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W. O. No. \_\_\_\_\_

STR. SITE No. 12-44

HWY. No. \_\_\_\_\_

LOCATION CONC. RD.  $\frac{1}{2}$  LUCKNOW RIVER,  
BRIDGE 367 ASHFIELD TWP, LOT 11, CON. ~~300~~ 4<sup>25</sup>

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: DOCUMENTS TO BE UNFOLDED BEFORE  
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# DOMINION SOIL INVESTIGATION LIMITED

CONSULTING SOIL & FOUNDATION ENGINEERS

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B.M. ROSS & ASSOCIATES LIMITED  
CONSULTING ENGINEERS  
62 NORTH STREET  
GODERICH ONTARIO

Report On  
SOIL INVESTIGATION  
for  
BRIDGE BR-367  
LOT 11, CONCESSIONS IV-V  
TOWNSHIP OF ASHFIELD

40P13-12

GEOCRE No.

*MTC Site 12-44*

by

Dominion Soil Investigation Limited  
164 Newbold Court  
London Ontario

Ref: 75-11-L1  
December 1975

STRUCTURE SITE No. 12 44

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I

INTRODUCTION

In accordance with a letter of authorization from B.M. Ross and Associates Limited, Consulting Engineers, a soil investigation has been carried out in the Township of Ashfield, where it is proposed to replace an existing bridge with a new structure. The existing structure is located at Lot 11, Concessions 4/5 of the Eastern Division of the Township, where the Concession road crosses the Lucknow River.

It is understood that the proposed structure is a 3-span spillway type beam bridge with the abutments placed on newly constructed fill and supported by piling. The approximate span ratio will be 50-60-50 (feet), and the new structure will be centred around the existing bridge. The requirements of the project were discussed with Mr. K.G. Dunn, P.Eng., who supplied the foregoing information.

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

## II

FIELD WORK

The field work, consisting of four boreholes, was carried out on November 4 and 27, 1975, at the locations shown on Enclosure 2. The holes were advanced to the sampling depths by a continuous flight power auger machine which was equipped for soil sampling.

Standard penetration tests were performed at frequent intervals of depth, as detailed in Appendix 'A', and the results are recorded on the borehole logs as 'N' values. The split-spoon samples were stored in air-tight containers which were transferred to our London laboratory for classification, testing and storage.

The field work was supervised by a soils technician, who also related the ground surface elevations to a local datum. The benchmark was taken as the centre of the existing bridge deck, and it was established by the client as having a Geodetic value, El. 782.44 feet.

## III

SUBSURFACE CONDITIONS

Detailed descriptions of the strata, which were encountered in each borehole, are given on the borehole logs comprising Enclosures 3, 4 and 5, and a general picture of the soil stratigraphy is presented in the form of a Subsurface Profile on Enclosure 2. The following notes are intended only to amplify this data.

Boreholes 1 and 2 were put down behind the existing abutments and revealed considerable depths of fill material, which is associated with the construction of the approaches to the existing bridge. The native subsoil consists of glacial sandy and clayey silt till, which is overlain by a layer of sand and gravel at borehole 2, 3 and 4 locations. The relative density of the sandy silt till is described as 'very dense' based on 'N' values of 50 blows for a penetration of 1 to 6 inches of the sampler, and the consistency of the clayey silt till is described as 'hard' based on 'N' values of 50 blows for a 5 to 6 inch penetration of the sampler.

Five attempts were made to put down the borehole at borehole 3 location and each time penetration of the augers further than 4 feet was not possible due to the presence of boulders. It may therefore be assumed that the native till materials contain a significant quantity of random boulders.

Grading analyses of representative samples of the silt till material are shown as grain size distribution curves on Enclosure 6. The moisture content of the sandy and clayey silt till materials was determined to range from 5.5% to 10.4%.

## IV

GROUNDWATER CONDITIONS

The water level in the existing river was observed at El. 763.4, and the following water levels were observed in the boreholes after completion of the borings:

<u>Borehole</u>	<u>Groundwater Elevation (feet)</u>
1	770.5
2	765.3 (Cave-in)
3	764.1 (Ground Surface)
4	763.5

From these observations it may be assumed that the prevailing water table is closely related to the river level at any particular time.

V DISCUSSION AND RECOMMENDATIONS

The investigation has shown that the native subsoil below the river bed level consists of 'very dense' sandy silt or 'hard' clayey silt till materials which contain larger gravel, cobble and boulder sizes.

Pier Footings

The 'very dense' till will provide adequate support for the construction of spread footing foundations, and the footing grade should be established at least 4 feet below the lowest river bed level to provide sufficient protection against heave due to frost action. A maximum allowable soil pressure of 5 tons per square foot may be used for the footing design, and this soil pressure incorporates a factor of safety of at least 3 against shear failure of the underlying soil.

Total settlement of footings mobilizing the maximum soil pressure is estimated to be 0.5 inch or less, and no appreciable differential settlement is anticipated.



Due to the 'very dense' nature of the subsoil it may be assumed that dewatering of the footing excavations will be carried out by normal pumping procedures once the river flow has been diverted from the footing area. Particular care should be taken to prevent softening or freezing of the subgrade once it has been exposed, and it is advisable to pour the footings as soon as possible after completion of the excavation to minimize disturbance to the subgrade. Alternatively a mud-slab of lean concrete can be poured on the footing grade to provide protection for the subgrade.

#### Abutment Foundations

It is recommended that the approach fills be constructed with approved granular fill, compacted to 95% of maximum standard Proctor dry density to preclude settlement of the approach pavement. Steel H-piles should be used for the abutment supports, and it is anticipated that 12 BP53 piles will develop a working load of 60 tons if they are driven 10 to 15 feet into the 'very dense' silt till. The rating of the driving hammer should be at least 12,000 foot pounds per blow, and the piles should be driven to a refusal value of 5 blows for  $\frac{1}{4}$  inch penetration.

The dynamic driving characteristics of the piles should be monitored, and the estimated theoretical working loads should be checked by calculation using a recognized pile driving formula. If the desired working load is not achieved within the anticipated depth of penetration, the piles should be extended or additional piles driven to provide the required support for the structure.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED



CJWA:eg

A handwritten signature in cursive script, appearing to read 'C.J.W. Atkinson'.

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

## APPENDIX 'A'

### THE STANDARD PENETRATION TEST.

In order to determine the relative density of non-cohesive soils, such as sands and gravels, the standard penetration test has been adopted. The test also gives an indication of the consistency of cohesive soils.







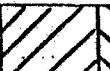

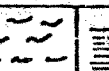

A two inch external diameter thick-walled sample tube is driven into the ground at the bottom of the borehole by means of a 140 lb. hammer falling freely through 30-ins. The tube is first driven an initial 6-inches to allow for the presence of disturbed material at the bottom of the borehole. The number of standard blows (N) required to drive the sampler a further 12-in. is recorded. The sample tube is one originally developed by Raymond Concrete Pile Company in the United States, where a sufficient number of tests have been made in conjunction with field investigations to show that the results, although essentially empirical, may be applied to foundation design.

For Sands:-

Values of 'N'	Density
Less than 10	Loose
Between 10 and 30	Compact
Between 30 and 50	Dense
Greater than 50	Very dense

# 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"	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					

## 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.

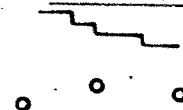
DYNAMIC PENETRATION RESISTANCE : to drive a 2"  $\phi$ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

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

LL % Liquid limit

PL % Plastic limit

PI % Plasticity index

LI Liquidity index



Natural bulk density (unit weight)



Void ratio

RD

Relative density

Cv

Coeff. of consolidation

mv

Coeff. of volume compressibility

k

Coeff. of permeability

C

Shear strength



Angle of int. friction

C'

Cohesion



Angle of int. friction

in terms of total stress

in terms of effective stress

## UNDRAINED SHEAR STRENGTH.

- DERIVED FROM -

TRIAXIAL

UNCONFINED

LABORATORY

FIELD

COMPRESSION TEST

TEST

VANE TEST

POCKET PENETROMETER TEST

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/sqft

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

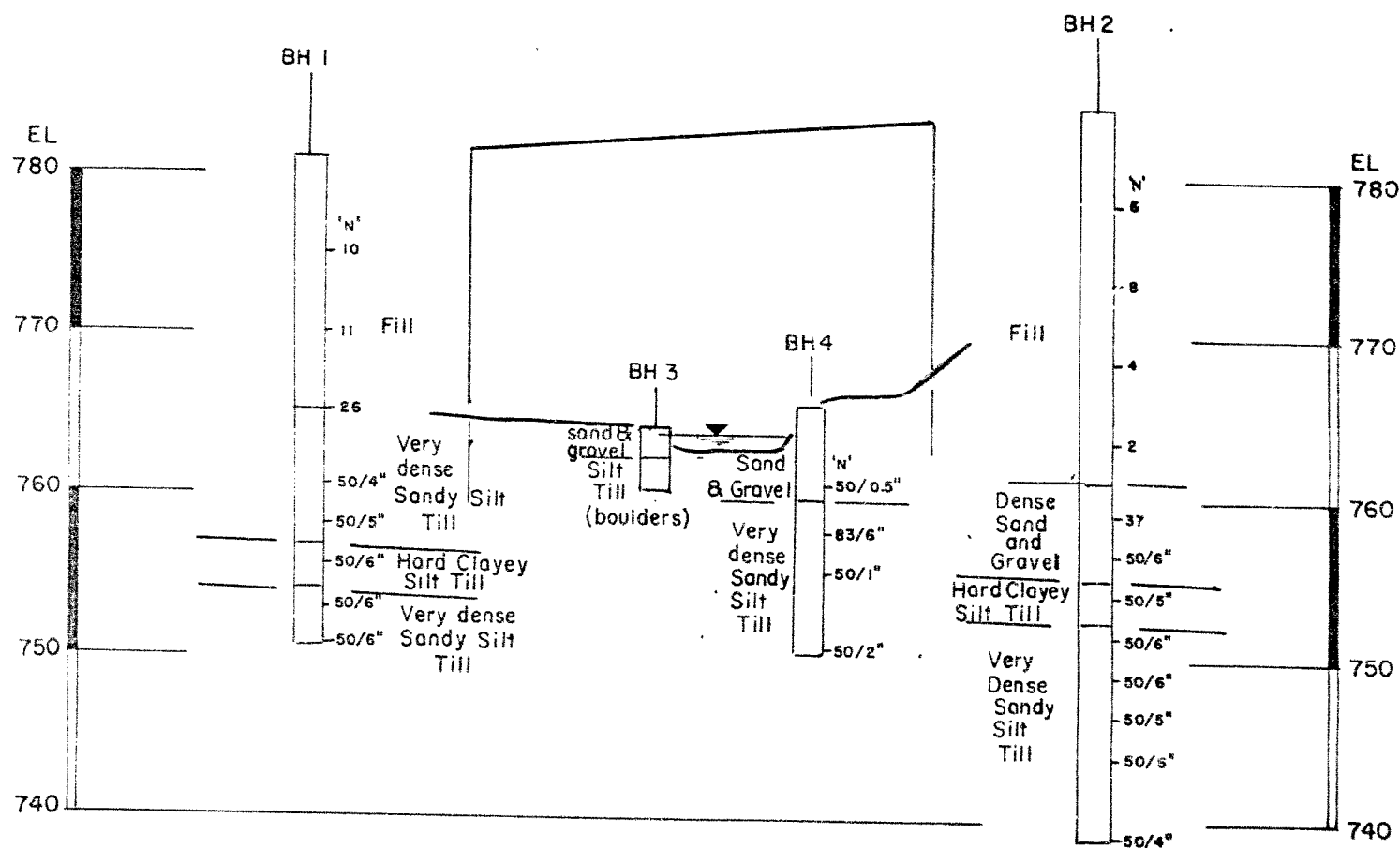
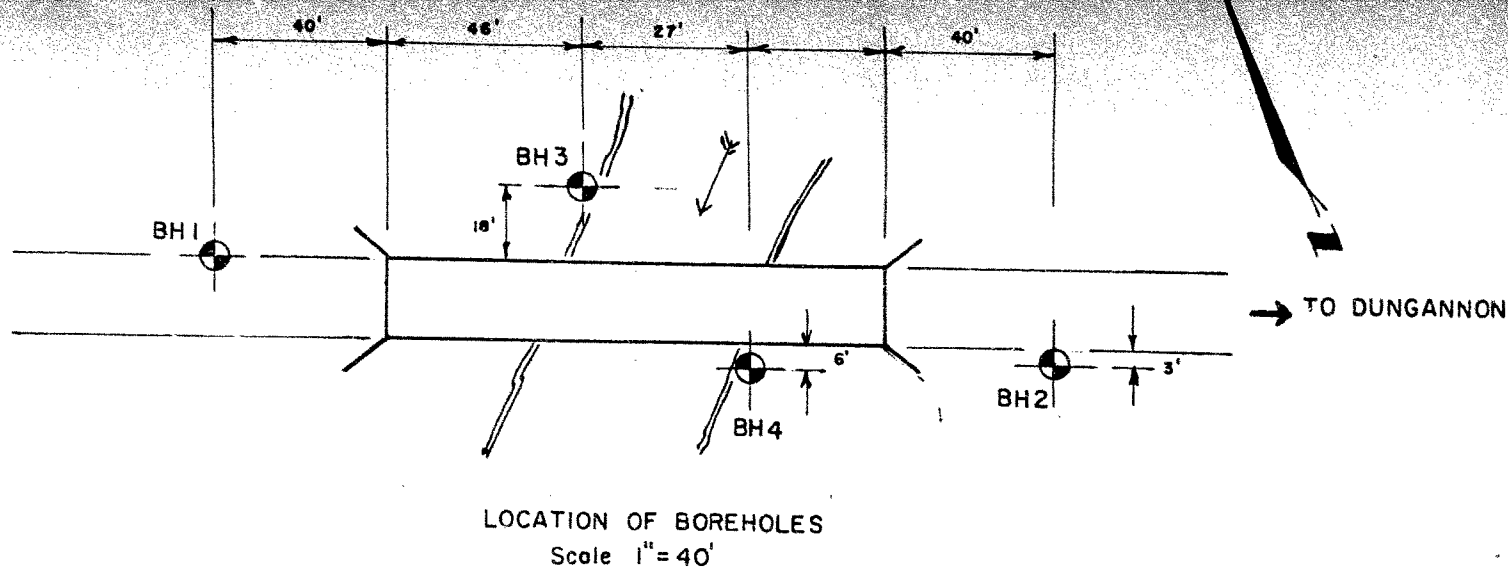
1000 - 2000

Very stiff

2000 - 4000

Hard

over 4000



SUBSURFACE PROFILE  
Hor. Scale 1" = 40'  
Vert. Scale 1" = 10'

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

Our Reference No. 75-11-L1

Enclosure No. 3

CLIENT: B.M. Ross and Associates Limited,  
PROJECT: Proposed Bridge,  
LOCATION: Township of Ashfield.  
DATUM ELEVATION: Top of deck, El. 782.44 feet

## DRILLING DATA

Method: Auger  
Diameter: hollow-stem  
Date: Nov. 4, 1975.

SUBSURFACE		PROFILE		SAMPLES			PENETRATION RESISTANCE					Blows/Ft.			WATER CONTENT %			REMARKS			
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Ft.	20	40	60	80	100	PLASTIC LIMIT	NATURAL	LIQUID LIMIT						
								UNDRAINED SHEAR STRENGTH	p.s.f.												
								+ FIELD VANE TEST	• COMPRESSION TEST												
781.0	0.0	Ground Surface																			
780		Sand and gravel, sandy silt and silty clay.																			
775					1	SS	10														
770		Fill.			2	SS	11														
765	16.0	Very dense sandy silt, some gravel.			3	SS	26														
760					4	SS	50/4"														
245		Hard grey gravelly sandy clayey silt.			5	SS	50/5"														
755		Very dense grey sandy silt some gravel.			6	SS	50/6"														
27.0					7	SS	50/6"														
750	30.5	End of Borehole			8	SS	50/6"														

Enclosure No. 4

[illegible]

## LOG OF BOREHOLE..... 3 &amp; 4.....

Our Reference No. 75-11-L1

Enclosure No. 5

CLIENT: B.M. Ross and Associates Ltd.,

PROJECT: Proposed Bridge, BR-367

LOCATION: Township of Ashfield

DATUM ELEVATION:centre of bridge deck, El. 782.44 feet Date: Nov. 27, 1975.

## DRILLING DATA

Method: Auger

Diameter: hollow-stem

Date: Nov. 27, 1975.

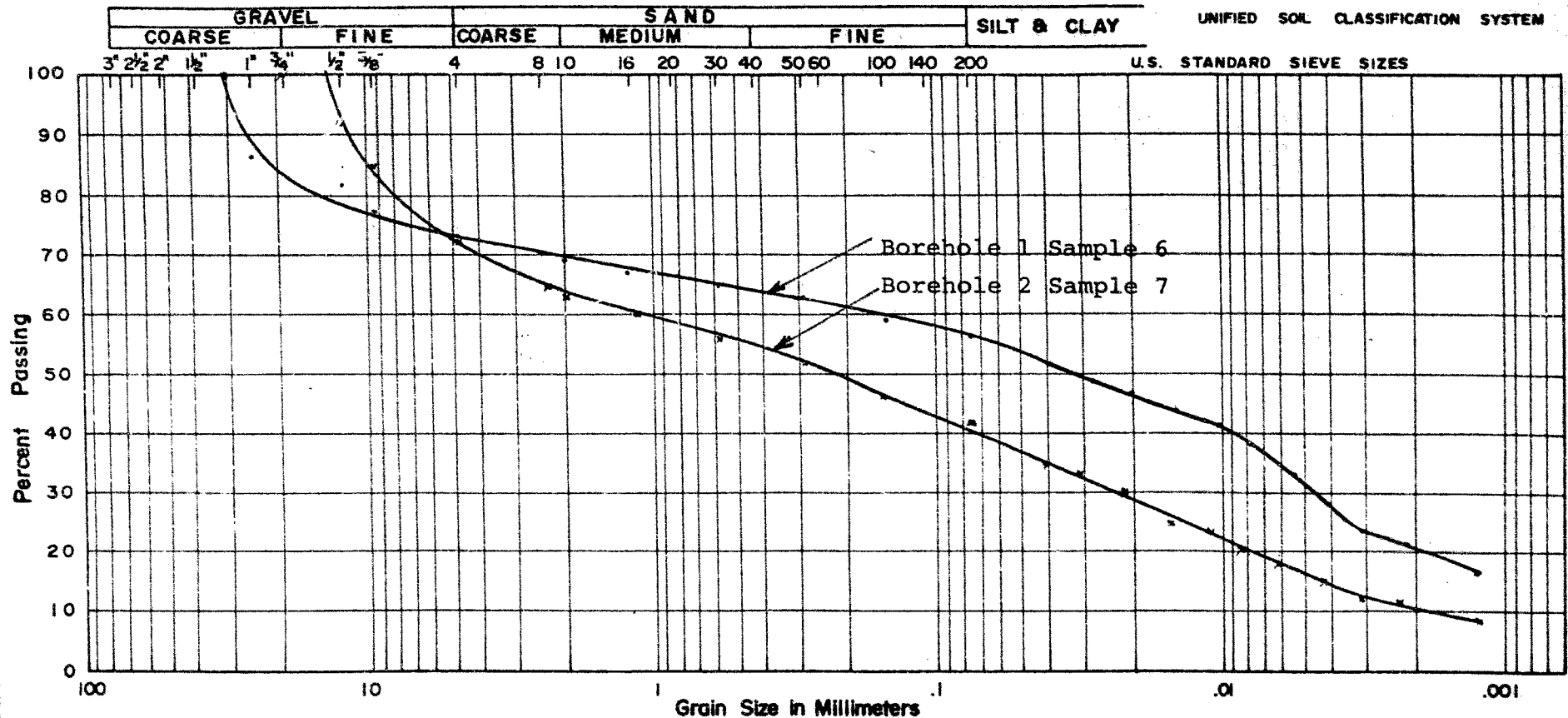
SUBSURFACE		PROFILE		SAMPLES			PENETRATION RESISTANCE					Blows/Ft.			WATER CONTENT %			REMARKS
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Ft.	20	40	60	80	100	PLASTIC LIMIT W <sub>p</sub>	NATURAL W	LIQUID LIMIT W <sub>L</sub>			
								UNDRAINED SHEAR STRENGTH p.s.f.										
								+ FIELD VANE TEST    • COMPRESSION TEST										
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# DOMINION SOIL INVESTIGATION LIMITED

## GRAIN SIZE DISTRIBUTION

75-11-L1  
OUR REFERENCE N<sup>o</sup> .....



PROJECT: Proposed Bridge,  
 LOCATION: Township of Ashfield.  
 BOREHOLE N<sup>o</sup>: 1 2  
 SAMPLE N<sup>o</sup>: 6 7  
 DEPTH: 26' 31'  
 ELEVATION: 755' 753'

COEFFICIENT OF UNIFORMITY :  
 COEFFICIENT OF CURVATURE :

**Classification of Sample and Group Symbol:**  
 Gravelly sandy clayey silt.

**PLASTIC PROPERTIES**  
 LIQUID LIMIT % =  
 PLASTIC LIMIT % =  
 PLASTICITY INDEX % =  
 MOISTURE CONTENT % =

DOCUMENT MICROFILMING IDENTIFICATION.

G.I.-30 SEPT. 1976

GEOCRES No. 40P14-15

DIST. 3 REGION SOUTHWESTERN

W.P. No. \_\_\_\_\_

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. 25-5

HWY. No. \_\_\_\_\_

LOCATION CONC. RD  $\frac{1}{2}$  LITTLE MAITLAND

RIVER, BRIDGE 293, WALLACE TWP, LOT 50

CON. 2  $\frac{1}{2}$  3

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: DOCUMENTS TO BE UNFOLDED

BEFORE MICROFILMED

\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

40P14-15
GEOCRES No.

B.M. ROSS & ASSOCIATES LIMITED  
CONSULTING ENGINEERS  
62 NORTH STREET  
GODERICH ONTARIO N7A 2T4

Report On  
SOIL INVESTIGATION  
for  
BRIDGE BR-293  
LOT 50, CONCESSIONS 2/3  
TOWNSHIP OF WALLACE

25-5

by

Dominion Soil Investigation Limited  
164 Newbold Court  
London Ontario N6E 1Z7

Ref: 76-1-L7  
March 8, 1976

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I

INTRODUCTION

In accordance with a letter of authorization from B.M. Ross & Associates Limited, Consulting Engineers, a soil investigation has been carried out in the Township of Wallace, where it is proposed to replace an existing bridge with a new structure. The existing structure is located at Lot 50, Concessions 2/3 of the Township, where the Concession road crosses the Little Maitland River.

The existing bridge is a 48 foot span steel truss and concrete deck structure, and it is understood that the new structure will be centred on the existing bridge. The requirements of the project were discussed with Mr. K.G. Dunn, P. Eng., who supplied the foregoing information.

The purpose of the investigation was to reveal the subsurface conditions at the 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, was carried out on February

6, 1976, at the locations shown on Enclosure 2. The holes were advanced to the sampling depths by a continuous flight power auger machine, which was equipped with hollow stem augers for soil sampling.

Standard penetration tests were performed at frequent intervals of depth, as detailed in Appendix 'A', and the results are recorded on the borehole logs as 'N' values. The split-spoon samples were stored in air-tight containers, which were transferred to our London laboratory for classification, testing and storage.

The field work was supervised by a soils technician, who also related the ground surface elevations to a local datum. The benchmark was taken as a nail in a hydro pole at station 12+69, and it was established by the client has having a Geodetic El. 1186.11 feet.

## III

SUBSURFACE CONDITIONS

Detailed descriptions of the strata, which were encountered in each borehole, are given on the borehole logs comprising Enclosures 3 and 4, and a general picture of the soil stratigraphy is presented in the form of a Subsurface Profile on Enclosure 2. The following notes are intended only to amplify this data.

Borehole 1 encountered a stratum of sandy silt at the ground surface, and this material extends to a depth of 14.5 feet. 'N' values of 28 and 61 blows per foot indicate that the sandy silt has a 'compact' to 'dense' relative density. At El. 1175 the borehole encountered a very dense sand and gravel stratum which extends down to El. 1167, and below the very dense sand and gravel the borehole encountered dense fine to medium sand which extends down to El. 1159. The borehole was terminated in a hard grey silty clay stratum at El. 1154.

Borehole 2 encountered a surface layer of sand and gravel fill which extends to a depth of 10.5 feet, overlying a thin layer of sandy silty clay containing wood fragments. The native subsoil consists of successive layers of silty sand, silty and fine sand and a laminated silty fine sand and silt structure which extends to a depth of 27.5 feet. These granular materials have a generally 'compact' relative density as indicated by 'N' values ranging from 15 to 39 blows per foot. The borehole encountered a 'very stiff' to 'hard' silty clay stratum at El. 1162, which extends down to El. 1155, and it is terminated in a very dense sandy silty till stratum containing cobbles and boulders at El. 1141.

Grading analyses were performed on three typical samples of the

granular materials from boreholes 1 and 2 and the results are shown as grain distribution curves on Enclosure 5.

IV

#### GROUNDWATER CONDITIONS

After completion of the field work, water levels were observed at El. 1180.4 and El. 1180.3 in boreholes 1 and 2 respectively, and it may be assumed that the groundwater table is influenced by the stream level at any particular time.

V

#### DISCUSSION AND RECOMMENDATIONS

The investigation has shown that the soil profile consists of compact to very dense granular materials to a depth of at least 16 feet below the stream bed. The stream bed extends down to El. 1178, therefore allowing 4 feet of cover for frost protection it is recommended that spread footing foundations be located at or below El. 1174. The finer granular materials encountered at borehole 2 location are readily susceptible to scour and deeper penetration of the footings or positive scour protection should be studied.

On the basis of the borehole results a maximum allowable soil pressure



of 4000 p.s.f. is appropriate for the design of footings at or below El. 1174, and this soil pressure incorporates a factor of safety of at least 3 against shear failure of the underlying soil.

Total settlement of footings will not exceed 1-inch, and the greater part of the settlement will occur immediately as the load is applied.

The coefficient of friction between the footings and the underlying subsoil may be taken as 0.45, and the factor of safety against horizontal sliding of the abutments must be at least 1.5.

A major problem in constructing footings in the prevailing ground conditions will be to control the groundwater and prevent a flow of soil and water into the footing excavations. This can be achieved by constructing the footings inside sheeted enclosures which should penetrate to a depth below the bottom of the excavation equal to the head of water above the bottom of the excavation to prevent 'boiling' or 'bottom heave' occurring due to an out-of-balance hydrostatic pressure.

#### Piling

Due to the difficulties associated with the construction of footings

in the saturated granular subsoil consideration should be given to the use of a piled foundation to transfer the load to a lower and more competent stratum.

Any driven type of pile may be used, however the most common types in this area are steel tube and H-piles. Steel tube piles would not penetrate as deep as H-piles due to the greater cross-sectional area, and working loads of 50 and 65 tons may be used for 10.75 and 12.75 inch diameter tubes respectively. The piles should be driven to refusal which is considered to be reached when 5 blows of an adequate hammer produce a total penetration of 0.25 inch. Based on the borehole results, it is estimated that a suitable set will be achieved at approximately El. 1155. The rating of the pile driving equipment should be not less than 10,000 foot pounds per blow for 50 ton capacity piles, and not less than 13,000 foot pounds per blow for 65 ton capacity piles.

Steel H-piles would penetrate an additional 5 to 10 feet, and the allowable load may be calculated using a stress of 9,000 p.s.i. in the steel.

The above working loads and sets are based on theoretical estimates,

and in practice the assumed working loads should be checked by the use of an approved dynamic pile driving formula. If the desired working load is not achieved within the anticipated depth of penetration, the piles should be extended or additional piles driven to provide the required support for the structure.



CJWA/cs

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED

A handwritten signature in dark ink, appearing to read "C.J.W. Atkinson", written over the typed name.

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

## APPENDIX 'A'

### THE STANDARD PENETRATION TEST.

In order to determine the relative density of non-cohesive soils, such as sands and gravels, the standard penetration test has been adopted. The test also gives an indication of the consistency of cohesive soils.

A two inch external diameter thick-walled sample tube is driven into the ground at the bottom of the borehole by means of a 140 lb. hammer falling freely through 30-ins. The tube is first driven an initial 6-inches to allow for the presence of disturbed material at the bottom of the borehole. The number of standard blows (N) required to drive the sampler a further 12-in. is recorded. The sample tube is one originally developed by Raymond Concrete Pile Company in the United States, where a sufficient number of tests have been made in conjunction with field investigations to show that the results, although essentially empirical, may be applied to foundation design.

For Sands:-

Values of 'N'	Density
Less than 10	Loose
Between 10 and 30	Compact
Between 30 and 50	Dense
Greater than 50	Very dense

# 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						
0	> 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					

## 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.

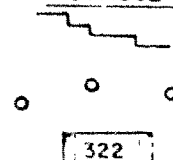
DYNAMIC PENETRATION RESISTANCE : to drive a 2"  $\phi$ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot

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 :



## SOIL PROPERTIES.

W % Water content

LL % Liquid limit

PL % Plastic limit

PI % Plasticity index

LI Liquidity index



Natural bulk density (unit weight)



Void ratio

RD

Relative density

C<sub>v</sub>

Coeff. of consolidation

m<sub>v</sub>

Coeff. of volume compressibility

k Coeff. of permeability

C Shear strength — in terms of total stress

 $\phi$  Angle of int friction

C' Cohesion — in terms of effective stress

 $\phi'$  Angle of int friction

## UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL

UNCONFINED

LABORATORY

FIELD

COMPRESSION TEST

VANE TEST

POCKET PENETROMETER TEST

Strain at failure is represented by direction of stem

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

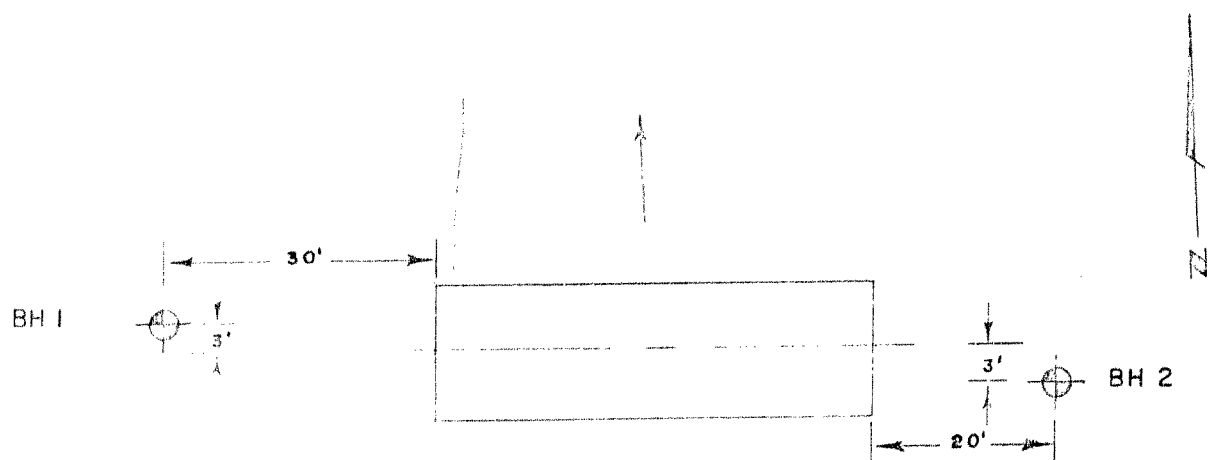
85 - 100 %

Very stiff

2000 - 4000

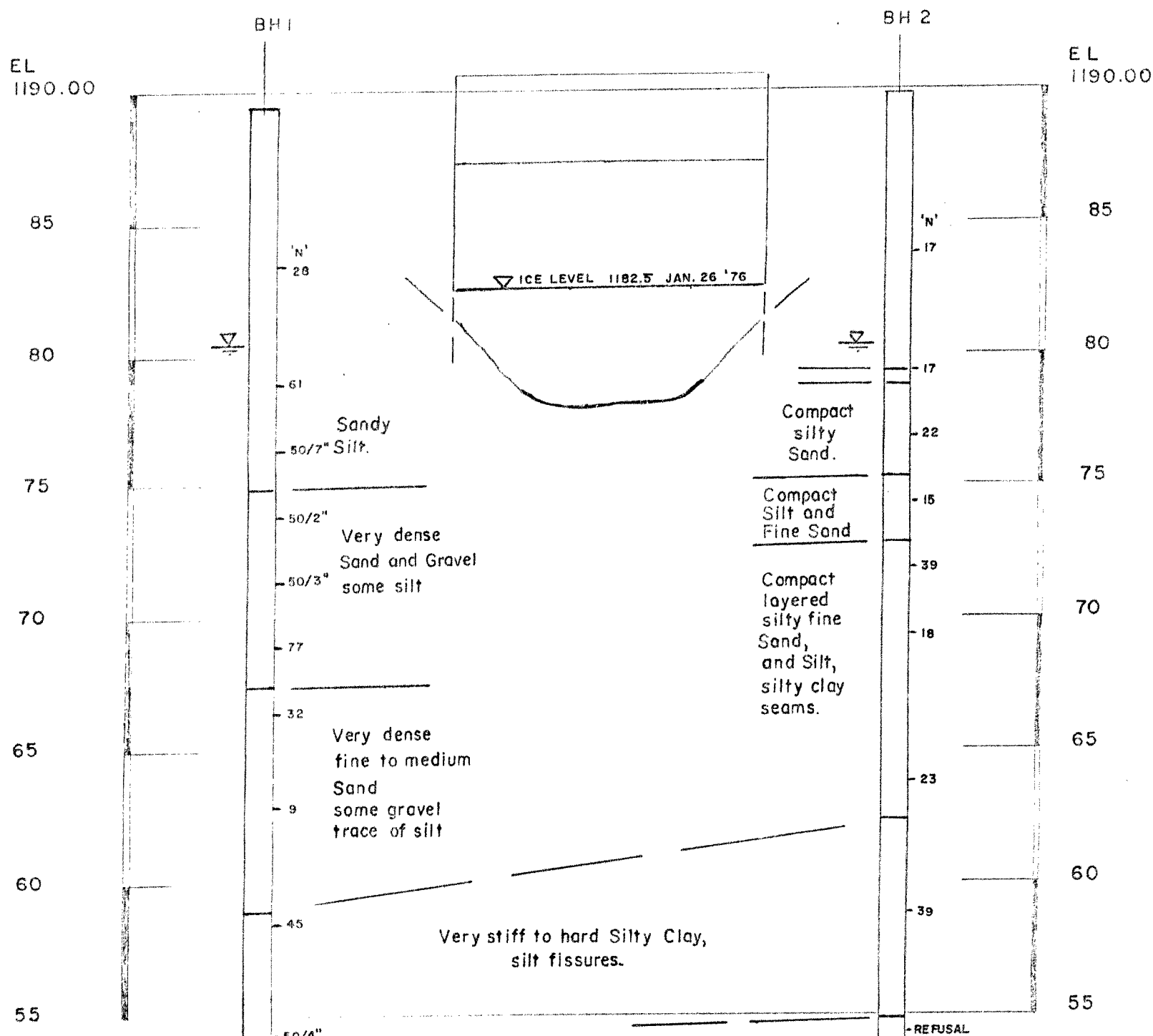
Hard

over 4000



B.M. NAIL IN HYDRO POLE  
STA. 12+69  
EL. 1186.11

SITE PLAN  
Scale 1" = 20'



SUBSURFACE PROFILE  
Scale Hor. 1" = 20'  
Vert. 1" = 5'

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

Our Reference No. 76-1-L7

Enclosure No. 3

CLIENT: B.M. Ross & Associates Ltd

PROJECT: Bridge BR-293

LOCATION: Township of Wallace

DATUM ELEVATION: Nail in hydro pole, El. 1186.11 feet

## DRILLING DATA

Method: Auger

Diameter: hollow-stem

Date: Feb. 6, 1976

[illegible]

VERTICAL SCALE: 1 inch to 5 ft.

DOMINION SOIL INVESTIGATION LIMITED

DRAWN:

CHECKED:

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

Enclosure No. ....4.....

Our Reference No. 76-1-17.....

CLIENT: B.M. Ross &amp; Associates Ltd.

PROJECT: Bridge BR-293

LOCATION: Township of Wallace,

DATUM ELEVATION: nail in hydro pole, El. 1186.11 feet

## DRILLING DATA

Method: Auger

Diameter: hollow-stem

Date: Feb. 6, 1976

SUBSURFACE PROFILE				SAMPLES			PENETRATION RESISTANCE					WATER CONTENT %			REMARKS					
ELEVATION Ft.	DEPTH Ft.	DESCRIPTION	SYMBOL	GROUND WATER	NUMBER	TYPE	'N' Blows/Ft.	20	40	60	80	100	PLASTIC LIMIT	NATURAL		LIQUID LIMIT				
								UNDRAINED SHEAR STRENGTH						p.s.f.			W <sub>p</sub>	W	W <sub>L</sub>	
								+ FIELD VANE TEST    • COMPRESSION TEST									10 20 30 40 50			
1189.8	0.0	Ground Surface																		
85		Sand and gravel. Fill			1	SS	17	○												
80																				
105		Sandy silty clay with wood			2	SS	17	○												
110		Compact silty sand, trace of gravel.			3	SS	22	○												
75																				
145		Compact silt and fine sand			4	SS	15	○												
170																				
70		Layers of compact silty fine sand and silt, seams of silty clay. (laminated)			5	SS	39	○												
65					6	SS	18	○												
60																				
275		Very stiff to hard grey silty clay, silt fissures.			8	SS	39	○												
55																				
350					9	SS	hammer bouncing-no penetration													
50		Very dense sandysilt, with gravel, cobbles and boulders.			10	SS	40/6"													
45					11	SS	100/8"													
40					12	Bx	10% recovery													
49.0		End of Borehole																		

VERTICAL SCALE: 1 inch to 5 ft.

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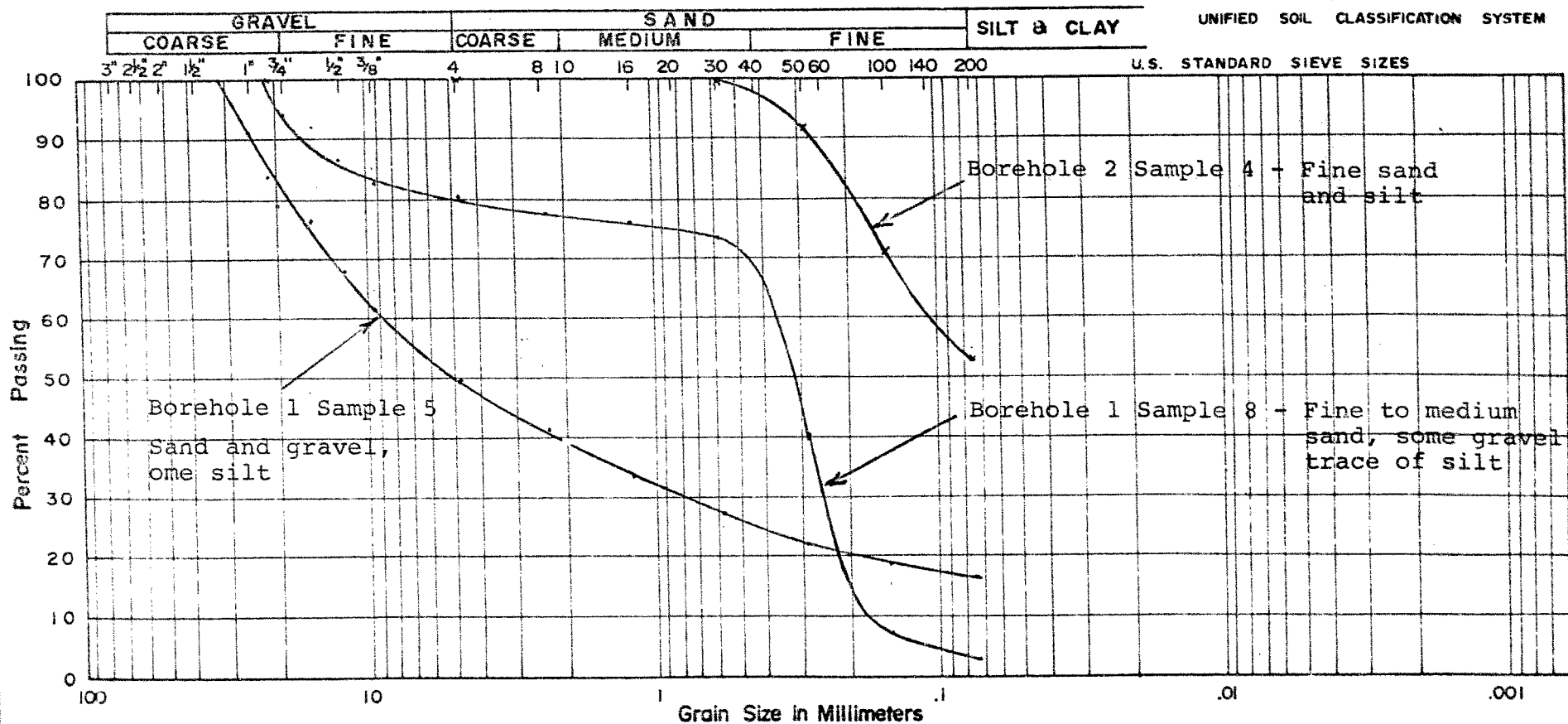
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## GRAIN SIZE DISTRIBUTION

OUR REFERENCE <sup>76</sup>1-L7



PROJECT: Bridge BR-293  
 LOCATION: Township of Wallace  
 BOREHOLE No: 1 1 2  
 SAMPLE No: 5 8 4  
 DEPTH: 18' 26' 16'  
 ELEVATION 1172' 1164' 1174'

COEFFICIENT OF UNIFORMITY :  
 COEFFICIENT OF CURVATURE :

Classification of Sample and Group Symbol:

PLASTIC PROPERTIES

LIQUID LIMIT	% =
PLASTIC LIMIT	% =
PLASTICITY INDEX	% =
MOISTURE CONTENT	% =

ENCLOSURE No 5