E. M. Saint
Z. Katona
A. Devata
B. K. Glassford