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S A L M O N   B R I D G E  
T O W N S H I P   O F   B I N B R O O K

F O U N D A T I O N   C O N D I T I O N S

Submitted by

DOMINION SOIL INVESTIGATION LIMITED  
77 Crockford Boulevard  
SCARBOROUGH - ONTARIO

OUR REFERENCE NO. 2-12-10

DECEMBER 1962

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## INTRODUCTION

A letter of authorization dated November 15th, 1962 was received from Mr. V. R. Astrop, Consulting Engineer, to conduct a foundation investigation at the site of a proposed bridge in the Township of Binbrook.

The existing concrete arch bridge was completed on October 14th, 1913 and at present, it is in an extremely poor condition. The new structure, meeting the requirements of modern traffic, will replace it.

Number and location of the boreholes were determined by the Client and recorded on a drawing supplied to us. The actual positions of the test holes were as close to those desired as practicable under the field conditions.

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

S U M M A R Y

- (1) Different subsoil conditions prevail under the two banks of the creek.
- (2) The bearing capacity of the substrata is generally low, hence spread footings do not appear to be practicable.
- (3) Considering the proximity of the dolomite bedrock (at about 26 ft. depth), end bearing piles should support the foundations.

## I. DESCRIPTION OF SITE AND GEOLOGY

The proposed bridge site lies about two miles south of Binbrook. It carries a gravel surfaced road over the Chippewa Creek which flows eastward. The area is a flat, agricultural plain surrounded by gently undulating country.

This clay plain was the bottom of the Old Lake Erie Basin. The nearly varved clay material is an indication of an ice front close by during deposition. A common term frequently applied to these types of soils is the "till" denoting a sediment of glacial origin.

The bedrock underlying the site is the flat lying dolomite and limestone of the Salina Formation, originated in the Palaeozoic Age. (Dolomite is a chemically formed, calcareous deposit, basically a magnesian limestone. Its appearance is very similar to that of the limestone but it is somewhat harder and only feebly effervescent when in contact with cold hydrochloric acid)

## II. FIELD WORK

Field work was carried out during the period December 6th to 28th, 1962 and comprised two boreholes and four dynamic penetration tests at the location shown on Enclosure #2. The positions of the test holes were set out on the site with the assistance of a drawing provided to us. Elevations were measured relative to the centreline of bridge deck = 85.6 ft.

The boreholes were of 4 in. diameter. They were advanced to the required sampling depths by a mounted, continuous flight power auger. At a later date, Borehole No. One was lined with Bx casing advanced to the surface of the bedrock by the repetitious procedure of alternately driving and washing.

Standard penetration tests were made at frequent intervals using a 2 in. outside diameter split spoon driven into the bottom of the clean borehole over a depth of three times six = eighteen inches and applying a constant

driving energy: 140 lb. hammer dropping 18 ins. (These tests provided disturbed samples from the substrata indicating their relative density and consistency).

The blows to advance the sampler six ins. were counted and thus three values obtained. The first one is discarded because the soil in the vicinity of the bottom of the borehole may have been disturbed by the borings. The second and third values are added and thus the blows required for one foot penetration (= Standard Penetration Resistance) is obtained and recorded on the data sheet. In some cases when the subsoil was so hard that the heavy pounding involved the danger of breaking the sampler, the penetration resistances were computed by extrapolation from a shorter advancement. These values are also indicated on the data sheets together with the details of which they were derived.

The dynamic cone penetration test is one type of deep sounding in which the A rods with a 2 in. diameter 60 degree apex cone driving point are driven into the subsoil without casing and applying the same driving energy as above. These tests provided a continuous record of soil density.

Undisturbed samples were taken with 2 in. diameter, thin-walled tubes forced into the subsoil by pushing.

The in situ shear strength of the cohesive strata was measured wherever the undrained shear strength was less than 3000 lbs. per sq. ft. A four bladed vane 4 ins. long and 2 ins. in diameter with a blade thickness of 1/3 in. was used. After remoulding, the shear strength was checked again thus providing the sensitivity index of the subsoil.

The bedrock was explored by diamond drilling. BxT size 1 5/8 in. diameter core was recovered.

The stratification of the subsoil in terms of depth from surface and elevation, the position and type of samples, the results of the penetration and vane tests together with percentage of core recovery are recorded on

geotechnical data sheets comprising Enclosures #3 and #4.

At the time of the field work, the immediate surroundings of the bridge was inundated with water due to heavy rains. This was one factor why the position of the boreholes had to be changed.

### III. LABORATORY WORK

All samples were shipped to our laboratory where they were thoroughly examined and classified. Those which were the most representative and whose engineering properties were particularly important for the needs of the present project were subjected to a detailed and accurate analysis in the laboratory.

The natural moisture content and the unit weight of the clayey soils are indices which are fairly characteristic. Therefore, these properties were determined on the samples below the assumed footing level. The liquid and plastic limits are specific water contents arbitrarily defined, at which the behaviour of the soils changes. By comparing the natural moisture content with those at the liquid and plastic limits, the probable state of the soil can be seen. The closer the water content to the liquid limits, the softer the subsoil. (The liquidity index expresses this relationship: 
$$\frac{W-PL}{LL-PL}$$
 If the LI is 0, the water content is at the plastic limit; if it is 1, at the liquid limit.)

The unconfined compression tests measure the shear strength of the soil. This property is important when the bearing capacity is being analyzed. The compressibility characteristics are evaluated with the aid of the consolidation test.

The table following is a summary of all laboratory tests. All test results are recorded on the corresponding data sheets except those of the consolidation test. The latter is presented in the form of a function (void ratio = f (log pressure)) and attached as a separate enclosure.

B.H. NO.	SA. NO.	ELEV. FT.	W %	LL %	PL %	PI %	LI	UNIT WEIGHT P C F	VOID RATIO	SHEAR STRENGTH P S F	STRAIN AT FAILURE %
1	1	80	27.4	-	-	-	-	-	-	-	-
	2	77	30.1	-	-	-	-	-	-	-	-
	3	75	32.6	46.4	24.5	21.9	.37	-	-	-	-
	4	73	33.6	-	-	-	-	119	0.9	-	-
	5	71	30.6	-	-	-	-	-	-	-	-
	6	68	27.0	-	-	-	-	120	0.8	920	7
	7	65	38.0	-	-	-	-	111	1.1	620	17.5
	8	64	37.3	-	-	-	-	-	-	-	-
	9	60	32.9	45.3	17.7	27.6	.55	-	-	-	-
2	1	78	32.1	-	-	-	-	-	-	-	-
	2	76	27.4	-	-	-	-	-	-	-	-
	3	73	26.0	-	-	-	-	-	-	-	-
	4	72	24.8	-	-	-	-	-	-	-	-
		71	45.8	-	-	-	-	-	-	-	-
	5	68	36.7	-	-	-	-	112	1.1	394	6.25
	6	67	40.0	46.8	24.5	22.3	.70	-	-	-	-
	7	64	33.7	-	-	-	-	-	-	-	-
	8	58	38.0	-	-	-	-	-	-	-	-



#### IV. SUBSURFACE CONDITIONS

The road slopes to the north rather steeply at the location of the bridge. The morphology of the area suggests that some time in the past, the creek may have occupied a much larger part of the valley than now. Consequently, marshy conditions prevailed on the north shore. This premise was confirmed by local residents and by the borings. Subsoil conditions are therefore different and the two boreholes will be treated separately.

<u>Approx. Elevations</u>	<u>Borehole #1</u>
83 - 79	Organic topsoil and porous, weathered silty clay. These layers are exposed to atmospheric effects, changes in temperature, seepage waters, etc. They have to be excavated.
79 - 75	Stiff, brown, silty clay with silt layers. This material is varved and the two factions were deposited in different seasons of the year.
75 - 68	Stiff, grey, lean clay. The deposit is a glacial till. Silt pockets, scattered gravel is embedded in the main stuff, which is in a stiff consistency.
68 - 57	Firm, grey, lean clay. Its properties are similar to the above stratum; however, the water content is higher - consequently, the material is only <u>firm</u> , that is, has a smaller shear strength and it is more compressible.
57 - 56	Grey, dense, gravelly till. This thin bed is a mixture of angular gravel, sand and silt.
56 -	Grey, dolomite bedrock in a very good condition.

#### Borehole #2

81 - 71	Fill - comprised of material probably excavated not far from the site because the properties of the earth are similar to those of the substrata. It is a weathered, more or less compacted sandy, clayey silt.
71 - 70	Organic clayey silt. The deposit is the remainder of the old, swampy topsoil. Its thickness may have been greater than it presently is but was compressed by the weight of the road fill.

Approx.  
Elevations

- 70 - 57.5      Grey, lean clay occupies this section. The material is about the same as that encountered on the south side but the water content is higher and, as a result, the soil is much softer.
- 57.5 - 57      Grey, dense, gravelly till. The same stuff as encountered in Borehole #2.
- 57 -            Bedrock, confirmed by refusal in the borehole and in the two dynamic cone penetration tests.

The ground water level in the substrata could not be measured because at the time of the field work, water poured into the hole from above. However, it is reasonable to assume that the ground water level follows closely the changes of that in the river due to the permeability of the upper strata.

Water under artesian pressure circulates in the pervious, gravelly silt stratum directly above the bedrock. Its head was at about elevation 84 ft.

V. DISCUSSION AND RECOMMENDATIONS

Two factors determine the foundation elevation of the proposed bridge:

- (i) the footings must be placed at a depth where the subgrade has enough bearing capacity to support the structure without the danger of soil rupture and excessive settlements.
- (ii) the footings must be placed at a depth where the danger of erosion by the creek no longer exists.

By weighing the above requirements, elevation 69 feet seems to be the probable highest base level. Consequently, the south footings would be placed into the stiff grey, lean clay stratum. Taking 900 psf as the average shear strength of the soil (based on the unconfined compression and field vane test results), the behaviour of a 30 ft. long x 8 ft. wide footing was investigated. Using Meyerhof's theory (VI. REFERENCES - (5)) in the analysis of the bearing

capacity and assuming 81 ft. as the average ground level, a gross allowable bearing pressure of 2800 psf is obtained. The settlement of the abutment - based on the results of the consolidation test performed on sample 1/6 - would be about  $1\frac{1}{2}$  in. which is probably on the high side in view of the fact that the clay is preconsolidated (VI. REFERENCES (4)). It must not be forgotten, however, that the qualities of the subsoil deteriorate with depth and it may well be that the more compressible, deeper lying strata will eliminate the gain obtained by taking the geological history of the deposit into account.

The subsoil is considerably softer on the north side. The presence of soft, organic deposits, the lower penetration resistances and higher water contents indicate the worse quality of the material. It is obvious that the bearing capacity would be much less and higher settlements could be expected here.

Considering that the substrata are different on the two sides of the bridge; furthermore, that the allowable bearing pressure - even in the better material - is not sufficient, piled foundations will suit the local circumstances.

The piles should be end-bearing, driven or bored against the bedrock. The dolomite is in a satisfactory condition and the compressive strength may be around 35,000 tons per square foot (VI. REFERENCES (6)). Taking 3 as a safety factor, 11,600 tons per sq. ft. is obtained as allowable bearing pressure. The National Building Code of Canada states 20 tons/sq. ft. as allowable bearing pressure for sedimentary rocks, which is very conservative. We may conclude that the bearing capacity of the piles as structural members will be the governing factor.

Any type of pile (steel, reinforced concrete or timber) will be suitable. Should steel piles be chosen, slightly more corrosion allowance should be taken into account, considering the organic content on the north side.

No construction problems are envisaged. The drainage around the footings should be carefully planned to collect all eventual waters that may seep upwards along the piles.

DOMINION SOIL INVESTIGATION LIMITED,



L. S. Rolko, P. Eng.,  
CHIEF SOILS ENGINEER.

LSR/oed

#### VI. REFERENCES

- (1) Procedures for Testing Soils, ASTM, April 1958, pp 186 to 198. (Unified Soil Classification System - by A. A. Wagner).
- (2) Terzaghi and Peck: Soil Mechanics in Engineering Practice, John Wiley and Sons, New York, 1948.
- (3) The Physiography of Southern Ontario by L. J. Chapman and D. F. Putnam of the Ontario Research Foundation - University of Toronto Press 1951.
- (4) A Contribution to the Settlement Analysis of Foundations on Clay by A. W. Skempton and L. Bjerrum - Geotechnique VII, (1957) and Amendment Thereto by A. M. Muir Wood (Correspondence, Geotechnique - Vol. IX).
- (5) The Ultimate Bearing Capacity of Foundations by G. G. Meyerhof, Geotechnique, Vol. II, 1950 & 1951.
- (6) Principles of Engineering Geology and Geotechnics, Krynine & Judd, McGraw-Hill, 1957.

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
2	> 8"	3"	3/4"	4.75mm	2.0	0.42	0.074	0.002	>			
U.S. Standard Sieve Size		No. 4		No. 10		No. 40		No. 200				

## 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.

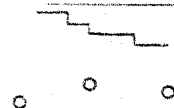
**DYNAMIC PENETRATION RESISTANCE** - to drive a 2" dia. 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

**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:



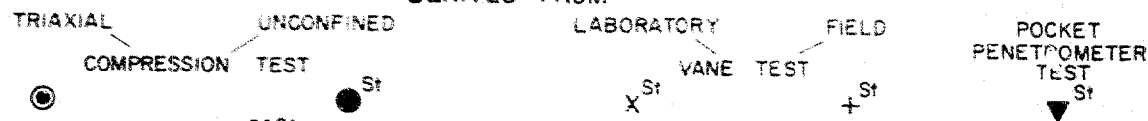
322

## SOIL PROPERTIES.

W %	Water content	$\gamma_s$	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	$\phi$	Angle of int. friction in terms of effective stress
PI %	Plasticity index	C <sub>v</sub>	Coeff. of consolidation	C'	Cohesion
LI	Liquidity index	m <sub>v</sub>	Coeff. of volume compressibility	$\phi'$	Angle of int. friction

## UNDRAINED SHEAR STRENGTH.

- DERIVED FROM -



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 %	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



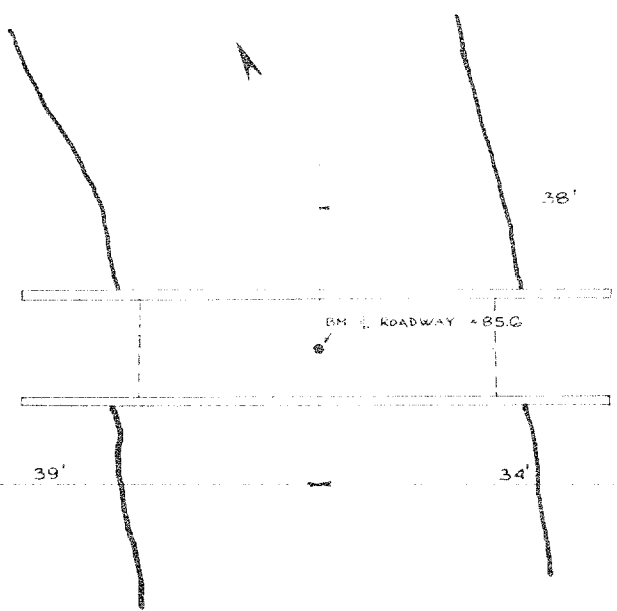
CONE 4  
BH & CONE 2

BH & CONE 1

15'

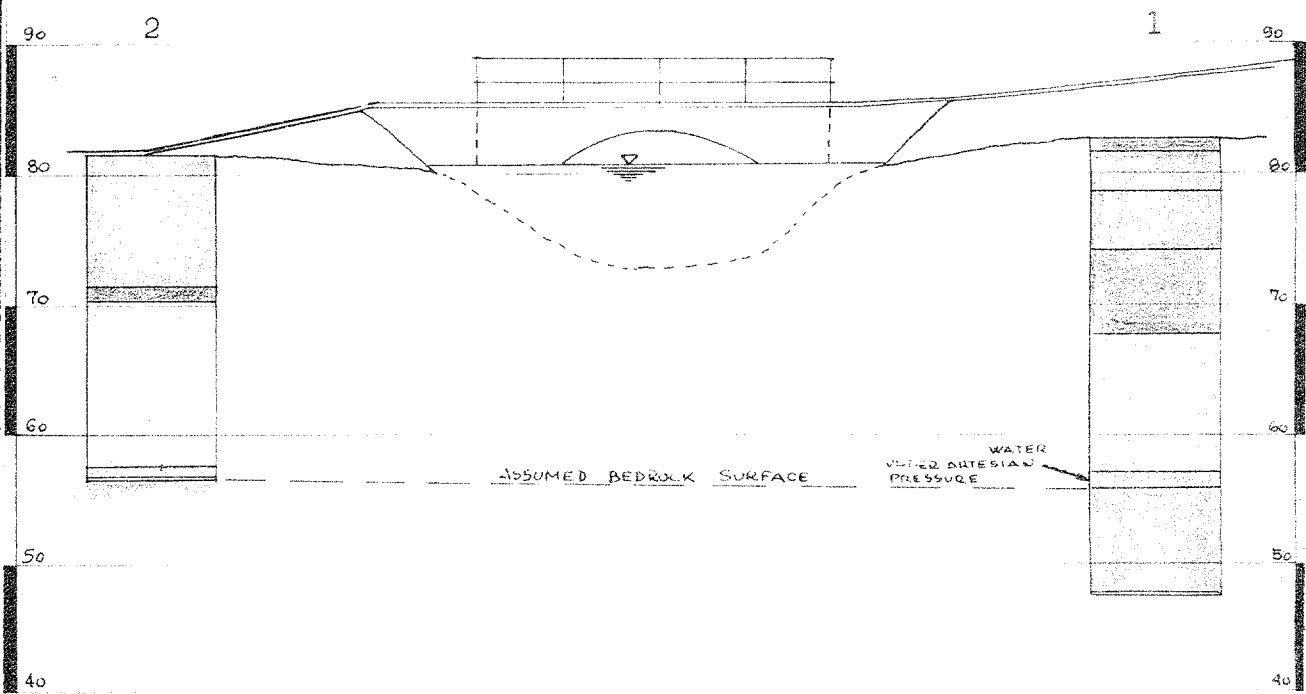
13'

CONE 3



## LOCATION OF BOREHOLES

SCALE: 1" TO 10'



## SUBSURFACE PROFILE

SCALE: 1" TO 10'

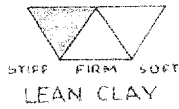
### LEGEND



TOPSOIL



WEATHERED SILTY CLAY AND CLAYEY SILT



STIFF FIRM SOFT LEAN CLAY



GRAVELLY TILL



DOLOMITE BEDROCK

# 2-12-10 GEOTECHNICAL DATA SHEET FOR BOREHOLE 1 & CONES 1 & 3

CONS. REFERENCE NO. 2-12-10

CLIENT V. R. ASTROP, CONSULTING ENGINEER

PROJECT SALMON BRIDGE

LOCATION TOWNSHIP OF BINBROOK

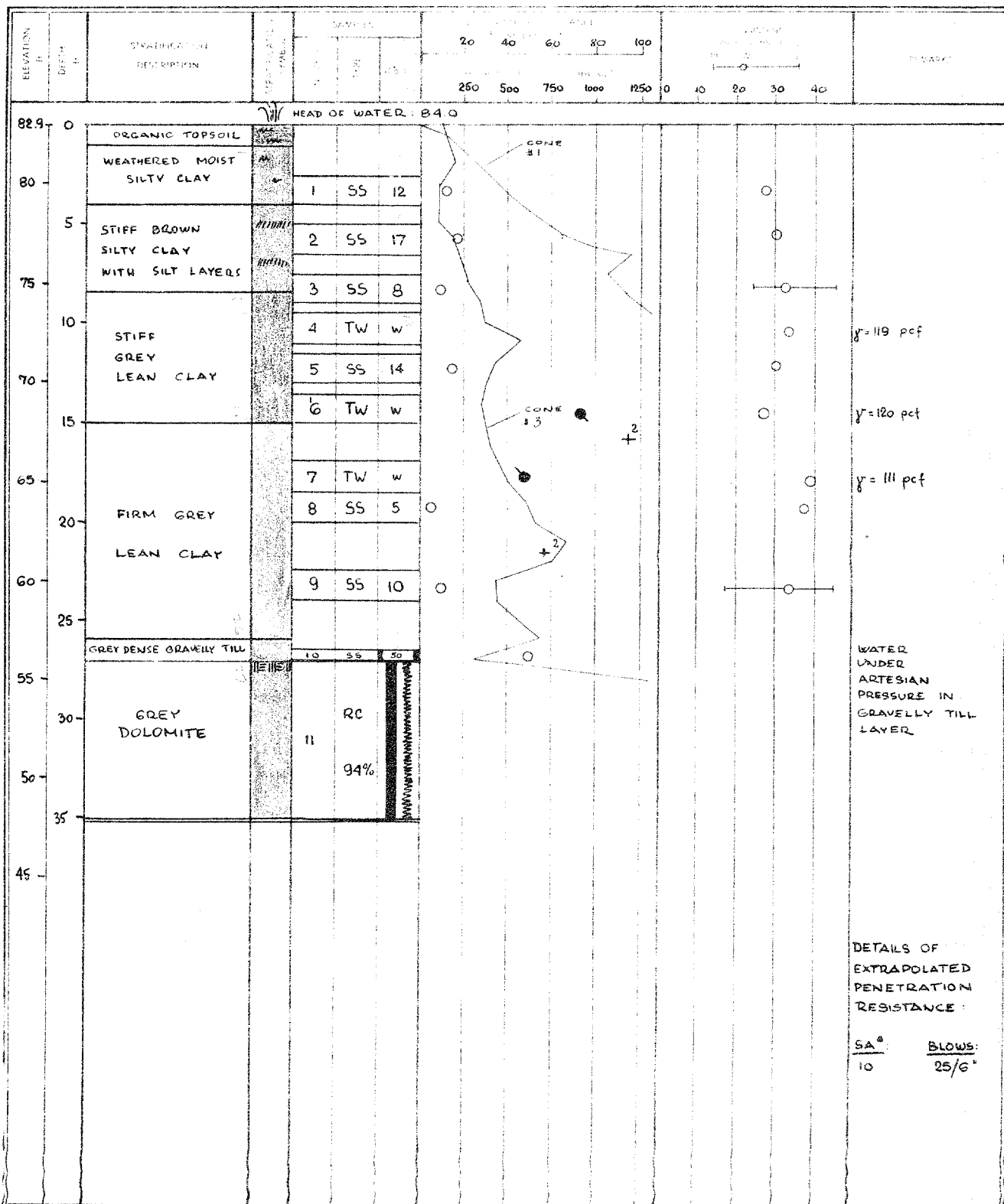
DAILY ELEVATION DECK OF EXISTING STRUCTURE: 65.6

TESTING METHOD AUGERING

TESTING EQUIPMENT 4 1/2"

DATE DEC. 11, 1962.

NO. OF TESTS 3





DATA REFERENCE NO.

# 2-12-10 GEOTECHNICAL DATA SHEET FOR BOREHOLE 2 & CONES 2 & 4

CLIENT V.R. ASTROP, CONSULTING ENGINEER

METHOD OF BORING AUGERING

DATE 12/11/10 4

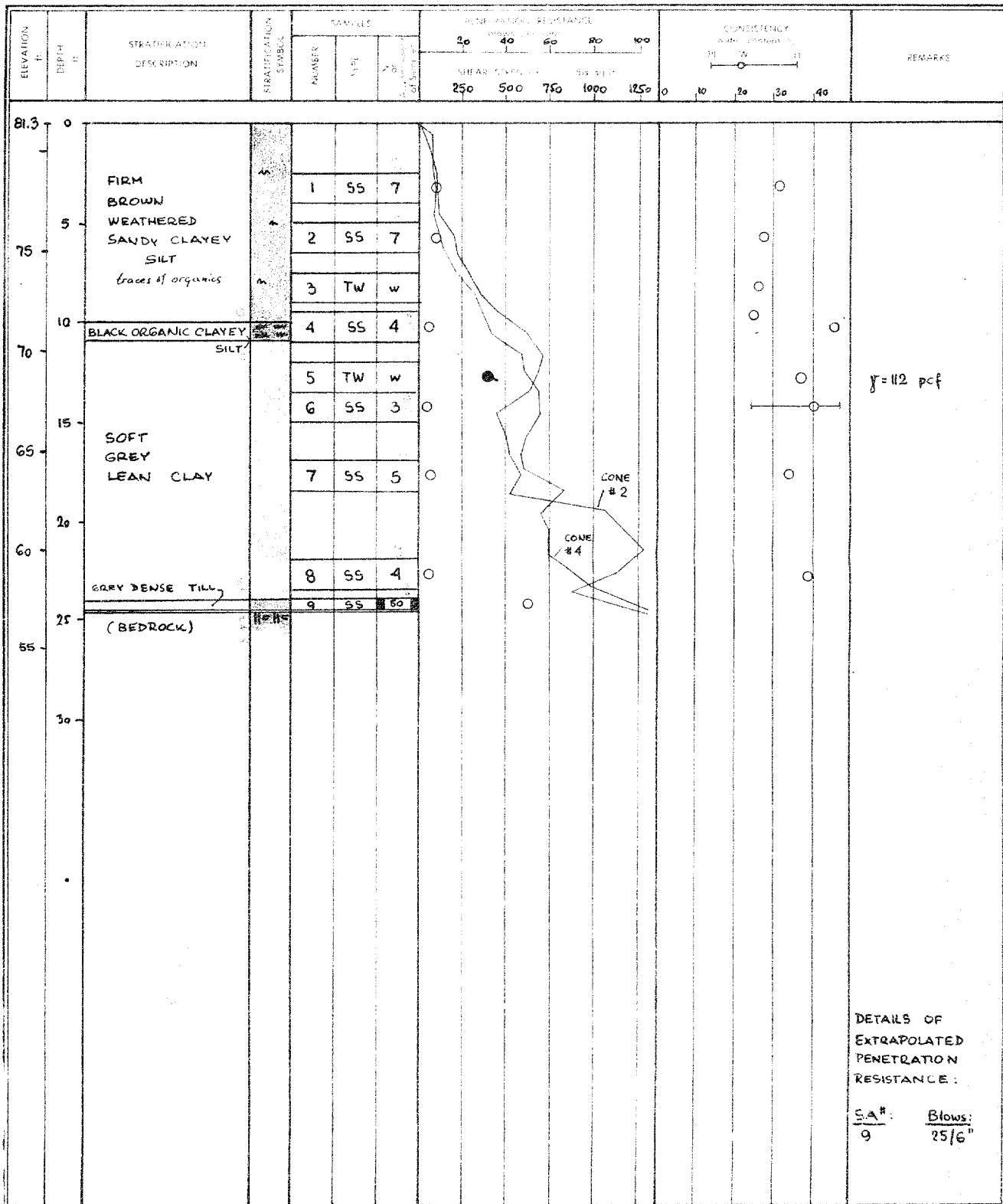
PROJECT SALMON BRIDGE

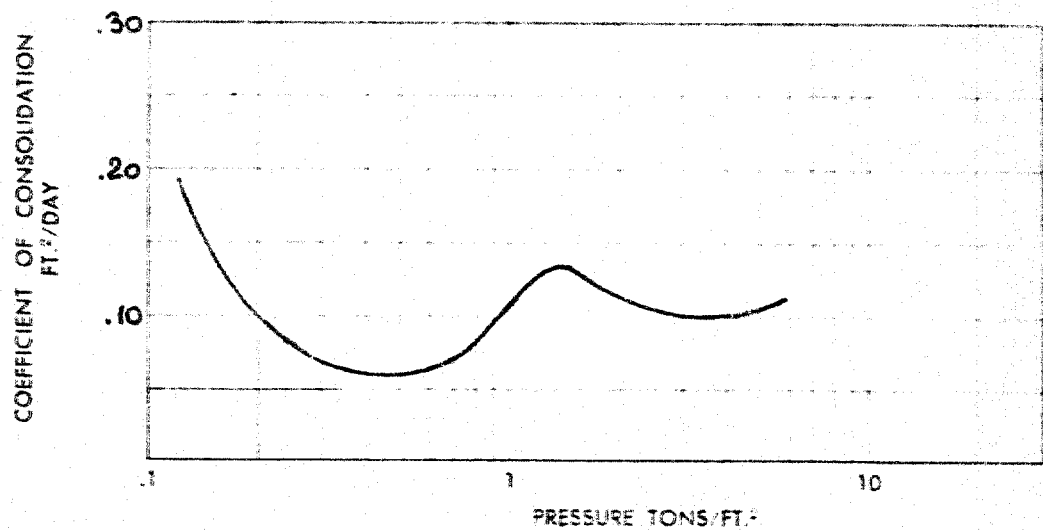
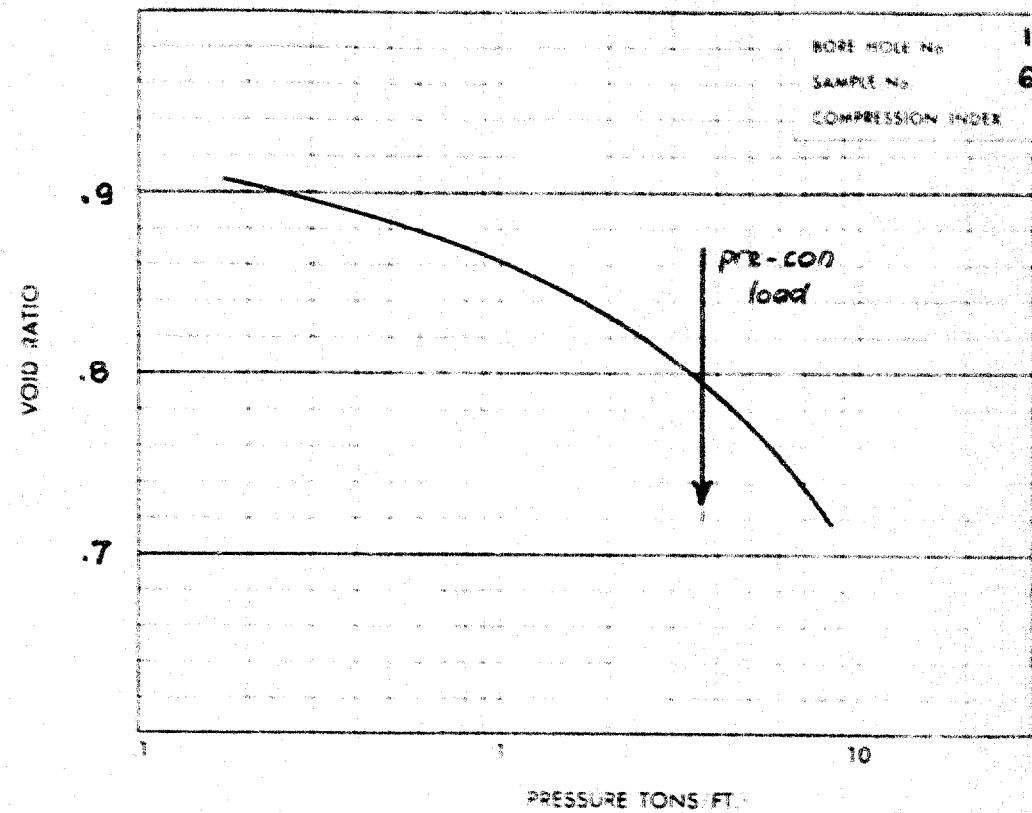
WATER TABLE DEPTH 4 1/2'

LOCATION TOWNSHIP OF BINBROOK

DATE DEC. 11, 1962.

DATUM ELEVATION DECK OF EXISTING STRUCTURE: 85.0



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

#63-F-287M  
SALMON BRIDGE  
BINBROOK TWP.  
LOTS <sup>#</sup>28 & <sup>#</sup>29  
CONCESSION IX

