

62 - F - 318 M

NITH RIVER

BRIDGE

NEAR WELLESLEY

BA 1383

MCCARGAR, FILER & HACHBORN LTD.

30 FRANCIS ST. SOUTH

KITCHENER

-

ONTARIO

REPORT ON
FOUNDATION INVESTIGATION
FOR
THE NITH RIVER BRIDGE
NEAR WELLESLEY, ONT.

62-F-31814

Submitted by

DOMINION SOIL INVESTIGATION LIMITED
77 Crockford Boulevard
SCARBOROUGH - ONTARIO

OUR REFERENCE: 2-2-11

March 1962

C O N T E N T S

	<u>Page</u>
INTRODUCTION	1
SUMMARY	2
I. DESCRIPTION OF SITE AND GEOLOGY ..	3
II. FIELD WORK	4
III. LABORATORY WORK	6
IV. SUBSURFACE CONDITIONS	7
V. DISCUSSION AND RECOMMENDATIONS ...	10
VI. REFERENCES	15

E N C L O S U R E S

LOCATION OF BOREHOLES AND SUBSURFACE PROFILE	Encl. #1
GEOTECHNICAL DATA SHEETS	Encls. #2 & 3
LABORATORY TEST RESULTS: CONSOLIDATION TEST CURVE	Encl. #4

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INTRODUCTION

Authorization was obtained from Messrs. McCargar, Filer and Hachborn on February 8th, 1962 to conduct a foundation investigation at the site of a proposed bridge above the Nith River, south of Wellesley, Ontario. The new structure will replace the existing narrow bridge.

The principal question is whether the existing abutments are capable of supporting the added weight of the enlarged bridge or the design and construction of new foundations will be required. This report endeavours to answer the above question from the point of view of subsoil. The evaluation of the structural suitability of the abutments should comprise the subject of another investigation.

The new bridge, similar to the existing one, will be a one-span structure; therefore the soil was tested at the two abutments only.

S U M M A R Y

- (1) A DETAILED SURVEY SHOULD BE CARRIED OUT TO REVEAL THE STRUCTURAL CONDITION AND DIMENSIONS OF THE FOUNDATIONS.
- (2) THE SUBSOIL CONDITIONS ARE DIFFERENT BELOW THE TWO ABUTMENTS. THEREFORE, THEY WILL BE TREATED SEPARATELY.

(3) SOUTH ABUTMENT

THE SUBSOIL BELOW ELEVATION 88 FT. IS A VERY DENSE, SILTY, FINE SAND WITH GRAVEL.

NORTH ABUTMENT

THE SUBSOIL BELOW ELEVATION 80 FT. IS A STIFF, LEAN CLAY WHICH CHANGES AT ELEVATION 56 FT. TO THE MATERIAL FOUND IN BOREHOLE #1.

IF THE EXISTING FOOTING IS IN A SATISFACTORY CONDITION :

- (4) THE ADDED WEIGHT CAN BE SUPPORTED ON IT. THE GROSS SOIL PRESSURE AT THE BASE LEVEL SHOULD NOT EXCEED THE VALUES TABULATED ON PAGE 11.
- (5) CONSIDERING THAT THE AVERAGE (GROSS) SOIL PRESSURE AT THE BOTTOM OF FOOTING WILL NOT EXCEED ABOUT 10,000 PSF, NO MEASURABLE SETTLEMENT WILL TAKE PLACE. THE SETTLEMENT WILL BE WITHIN ONE INCH AND WILL REQUIRE A LONGER PERIOD TO BE COMPLETE.

- (6) IF THE DIMENSIONS OF THE EXISTING FOOTINGS ARE NOT SUFFICIENT TO MAINTAIN A GROSS SOIL PRESSURE BELOW THE TABULATED ONES, THE FOUNDATIONS SHOULD BE ENLARGED.
- - - -

IF THE STRUCTURAL CONDITION IS NOT SATISFACTORY AND NEW FOUNDATIONS MUST BE CONSTRUCTED :

- (7) AN ALLOWABLE GROSS BEARING PRESSURE OF 10,000 PSF PLUS IS RECOMMENDED. THE ALLOWABLE GROSS BEARING PRESSURE SHOULD BE BASED ON THE TABULATED VALUES.

END BEARING PILES COULD BE USED FOR BOTH ABUTMENTS :

- (8) THEY WILL ATTAIN THE REQUIRED BEARING CAPACITY BETWEEN ELEVATIONS 70 AND 80 FT. THEY WILL ATTAIN THE REQUIRED BEARING CAPACITY BETWEEN ELEVATIONS 45 AND 55 FT.
- (9) NO CONSTRUCTION PROBLEMS ARE ENVISAGED. NO CONSTRUCTION PROBLEMS ARE ENVISAGED.

I. DESCRIPTION OF SITE AND GEOLOGY

A County road connects Phillipsburg and Wellesley in the Township of Wilmot, Waterloo County. The Nith River crosses this route about a mile south of Wellesley. The proposed structure will be built at this intersection.

The river flows to the south-east in an ancient spillway which has been cut into an undrumlinized till plain. At the site, the riverbed occupies the northern extreme. Therefore, whereas the northern embankment is the till plain proper, the walls of the southern one were cut into an end moraine.

The region is glaciated. In four successive cold periods of the Pleistocene, vast masses of ice moved across it, scouring the bedrock, breaking and pulverizing the dislodged pieces, over-riding and moulding the resultant debris. The underlying bedrock (limestone, sandstone and shales) deposited in the Palaeozoic era was fairly easily eroded and it was found that the glacial sediments generally were not carried far away from their place of origin. The limestone fragments found in the main material confirms this.

Spillways are the ancient drainage channels of the glaciers. Meltwaters from the ice were running in these channels, carrying and depositing the drift. Usually, the spillways are presently occupied by a creek or river.

The Nith River is such a secondary stream. It drains an area over 430 square miles and removes an equivalent of 12.3 inches of rainfall. The average volume of flow is 388 cubic foot per second, with a maximum value of 1,400 c.f.s. in flood months and 100 c.f.s. in dry periods. (Reference 3).

II. FIELD WORK

Field work was carried out during the period February 16th to February 27th, 1962 and comprised three boreholes and five dynamic cone penetration tests at the locations shown on Enclosure No. 1. The positions of the test holes were as agreed upon with the Consulting Engineers. Elevations were measured relative to the top of pavement at the east edge in the centerline of the bridge. (The permanent benchmark - i.e. a cross cut into the north-west wing-wall) could not be found owing to the extremely heavy snow at the time of the field work).

The boreholes were of 2 7/8 in. diameter. They were lined with Bx casing advanced to the required sampling depths 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 by a constant driving energy (140 pound hammer dropping 30 inches). The dynamic cone penetration test is one type of deep sounding in which the Bx 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. The former test provided disturbed samples of the substrata indicating their relative density and consistency and the latter a continuous record of soil density.

Undisturbed samples were taken with 2 in. diameter thin walled tubes forced into the subsoil in one rapid continuous movement. In some instances, the tubes were tapped into the substrata.

In addition to the split spoon and thin walled tube samples, some subsurface material removed from the withdrawn casing or brought up by the washwater was also preserved.

The in situ shear strength of the cohesive strata was measured wherever the undrained shear strength was less than 3000 pounds per square foot. A four-bladed vane 5 ins. long and 2 ins. in diameter with a blade thickness of 1/8 in. was used. The remoulded shear strength was also measured, thus providing the sensitivity index of the subsoil.

The stratification of the subsoil, sampling depths and the results of the penetration and vane tests, together with the laboratory test results, are recorded on geotechnical data sheets comprising Enclosures 2 and 3.

The heavy snow and the steep slope of the embankments made the moving rather difficult and time consuming; furthermore, they necessitated the removal of snow and earth to enable the safe set-up of equipment.

Borehole #3A was drilled later when it was found that the sample taken with a thin-walled tube sampler (Borehole #3, sample #5) has been disturbed by a gravel which cut along its entire length. Regarding the fact that this sample is of primary importance, another sample has been taken from a relocated borehole at the same depth.

III. LABORATORY WORK

The soil samples were subjected to a laboratory testing programme to determine their geotechnical properties with the most accurate methods.

Only the water content of the samples from Borehole #1 was measured. The average moisture content is about 10 percent with variations between 8.4 and 13.9 percent.

The clay samples taken from Boreholes #3 and #3A were further analyzed. The moisture contents being practically the same (the average value is 22.7 percent with results between 20.7 and 24.4 percent) only one pair of Atterberg limits (Liquid and Plastic Limits) was determined:

Liquid Limit: 31% Plastic Limit: 20.2%

Hence, the Plasticity Index = 10.8%.

The subsurface material is therefore a lean clay ("CL"). Combining the above data with the natural moisture content, the Liquidity Index is found to be 0.23.

The undrained shear strength was measured by unconfined compression tests performed on two samples. The detailed test data:

Sample Elevation	Shear Strength	Strain & Failure	E _{in}	W	Bulk Density
73	2160 psf	13.1%	32 TSF	23.2%	129 pcf
63'	1920 psf	10 %	12 TSF	21.4%	131 pcf

In addition, the sample obtained from elevation 73 ft. was subjected to a consolidation test to determine its compressibility.

The specimen data are as follows:

Bulk Density	W	Specific Gravity of Solids	Void Ratio	Compression Index
132 pcf	23.5%	(assumed) 2.75 gr/ccm	.61	.104

IV. SUBSURFACE CONDITIONS

Owing to the different geological history of the two embankments, there is a marked difference between the two subsoil conditions; therefore, the stratigraphy at the two abutments will be treated separately.

(i) South Abutment

The steep slope of the shoreline is covered with a silty, sandy, clayey topsoil in a wet, soft condition. Below it, the stratum changes abruptly into a very dense, silty fine sand with gravel. The grain-size distribution based on visual examination is as follows:

Gravel (angular)	5	percent
Sand (mainly fine)	65	" "
Silt (cohesionless)	30-20	" "
Clay (cohesive)	0-10	" "

The soil is essentially non-plastic, and slightly cemented (probably caused by lime which has been leached out from the upper strata). The brown colour suggests that the stratum is oxydized.

A common name for the above soil type is "glacial till" being a "ground-up rock debris which was carried by the glacier and deposited into a compact, unstratified mass of angular fragments of all sizes, clay, silt, sand, stones and boulders. Its outstanding characteristic is that it is non-sorted. It may consist of 99% clay particles or 99% large boulders, or any combination of these and intermediate sizes". (Goldthwaite & Flint, Ref: 7). The till at the present site is a silty, sand fill.

Based on the results obtained by others (see Ref: 7), the undrained shear strength parameters of this sedimentate assumed to be as set forth:

Cohesion	400 psf
Internal angle of friction	39°

The low moisture content (approx. 10%) suggests that if we suppose the soil is saturated with water, then the void ratio is around 0.27 (specific gravity assumed to be 2.7 g./ccm). This index alone indicates the extreme density. The bulk density should be around 145 pcf which is not unusual in the case of soils such as this.

(ii) North Abutment

Undisturbed samples were obtained from this Borehole, hence the properties of the subsoil could be directly determined (see III. Laboratory Tests).

Silty, clayey, wet, soft topsoil (probably flood deposit) occupies the first six ft. of the subsoil here. After a gravel layer has been penetrated, a stiff, lean clay stratum of low sensitivity was encountered between elevations 80 and 56 ft.

Essentially, this material is a glacial till also, consisting of material passing the No. 200 sieve only, and about 25 to 30% of the particles is smaller than 2 microns ($=0.002$ millimetres). Its shear strength was measured by a field vane and by two laboratory unconfined compression tests and it was found to be around 2,000 psf - (the field and laboratory results agree remarkably well). The average water content is 22.7%, the bulk density 130 pcf and assuming a specific gravity of 2.75 gr/ccm, the void ratio is .61. The material is only slightly compressible - the average modulus of compressibility is 135 TSF between 1 and 4 TSF loading, and it has been highly preconsolidated by the enormous weight of the ice shields having lain above for thousands of years. (The stiffness of the clay suggests its fissuredness, although they were not noticeable by the naked eye. In this event, the unconfined compression test underestimates the undrained shear strength - See Ref: 8 - which is an added margin of safety).

Owing to the high preconsolidation ratio, the pore-pressure coefficient "A" (See Ref: 5) should be around 0.2. Hence the settlements computed using the void ratio versus pressure curve as obtained in the oedometer can be reduced. The reduction factor, considering that the Z/B ratio varies between about 4 and 1, (where Z denotes the depth of incompressible stratum below the base level and B denotes the width of footing) may be taken as 0.45. This in plain terms means that only 45% of the settlements will take place because of the preconsolidation.

At elevation 56 ft., the same very dense glacial till was encountered which begins at el. 88 ft. in Borehole #1.

V. DISCUSSION AND RECOMMENDATIONS

The following procedure is recommended:

A detailed survey should be carried out to reveal the following data of the abutment foundations: their base levels and footing dimensions; and last but not least, their structural conditions. (Please note that the footings should not be exposed unless the danger of frost damage ceased to exist. The open excavation should be kept dry because soils of this type are inclined to lose much of their shear strength in contact with water).

If the structural condition of the foundations is satisfactory, the allowable bearing pressure and anticipated settlements should be analyzed as set forth.

All bearing pressures are quoted as gross pressures, that is, the beneficial effect of surcharge is included in the values.

(i) South Abutment

The ultimate bearing capacity of a footing placed at or below elevation 85 ft. was computed using the new Terzaghi theory. (The general shear failure was taken as design criterion). The footing was assumed to be 8 ft. wide and the average depth of surcharge was taken as 6 ft. Using the geotechnical properties as outlined in "IV. Subsurface Conditions", an ultimate bearing capacity of 89,400 psf is obtained. With a safety factor of 3, the allowable bearing pressure is 29,800 psf.

From the above discussion, it can be seen that the subsoil is capable of supporting the widened bridge on the existing footing. No measurable settlement is expected as the result of added loading.

(ii) North Abutment

The allowable bearing pressure was computed with the Meyerhof theory. A shear strength of 2,000 psf was used in the computations. (For this and other data of the material, see "IV Subsurface Conditions").

Footing widths of 6, 9 & 12 ft. and three assumed footing levels - elevations 77, 74 and 70 ft. were analyzed and the length of the footing was taken as 25 ft. Values tabulated below are intended to give an insight of the combined effect of footing width and depth on the bearing capacity of the foundation. The average ground level was taken as elevation 85.0 ft.

ALLOWABLE BEARING PRESSURE - (PSF)

FOUNDATION ELEVATION (Ft.)		77.0 57	74.0 54	70.0 — 50 —
FOOTING	6	5700	6250	6310
WIDTH	9	5450	5650	6160
(FT)	12	5240	5550	6100

Reference plan

Settlements were calculated on the basis of the following assumptions:

The existing bearing pressure was taken as 3,000 psf. Then the stress increment only was considered as causing settlement. (This is justified because the consolidation of the clay under the existing bridge may be considered practically complete. The stress increments were taken based on the tabulated values).

With the help of data obtained in the consolidation test and applying the correction factor by Shempton (See "IV. Subsurface Conditions"), the following settlements were arrived at in the computations.

Footing width - 6' - footing level @ el. 77 ft. - approx. $\frac{1}{2}$ "
 " " 12' " " " 70 " - " $\frac{3}{4}$ "

Assuming that the hard till layer below is practically impermeable, the rate of settlement based on the coefficient of consolidation is as follows:

	F O O T I N G L E V E L		
	77.0'	74.0'	70.0'
TIME NEEDED FOR 50% CONSOLIDATION (DAYS)	340	256	154
TIME NEEDED FOR 90% CONSOLIDATION (DAYS)	1500	1100	666

(The differences between allowable bearing pressures were not considered and an average coefficient of consolidation = $0.25 \text{ ft.}^2/\text{day}$ was used).

If the footing dimensions and elevation are known, the corresponding allowable bearing pressure should be taken from the table. Should the combined live and dead load cause a soil pressure greater than the allowable, the enlargement of the footing would be required. The new and old foundation sections should be rigidly connected to each other.

. . . .

The possibility is not excluded that piles will support more economically the new structure.

Consideration should be given to this possibility also and the following is recommended:

(i) South Abutment

Any type of pile would penetrate through the road fill and will attain a satisfactory bearing capacity between elevations 70 and 80 ft.

(ii) North Abutment

Here the piles should penetrate through the lean clay stratum; hence non-displacement type piles (e.g. steel H bearing piles) should be used. They will attain the required bearing capacity between elevations 45 and 55 ft.

In both cases, the bearing capacity should be checked and computed with the Hiley formula.


Construction Procedure

The excavation should be kept dry by enclosing it with steel sheet piles. This wall will withhold the surface or river waters together with the seeping water through the upper more permeable strata. The tills of both abutments are practically impermeable and this assures the watertightness of the bottom. Any water still getting into the excavation can then be removed by pumping.

The backfill should be drained by weeping tiles.

Yours very truly,

DOMINION SOIL INVESTIGATION LIMITED


L. R. Szalatnay, P.Eng.,
Senior Soils Engineer.

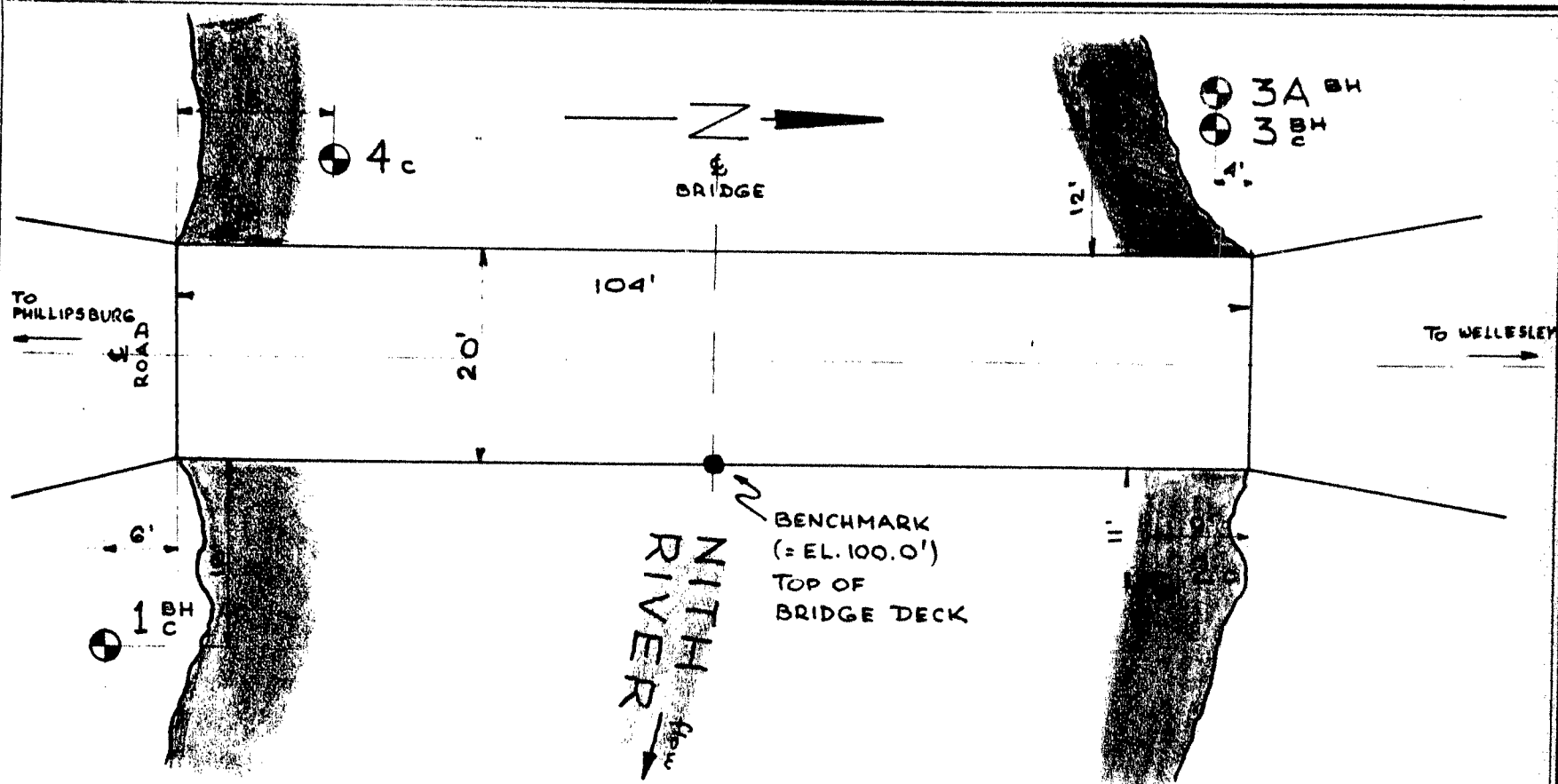
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Encls.

VI. REFERENCES

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- (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) Alapozasok (Foundation Engineering - in Hungarian)- by Ch. Szechy Budapest, 1957.
- (5) 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.).
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- (8) Proceedings of the Conference on the Measurement of Shear Strength of Soils in Relation to Practice. Paper presented by Messrs. A. W. Skempton and A.W. Bishop : The Measurement of Shear Strength of Soils - London, June 1950 (See Geotechnique, Vol. II).

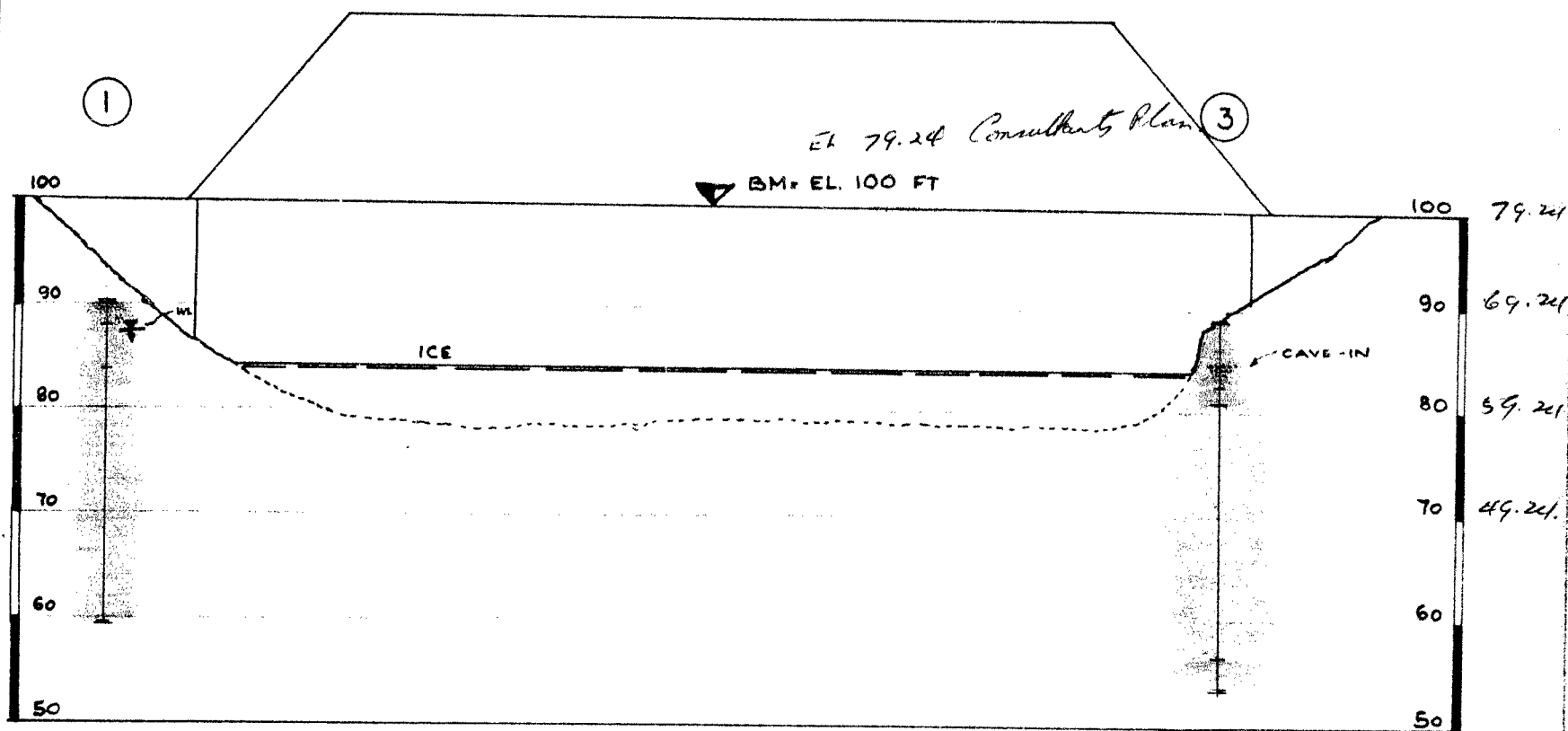
E n c l o s u r e s



LOCATION OF TEST HOLES

SCALE: 1" TO 15'

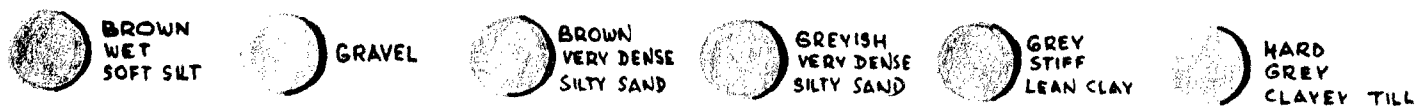
[BH = BOREHOLE
C = DYN. CONE PEN. TEST]



SUBSURFACE PROFILE

SCALE: 1" TO 15'

LEGEND:

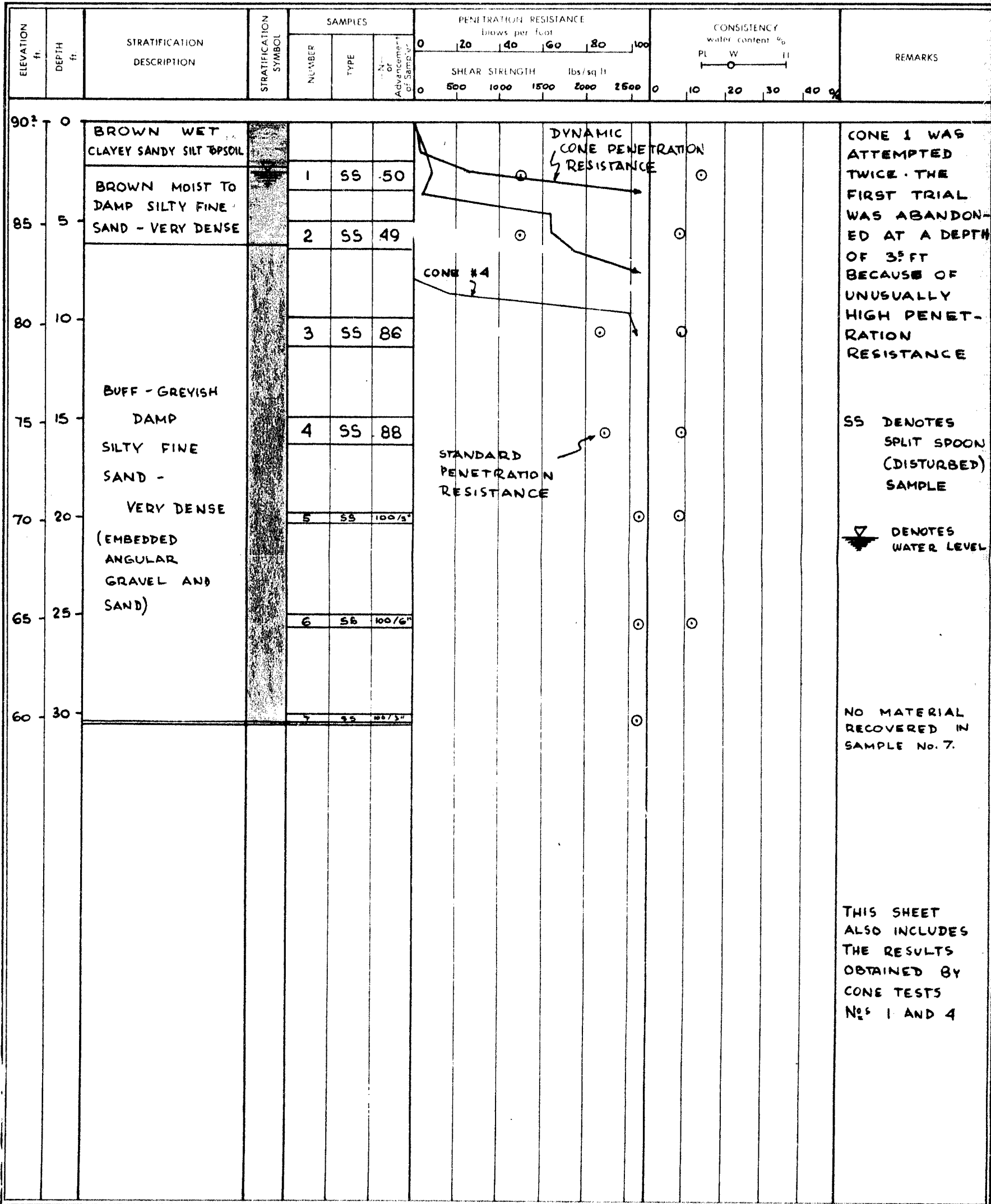


OUR REFERENCE NO 2-2-11 GEOTECHNICAL DATA SHEET FOR BOREHOLE . . 1 . . .

CLIENT: M^cCARGAR · FILER & HACHBORN LTD.
 PROJECT: NITH RIVER BRIDGE
 LOCATION: WELLESLEY, ONT.
 DATUM ELEVATION: 90.2 ft

METHOD OF BORING WASHBORING
 DIAMETER OF BOREHOLE 2 7/8"
 DATE: MARCH, 9, 1962.

ENCLOSURE NO 2



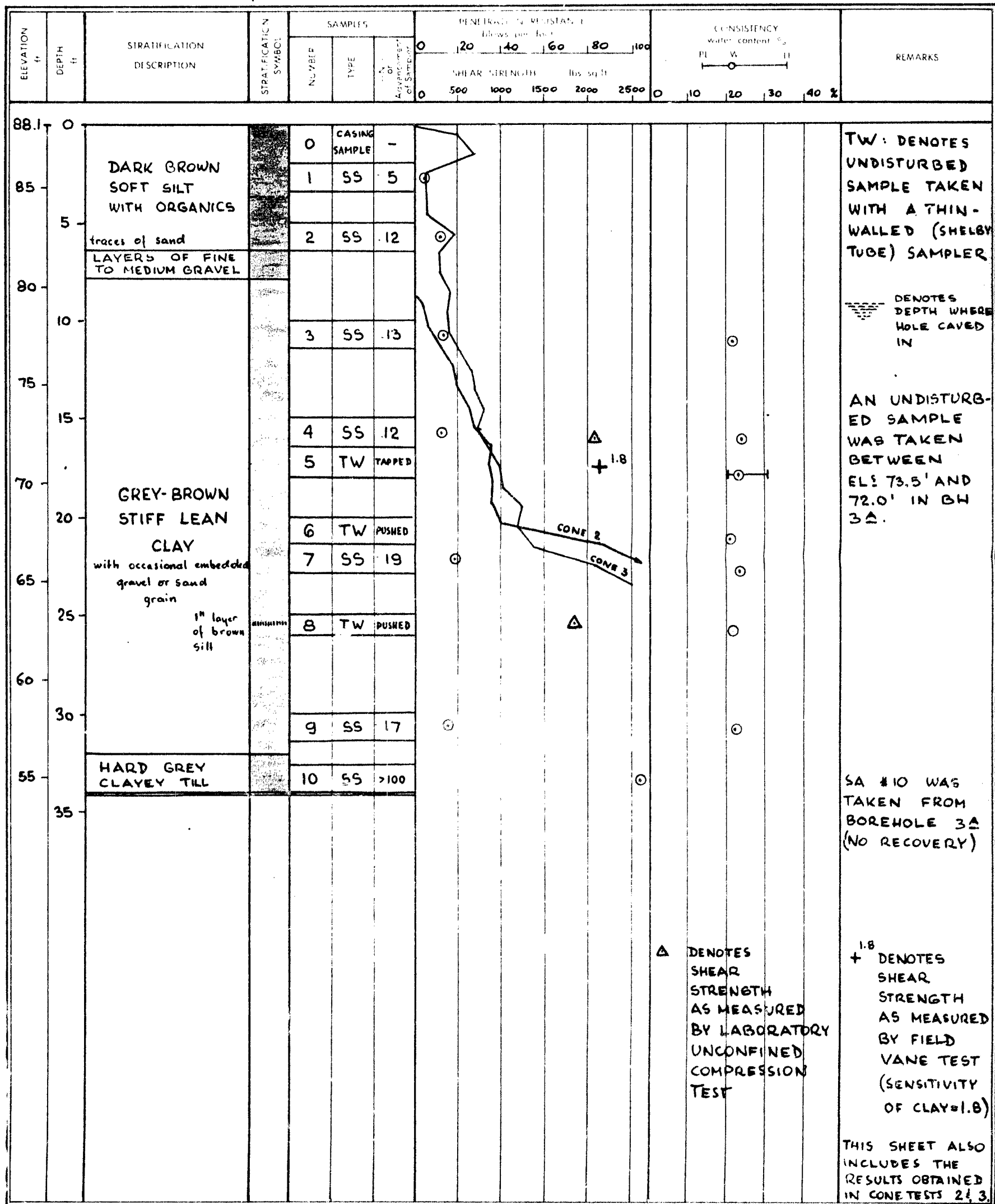
GEOTECHNICAL DATA SHEET FOR BOREHOLE 3 & 3^A

DRP REFERENCE NO. 2-2-II

CLIENT: MCGARGAR, FILER & HACHBORN LTD.
PROJECT: NITH RIVER BRIDGE
LOCATION: WELLESLEY, ONT.
DATUM ELEVATION: 88.1 ft

METHOD: M. W. W. WASHBORING
DIAMETER OF BOREHOLE: 2 7/8"
DATE: MARCH 9, 1962.

ENCLOSURE NO. 3



VERTICAL SCALE: 1 IN TO 5 FT

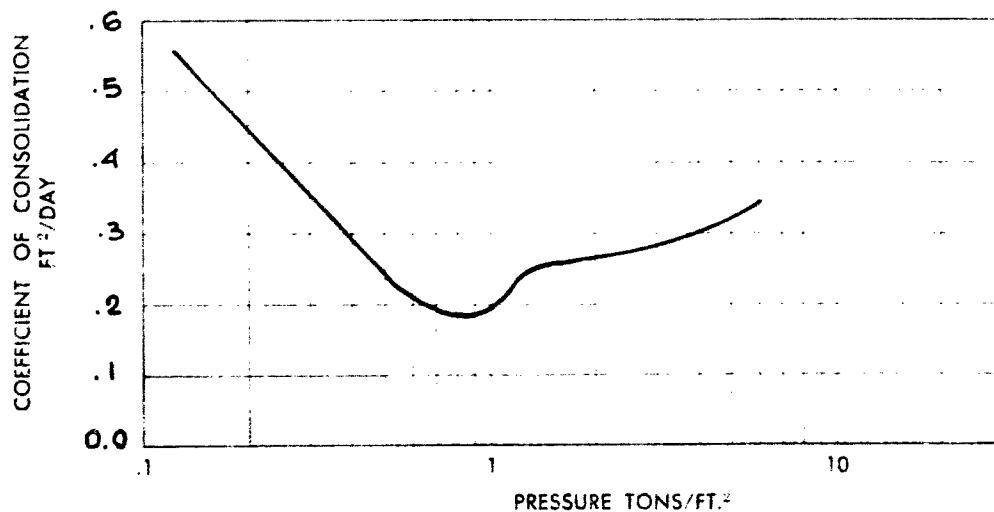
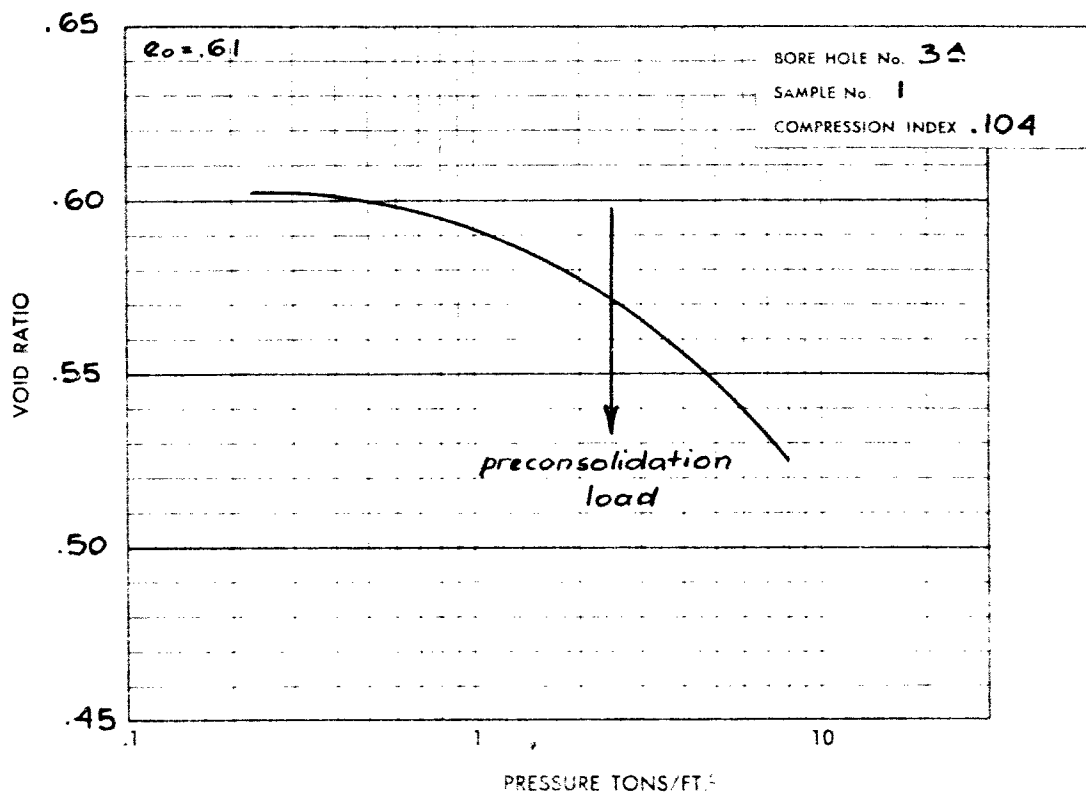
DOMINION SOIL INVESTIGATION LIMITED

MADE L CHD *Salisbury*

Dominion Soil Investigation Ltd.

CONSOLIDATION TEST

The clay is
highly preconsolidated.



Mr. K. Kleinsteinber

April 19, 1962.

Mun. Br. Liaison Engr.,

Waterloo County Bridge,
Wilmot Twp. Lot 18 Con. IV B,
With River South of Wellesley,
Mun. Dist. 3, WO 366-62-1 EW 639.

J. D. Harris

The watershed area is 165 square miles, and the flood rise approximately 12', to elevation 1176. Until the north approach was raised recently, some flooding of the road occurred during severe floods, thereby by-passing the existing bridge.

The measured depth of scour at the bridge is 7', which according to the original drawing no. 2240, dated 1924, is roughly level with the bottom of footings. However, the true scour near midspan may be greater, and may be concealed by re-deposited material.

The soil report shows that the subsoil at the north bank is stiff clay, but at the south bank is dense silty sand. Thus it appears that scour could be a problem at the new south pier and abutment but not at the north abutment. The report indicates that steel sheet piles will probably be required for dewatering, and also that piles may be economic.

Summary of Recommendations

1. Width of waterway 120' approximately. If a three span spill-through bridge is used, the average width of 120' should be measured at elevation 1170.
2. Angle of skew - could probably be 0°, but a slight left hand skew would reduce erosion of the downstream south bank.
3. Elevation of lowest point of soffit: 1178 to 1179.0.
4. Scour protection: if piles are used, footings could be at a nominal depth. If spread footings are used, steel sheeting should be left in place permanently and designed on the basis of possible scour to elev. 1153.
5. Fill should be rip-rapped up to elevation 1177.
6. The sag in the north approach road should not be eliminated.

JDH:go

J. D. Harris,
for B. Wilkie,
Bridge Hydrology Engineer.