

62 - F - 317 M

PHILLIPSBURG BRIDGE

LOT 18/19 , CON. S

ERB RD.

WILMOT TWP.

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attn: Mr. K.L. Kleinsteinber,
Municipal Bridge Liaison Engr.

Mr. A. G. Stermac,
Foundation Section,
Materials & Research Division.

September 14, 1962.

Re: County of Waterloo, Phillipsburg
Bridge, Twp. of Wilmot, Lot 18/19
Con. S, Erb Rd., Structure Site #34-72
Bridge Office Ref. No. B.A. 1492.

We have reviewed the Foundation Report by
Dominion Soil Investigation, Ltd., for the above mentioned
site, and submit the following comments:-

We suggest two alternative proposals for the
pier foundations provided they meet the hydrological
requirements:

1) Footings founded on dense clay till between
Elev. 1122.5 and Elev. 1119.0, will provide a safe bearing
load of 3 T.S.F. If sheeting is required for scour protection
this should be incorporated in the footing design.

2) Footings for piers can also be supported on
steel tube piles (12 $\frac{1}{2}$ " O.D. x 0.25"), driven to practical
refusal into the dense stratum of sand and gravel. For piles
driven to this depth, a safe design load of 60 T/pile may be
used for design work.

At the time of writing this memo, we do not have
the hydrology report. However, the piles driven to practical
refusal into the sand and gravel stratum, should be deep enough
to meet the requirements of scour protection.

If we can be of further assistance in this matter,
please contact our Office.

MD/MdeF

cc: Foundations Office
Gen. Files.

M. Devata
M. Devata,
SR. FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.



ONTARIO

DEPARTMENT OF HIGHWAYS

Stratford, Ont., September 11, 1962.

MEMORANDUM TO

Mr. K.L. Kleinsteinber,
Municipal Liaison Bridge Engineer,
Department of Highways Ontario,
Parliament Buildings,
Toronto 5, Ont.

Re: County of Waterloo
Phillipsburg Bridge.

Enclosed herewith please find duplicate copies of a revised Soil report for the above mentioned project. This revision was made because of certain discrepancies in elevations between the original Dominion Soils Investigation Ltd. report and those of the Survey notes of the Consultants McCargar, Fife & Hargbourn Ltd. The Consultants have also requested that the original reports submitted with the preliminary data sheet be destroyed.

The Consultants also requested a hydrology report in their original submission and in reply to your memorandum on this of September 5th, 1962 we would advise that the Consultants would like this as early as possible in October so that preparation of drawings and specifications could be completed in the following two months.

Water elevation should be 1131.0 (see this report) and lowest elevation of stream bed is 1127.5. This information in reply to your teletype of this date.

Yours very truly

W.H. Venn
District Municipal Engineer.

WHV/m



MESSRS. McCARGER, FILER AND HACHBORN LIMITED
CONSULTING ENGINEERS
30 Francis Street South
KITCHENER ONTARIO

Report on
SOIL INVESTIGATION
for
COUNTY ROAD BRIDGE
NEAR PHILLIPSBURG, TOWNSHIP OF WILMOT
COUNTY OF WATERLOO

62-F-317M

by
DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 2-6-L4

June 1962

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INTRODUCTION

In accordance with a letter of authorization dated 14th June 1962 from Messrs. McCarger, Filer and Hachborn Limited, a soil investigation has been carried out at a site in the Township of Wilmot where it is proposed to replace an existing single-span county road bridge with a new structure.

The writer visited the site on June 20th, 1962 with Mr. E.G. Hachborn to discuss the engineering requirements of the project. It is understood that the new structure will be of 3-span design with an approximate span arrangement of 50-65-50 feet. The proposed positions of the new piers and abutments were located approximately on the site, and boreholes were placed as close as practicable to these support points.

The purpose of this investigation was to reveal the subsurface conditions and to determine the necessary soil properties for the design and construction of foundations. Because of the known presence of an area of deep fill near the present south abutment, and the possibility that deep footings would be required to prevent scour below the piers of the new structure, attention was drawn specifically to the choice of the best type of foundation in the prevailing conditions.

I DESCRIPTION OF SITE AND GEOLOGY

The site is located at a crossing of the River Nith on a county road running south from Wellesley to Hwy. #7A and #8A. The Village of Phillipsburg is approximately one-half mile to the north. There is evidence from the condition of the bearings on the existing bridge that the abutments have moved together, and this is most probably a result of erosion of the stream bed.

The area lies in the southwest of the Stratford Till Plain which occupies most of Perth County, extending into Middlesex and Wellington, and including a narrow strip on the western boundary of Waterloo County. It is an extensive, heavily glaciated region and includes much of the first land to be uncovered in Ontario by the recession of the Wisconsin Glacier, i.e. at the end of the last ice era. The plain is cut by many spillway valleys formed by the melting waters of the glacier and now occupied by lesser streams. The Nith follows the general course of such a spillway and meanders along its wide floor, flowing southward to join the Grand at Paris.

The steeply rising ground to the north of the site is apparently the slope of the original spillway while to the south the ground is relatively flat. Here the spillway floor is almost a mile wide and gives the appearance of a plain.

Traces of soft organic material found in the most southerly borehole at the level of the present river bed suggest that the river has moved northward, probably employing the conventional mechanics of a meanders, and may presently be eroding and steepening the north bank.

II FIELD WORK

Field work was carried out during the period 21st to 26th June 1962 and consisted of 4 boreholes at the locations shown on enclosure 2. The holes were washbored and lined with Bx casing. Standard Penetration tests were made at frequent intervals and dynamic cone penetration tests were made adjacent to each borehole. The former test provided disturbed samples of the strata and a measure of relative density or stiffness, while the latter test gave a continuous record of soil density in the upper layers.

Undisturbed samples were recovered from the cohesive stratum in borehole 1 using 2-inch diameter thin-walled tubes, and insitu vane shear tests were performed using a 2-inch diameter 4-bladed vane.

Axt core was recovered from a boulder at a depth of 21 feet in borehole 2.

The results of the field tests are recorded on geotechnical data sheets comprising enclosures 3 to 6. Elevations have been referred to a benchmark on the existing bridge (cut cross on northeast wingwall, El. 1154.66 feet).

III SUBSURFACE CONDITIONS

Details of the stratification at each borehole are given on the data sheets and a general picture is provided by the subsurface profile shown on enclosure 2. The conditions were to some extent different at every hole, so they will be described separately.

Borehole 1

Under a shallow surface layer of clayey fill, a very stiff grey clay, containing many fine silt seams, extends to a depth of 22 feet from surface. A layer of fine sand was found within the clay between 17 and 19 feet. Below the clay a layer of dense, cohesionless silt containing a few clay seams, extends for a further 4 to 5 feet, until a hard grey clay till is encountered. This material contains 5 to 10% of subangular gravel generally less than 1 inch in diameter. Finally the hole was terminated in a deposit of very dense, clayey gravel and sand made up of all sizes of rounded and subangular granular particles up to 3/4-inch diameter.

Borehole 2

Boulders, cobbles, gravel and unconsolidated clayey sediments form the first 5 to 6 feet, and below this a hard grey clay till extends to 17 feet from surface. The till contains 5 to 10% of subangular gravel, generally less than 1 inch in diameter, to a depth of 13'6". Thereafter the gravel content increases and at a depth of 14'6" artesian conditions were encountered. Water flowed steadily from the top of the borehole for two days

after completion until the hole was plugged. The source of water is apparently the underlying gravel deposit, starting at 17 feet, which rises to the north and thus provides a head of water. A boulder approximately 12 inches in diameter was encountered in the very dense gravel layer at 20'6" to 21'6".

Borehole 3

A hard clay till extends from surface to a depth of 19 feet. The till is more gravelly here than in other locations, the proportion of granular material varying up to 50% with particle sizes up to 2 inches. The dense gravel deposit was encountered at 19 feet.

Borehole 4

A loose brown clayey sand fill containing traces of organic matter extends from surface (near the road bed) to a depth of 9 feet. Below this is 5 feet of stiff brown clay till, and then a second layer of loose clayey sand extending to 21'6". The lower 18 inches of this sand layer contains a high proportion of compressible organic matter, and the fact that this elevation corresponds to the present river bed suggests that this area may previously have been under water.

At 21'6" the first dense deposits are encountered, first a thin layer of gravel and then a hard grey silt till. The till is predominantly silty, but contains a small fraction of clay and 2 to 3% of fine gravel particles. Below the silt till there are alternating layers of very dense gravel and hard till as illustrated on enclosure 6.

IV LABORATORY WORK

A laboratory testing programme was carried out to examine the characteristics of the clay deposit encountered at borehole 1, in the event that the north abutment might be carried on a spread footing. This consisted of:

- (a) Determination of Atterberg limits for the purpose of classification and as a guide to the behaviour of the soil.
- (b) Unconfined compression tests to determine shear strength (and thus load bearing capacity) and to provide confirmation for the field vane results.

- (c) One consolidation test to examine the compressibility of the clay and to determine the load - settlement - time relationship.

The results from (a) and (b) are recorded on the data sheets and a summary is given on enclosure 8. The consolidation test results appear on enclosure 7.

The tests show the material to be a very stiff clay of intermediate plasticity with a natural moisture content just above the plastic limit. A consideration of the shear strength values from the vane and unconfined compressive strength tests leads to a choice of the value 3000 p.s.f. for the cohesive shear strength for use in calculations.

The estimated preconsolidation load (from the shape of the e-p curve) is 4.4 t.s.f., a high value which is in keeping with the glacial history of the region. The present overburden pressure at the level of the consolidation test sample is 0.55 t.s.f. giving an overconsolidation ratio of approximately 8.

For the purpose of construction, the foregoing physical properties mean that the clay will be stiff, comparatively dry, and easy to work with.

V

FACTORS AFFECTING THE CHOICE OF FOUNDATIONS

The variable soil conditions require that the choice of foundation type should be given special consideration. Factors influencing this choice are as follows:

- (a) At the north abutment the prevailing soil conditions are suitable either for a conventional spread footing or for several types of pile. The only special factor here is the stability of the slope which depends on the future behaviour of the river. If a footing is used in this location the slope must be protected from erosion.
- (b) At the north and south piers the location of footings depends on the depth required for scour protection. This in turn will be defined by a hydrological report. The lowest point on the river bed is at El. 1127 feet so that assuming a minimum scour allowance of 5 feet, footings could be placed at El. 1122. Artesian conditions were encountered

in borehole 2 at El. 1117.5, or 4.5 feet below this level, and it is felt that this is hardly an adequate depth of material to ensure the exclusion of artesian water from the excavation. Remembering that only one point on the footing area (which may be 40 or more feet wide) has been examined, it is quite conceivable that water under pressure may be encountered at higher elevations in other locations.

The foregoing reasoning leads to the conclusion that only a piled foundation will be satisfactory at the north pier. At the south pier there is no evidence to exclude the use of a footing.

- (c) At the south abutment the highest satisfactory bearing stratum is located at El. 1131.5 feet or 21'6" below the present road level. Also the soil at this level is pervious. To avoid a very deep excavation in addition to dewatering problems, the use of piles is clearly indicated.
- (d) Whereas footings may be employed at the north abutment and south pier, piles provide the best solution at the south abutment and north pier. It may be uneconomic, or unsatisfactory from the designer's viewpoint, to combine these two types of foundation in one structure. In either case the best solution is to use piles at all locations.
- (e) The choice of type of pile is influenced by the ground water conditions, the hard driving which will be experienced in the dense till and gravel, and on the scour requirements. A cast-in-place concrete pile is not suitable in the artesian conditions at borehole 2. Steel pipe piles will probably set around El. 1115 at boreholes 2 and 3 but it is conceivable that they will not reach this level if very hard layers are encountered, e.g. at El. 1122 in borehole 3. In such a case the pile tips may not be deep enough to meet the requirements of scour protection.

The only type of pile which will satisfy all the conditions without question is a steel H-section. Such piles are commonly driven in accordance with the Hiley formula and reach considerable depths in very dense soil. It is impossible to predict with accuracy where the piles will set in this case, but as a rough guide they will probably set within 10 feet below the surface of the deep gravel and sand stratum.

VI BEARING CAPACITY AND SETTLEMENT OF FOOTINGS

For the reasons set out in the foregoing section the use of spread footings appears improbable. However, in case the information should be required, the maximum gross soil pressures which should be used in the design of footings are given in the following table.

Borehole	Elevation (feet)	Soil Pressure (p.s.f.)
1	1154 to 1136	5,000 *
	1136 to 1131	9,000
	1131 to 1126	12,000
2	1125 to 1122	7,000
	1122 to 1115	12,000
	1115 to 1100	15,000
3	1127 to 1122	7,000
	1122 to 1110	10,000
4	1131 to 1129	5,000
	1129 to 1120	10,000

Provided the footings are poured on an undisturbed grade the total settlement associated with the foregoing values is not expected to exceed one inch except in the clay layer at borehole 1. The deflection will occur almost immediately as the loads are applied.

If a footing is used at the north abutment the stability of the final design arrangement should be checked in relation to the slope.

* The bearing capacity of a footing 5 feet wide and 40 feet long with an effective depth of zero has been calculated according to the theory of Meyerhof. The ultimate bearing capacity is found to be 15,960 p.s.f. Applying a factor of safety of 3 the allowable soil pressure is 5320 p.s.f. Using this figure the consolidation settlement has been estimated according to Csetovich and, applying the Skempton-Bjerrum correction factor, is found to be 0.65 inches.

VII CONSTRUCTION

The use of piles will eliminate all construction problems related to excavation.

In the event of footings being used either at the north abutment or south pier no unusual difficulty is foreseen in dewatering the excavations. At the north abutment no flow of groundwater will occur within the clay layer. At the south pier the excavation should be surrounded by a sheet pile coffer dam, with the piles driven 2 to 3 feet below the footing grade level.

VIII SUMMARY

1. The strata consist of dense heavily preconsolidated glacial deposits. A profile is shown on enclosure 2. Only at the location of the south abutment is there any soft or loose soil, and here the dense deposits are encountered at 21'6" below the present road level.
2. Artesian water was encountered in the clay till on the site of the north pier at El. 1117.5 feet.
3. The soil conditions at boreholes 2 and 4 would make the use of footings difficult, if not impracticable, in these areas.
4. The use of piles throughout the structure appears to be the most practical solution, and only steel H-sections are certain to be satisfactory in all conditions.
5. Soil pressures for use in the design of footings, if required, are tabulated on page 7.
6. If a footing is used at the north abutment, the stability of the final design arrangement should be checked in relation to the adjacent slope. Also the slope must have permanent protection from erosion.

IX REFERENCES

1. The Physiography of Southern Ontario by L.J. Chapman and D.F. Putman of the Ontario Research Foundation, University of Toronto Press, 1951.

2. Procedures for Testing Soils, ASTM, April 1958, pp. 186 to 198. (Unified Soil Classification System, by A.A. Wagner)
3. The Ultimate Bearing Capacity of Foundations by G.G. Meyerhof, Geotechnique, Vol. II, 1950 and 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. N.A. Csutovich, Calculation of Settlement, Extract from "Foundation Engineering" by Ch. Szechy, Budapest 1957, transl. by L.R. Szalatkay.
6. The Measurement of Soil Properties in the Triaxial Test, by Bishop and Henkel, London, 1957.
7. Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering (Research on Determining the Density of Sands by Spoon Penetration Testing, by H.J. Gibbs and W.G. Holtz of the United States Bureau of Reclamation) London 1957.
8. Terzaghi and Peck: Soil Mechanics in Engineering Practice. John Wiley and Sons, New York, 1948.
9. T.W. Lambe, Soil Testing for Engineers, Massachusetts Institute of Technology, John Wiley, 1951.
10. R.D. Chellis, Pile Foundations, McGraw Hill, 1961.



JP/mc

DOMINION SOIL INVESTIGATION LIMITED

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

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						
Ø	> 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	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.

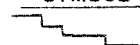
DYNAMIC PENETRATION RESISTANCE : to drive a 2" ϕ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot.

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	γ^* 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	ϕ Angle of int friction
PI % Plasticity index	C_v Coeff. of consolidation	C' Cohesion in terms of effective stress
LI Liquidity index	m_v Coeff. of volume compressibility	ϕ' Angle of int friction

UNDRAINED SHEAR STRENGTH.

— DERIVED FROM —

TRIAXIAL COMPRESSION TEST

20%
15% — 5%
10%

Strain at failure is represented by direction of stem

St

LABORATORY VANE TEST

X St

+ St

POCKET PENETROMETER TEST

St

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

SOIL DESCRIPTION.

COHESIONLESS SOILS :

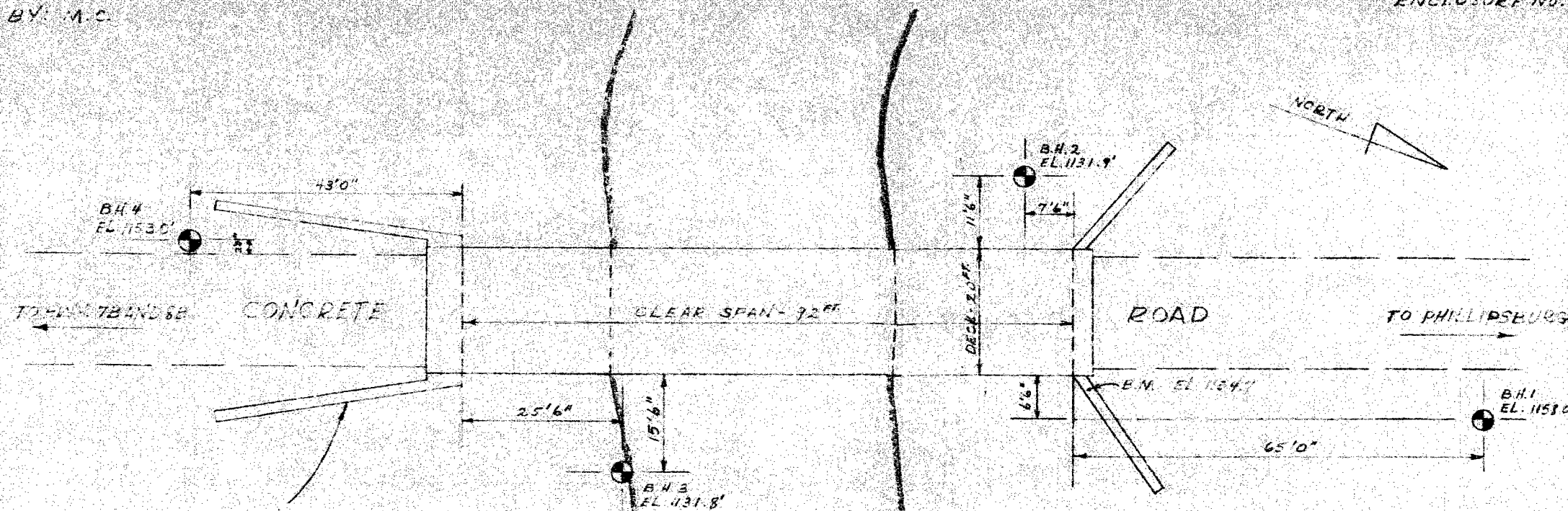
RD :

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

COHESIVE SOILS :

C lbs/sq.ft.

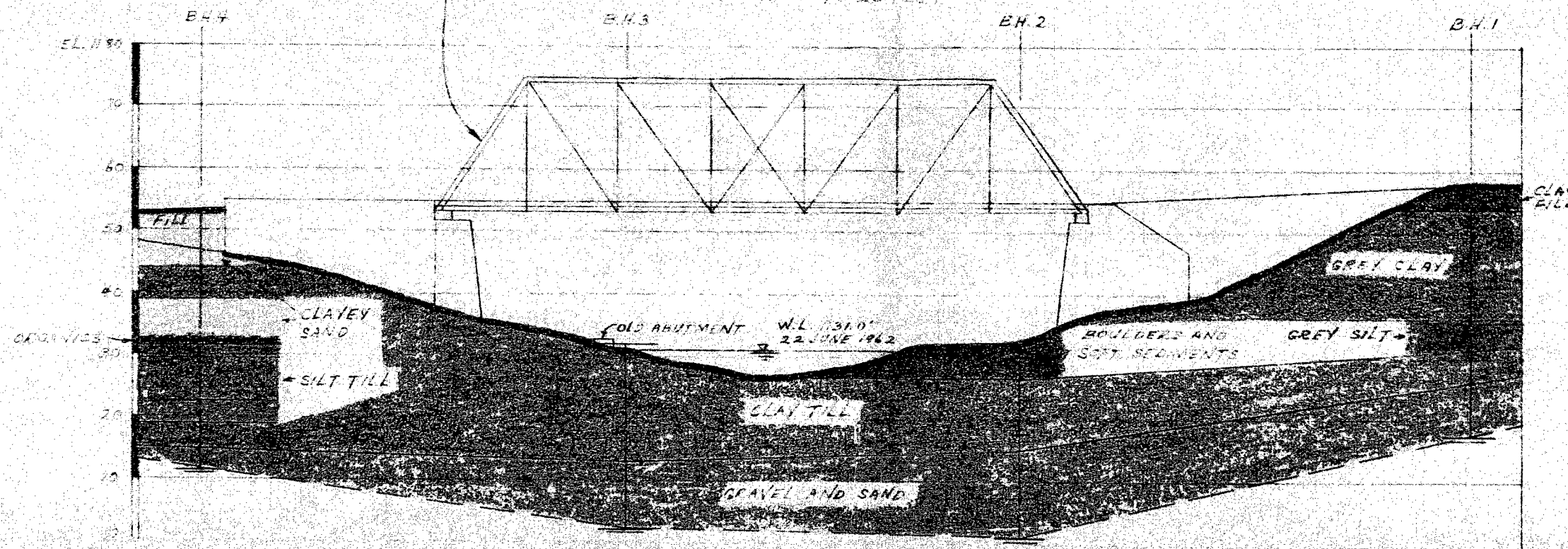
Very soft	less than 250
Soft	250 - 500
Firm	500 - 1000
Stiff	1000 - 2000
Very stiff	2000 - 4000
Hard	over 4000



OUTLINE OF
EXISTING BRIDGE

LOCATION OF BOREHOLES

SCALE: 1 INCH TO 20 FEET



SUBSURFACE PROFILE (LOOKING WEST)

SCALE: 1 INCH TO 20 FEET

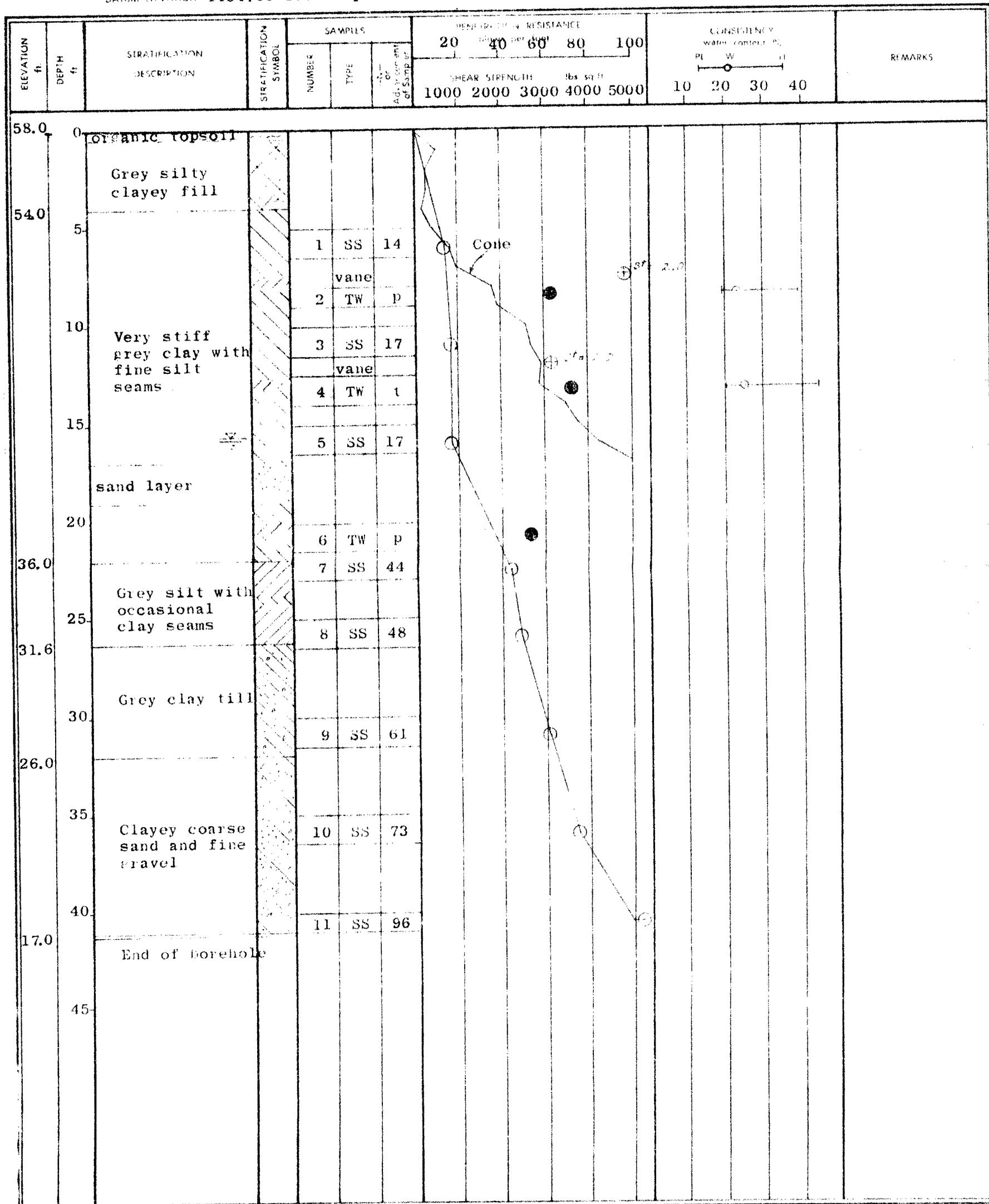
DUNNIN SOIL INVESTIGATION LIMITED

GEOTECHNICAL DATA SHEET FOR BOREHOLE . 1 . . .

OUR REFERENCE NO. 2-6-L4

CLIENT: McCarrar, Filer and Hachborn, Limited METHOD OF BORING: Washboring
PROJECT: County Road Bridge near Phillipsburg DIAMETER OF BOREHOLE: Bx (2-7/8")
LOCATION: Township of Wilmot, County of Waterloo DATE: 25 June 1962
DATUM ELEVATION: 1154.65 feet top of northeast wingwall

ENCLOSURE NO. 3



GEOTECHNICAL DATA SHEET FOR BOREHOLE . . . 2 . . .

OUR REFERENCE NO. 2-6-L4

CLIENT McCargar, Filer and Hachborn, Limited
PROJECT County Road Bridge near Phillipsburg
LOCATION Township of Wilmot, County of Waterloo
DATE 21-22 June 1962
DATUM ELEVATION: 1154.66 feet top of northeast wingwall

METHOD OF BORING Washboring
DIAMETER OF BOREHOLE Bx (2-7/8")

ENCLOSURE NO. 4

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE lb./sq. in. (100 lb. test)					CONSISTENCY water content %			REMARKS
				NO. TEST	TYPE	NO. OF Ac. or Samp.	20	40	60	80	100	PL	W	LI	
1132.0	0	Boulders, cobbles, gravel in unconsolidated clay sediment													
	5			1	SS	5									
26.5				2	SS	29									
	10	Hard grey clay till		3	SS	50									
	15	Gravelly		4	SS	89									
15.0				5	SS	117									
	20	Boulder		6	RC	50%									
		Gravel and sand													
	25			7	SS	80									
	30			8	SS	82									
01.0		End of borehole													
	35														

Artesian water encountered at 14'6" (El. 1117.5 ft.)

OUR REFERENCE NO. 2-6-14

GEOTECHNICAL DATA SHEET FOR BOREHOLE 3

CLIENT: McCarrar, Filer and Hachborn, Limited
PROJECT: County Road Bridge near Phillipsburg
LOCATION: Township of Wilmot, County of Waterloo
DATE: 23 June 1962
DUM ELEVATION: 1154.66 feet top of northeast wingwall

ENCLOSURE NO. 5

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE Blows per foot					CONSISTENCY water content % PI W LI	REMARKS
				NUMBER	TYPE	No. of Advancement of Sampler	20	40	60	80	100		
1132.0	0	organics											
	5	Sandy, gravelly clay till		1	SS	30							
				2	SS	42							
	10			3	SS	114							
	15			4	SS	42							
13.0	20	Gravel and sand (all sizes)		5	SS	40							
	25			6	SS	1200							
02.0	30			7	SS	122							
		End of borehole											

VERTICAL SCALE: 1 IN. TO 5 FT.

DOMINION SOIL INVESTIGATION LIMITED

MADE MC CHD JP

GEOTECHNICAL DATA SHEET FOR BOREHOLE 1

OUR REFERENCE NO. 2-6-L4

Client: McCarthy, Filer and Hachorn, Limited
 Location: County Road Bridge near Phillipsburg
 Township of Wilmot, County of Waterloo
 DATE: 24 June 1962
 DATUM ELEVATION: 1154.66 feet top of northeast wingwall

WATER OR BOREHOLE WAS DRIVING

DIAMETER OF BOREHOLE Bx (2-7/8")

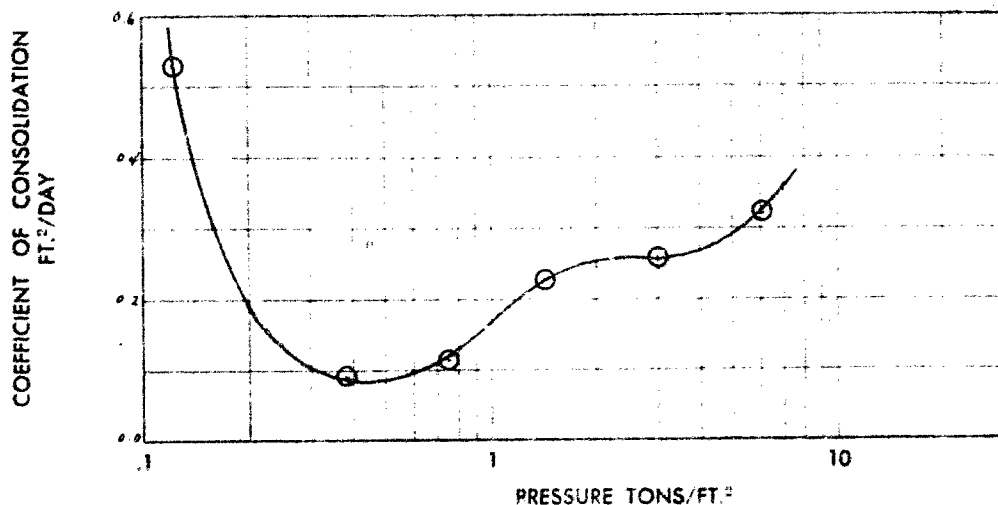
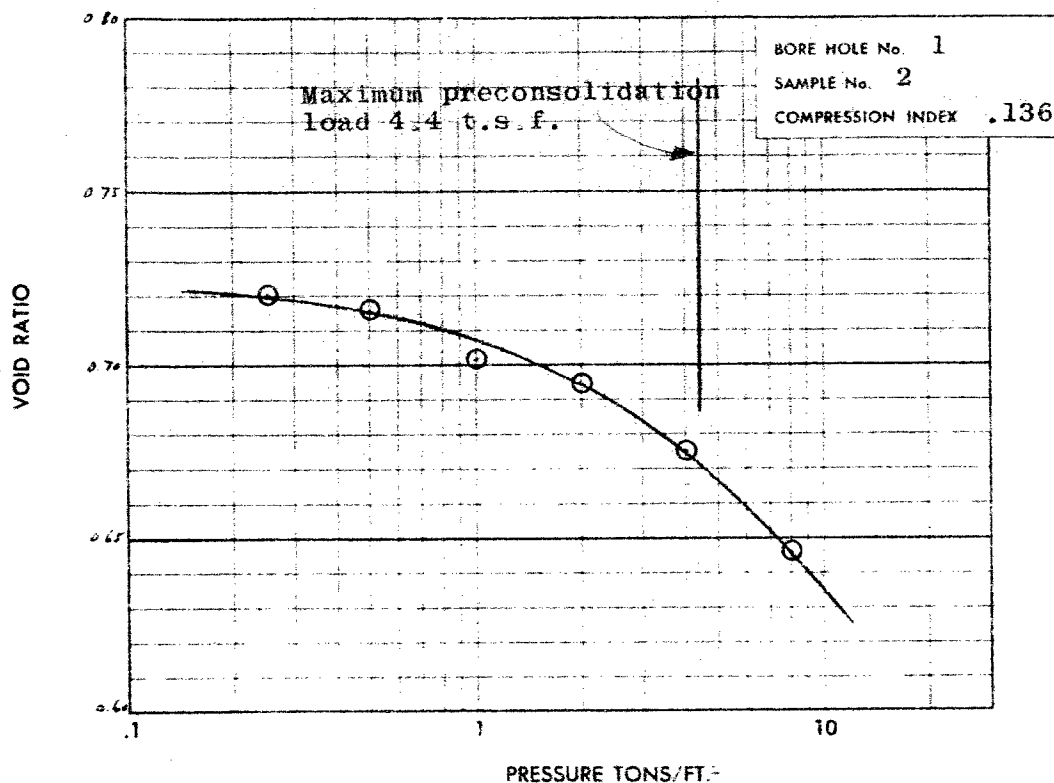
ENCLOSURE NO. 6

ELEVATION ft	DEPTH ft	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLE			PENETRATION RESISTANCE Blows per foot					CONSISTENCY		REMARKS
				NUMBER	TYPE	DEPTH ft	20	40	60	80	100	PL	W	
53.0	0	organic topsoil												
	5	Brown clayey sand, traces of organics (fill)		1	SS	6								
44.0	10	Brown clay till		2	SS	15								
39.0	15	Brown clayey sand		3	SS	4								
	20	decaying organics		4	SS	2								
31.5		Gravel												
29.5	25	Grey silt till		5	SS	94								
22.5	30	Clayey gravel and sand		6	SS	111								
19.0	35	Grey clay till		7	SS	38								
15.0	40	Gravel and sand		8	SS	52								
11.5		End of borehole												
	45													

VERTICAL SCALE: 1 IN TO 5 FT

DOMINION SOIL INVESTIGATION LIMITED

MAD: MC CHD: JP

Dominion Soil Investigation Ltd.**CONSOLIDATION TEST**

SUMMARY OF LABORATORY TEST DATA

Borehole No.	1	1	1
Sample No.	2	4	6
Depth (feet)	8.5	13	21
Elevation (feet)	1149.5	1145	1137
Liquid limit (%)	38.8	44.2	-
Plastic limit (%)	19.2	20.1	-
Liquidity index	0.18	0.21	-
Plasticity index	19.6	24.1	-
Natural moisture content (%)	22.7	25.1	21.2
Shear strength (p.s.f.)	3190	3610	2650
Void Ratio	0.71	-	-
Degree of saturation (%)	100	-	-
Elastic modulus (t.s.f.)	45	72	37
Bulk density (p.c.f.)	133	128	135.4
Compression index	0.136	-	-
Group symbol	CI	CI	-