

#63-F-284 M

NITH RIVER

BRIDGE

LOT 16, CON II/III

BLOCK 'B'

Mr. K. L. Kleinstelber,
Mun. Bridge Liaison Engr.,
Bridge Division.

Attn: Mr. G.C.E. Burkhardt

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

April 26, 1963

Township of Wilmot,
Proposed Nith River Bridge,
Lot 16, Con. II/III Block B,
County of Waterloo,
Structure Site No. 34-68,
Bridge Office File No. BA 1627

We have reviewed the above-mentioned report submitted by Dominion Soil Investigation, Ltd., and also the bridge drawing prepared by McCargar, Filer and Hachborn, Ltd., Consulting Engineers, and herewith submit our comments for your consideration.

The structure is founded on spread footings as recommended by the Soil Consultant. The elevation of the footing bottom of the west abutment is 1164.0. A distance of 10 ft. is allowed from the face of the footing to the slope face.

High water level is indicated on the drawing as 1164.0, which coincides with the abutment footing elevation.

Because scour could affect very adversely, the west abutment, we would recommend that the footings be placed three feet deeper - i.e., at elevation 1161.0, allowing thus, a distance of 14 ft. from the footing to the slope.

AGS/MdeF
Encls. (2)

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

P.S. -- Attached, we are returning to you
the bridge drawings, as requested.

cc: Foundations Office
Gen. Files

MEMORANDUM

TO: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

FROM: G. C. E. Burkhardt

DATE: April 25, 1963.

OUR FILE REF.

IN REPLY TO

SUBJECT: Township of Wilmot
Proposed Nith River Bridge
Lot 16, Con. II/III Block B,
County of Waterloo
Structure Site No. 34-68
Our File No. BA 1627


Attached please find one copy of the Foundation Report, by Dominion Soil Investigation Limited, and one copy of the Preliminary Plans for your comments.

As you will notice on the Plan #2 the west abutment has been founded well above the other footings. Could you be so kind and give this matter your special attention.

We hope to approve the preliminary design not later than May 3rd and we would appreciate it very much if we could have your comments within one week.

Since we do not have enough copies of the plans, we would like to have the plans back, which we are forwarding to you today.

GCEB/et


G. C. E. Burkhardt,
for K. L. Kleinsteinber,
Mun. Bridge Liaison Engineer.

BA 1627

STRUCTURE SITE No. 34-68

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

Report on
SOIL INVESTIGATION
for
NITH RIVER BRIDGE
TOWNSHIP OF WILMOT
COUNTY OF WATERLOO

63-0284M

by
DOMINION SOIL INVESTIGATION LIMITED
363 Queens Avenue
LONDON ONTARIO

Reference No. 3-2-L5
February
1963

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INTRODUCTION

In accordance with a letter of authorization from Messrs. McCargar, Filer and Hachborn dated 1st February 1963, a soil investigation has been carried out at a site in the Township of Wilmot where it is proposed to replace an existing Township road bridge with a new structure.

The requirements of the work including the approximate location of the boreholes were discussed with Mr. E.G. Hachborn on the 11th of February 1963. The new structure will probably have a span arrangement of 45 - 70 - 45 feet and will be situated in approximately the same position as the existing single-span bridge. It was suggested that the piers of the new bridge would be carried on spread footings, if soil conditions permit, at a depth of 5 to 7 feet below the existing level of the river bed.

The purpose of this investigation was to reveal the sub-surface conditions and to determine the necessary soil properties for the design and construction of the new bridge.

I PHYSIOGRAPHY

The site is located approximately 2 miles to the southeast of the Town of Wellesley where a gravel township road crosses the River Nith. The area is part of the Stratford Till Plain which extends westwards as far as London in the south and Blyth in the north. The plain, composed chiefly of ground moraine, has a slightly undulating surface except where it is intersected by the ridges of terminal moraines or eroded by the spillway valleys which form the principal drainage courses. The latter have been formed by the melting waters of the Wisconsin glacier whose recession marked the end of the last ice era in the region. These valleys are now generally occupied by lesser streams such as the Nith.

At the site of the proposed new structure the U-shaped spillway is approximately 1/2 a mile wide. The Nith has cut a shallow meandering path across the relatively flat bottom and, in the vicinity of the bridge, its course has been controlled by the steep west slope of the valley.

At the east end of the existing bridge an artificial embankment has been formed to raise the road above the flood plain.

II FIELD WORK

Field work was carried out during the period 13th to 21st February 1963, and consisted of 4 boreholes at the locations shown on enclosure 2. Dynamic cone penetration tests were performed adjacent to each borehole. Two additional such tests were made, one from the bottom of borehole 1 and one at a 5th location (cone 5) at the east end of the bridge. The holes were advanced by washboring and lined with Bx (3-inch) casing. Diamond drilling techniques were employed to advance boreholes 2 and 3 into a stratum of dense granular material.

Standard Penetration tests were performed at frequent intervals of depth. Vane shear tests were attempted at two points in borehole 4, but in general the soil is too stiff to yield results in this type of test. One sample in borehole 4 was recovered in a Shelby tube. It was necessary to tap the tube into the soil, thus creating disturbance to the

sample and damage to the tube. The strata are generally too stiff for the recovery of undisturbed samples.

The results of the field tests are recorded on geotechnical data sheets comprising enclosures 3 to 6. Elevations have been referred to a local benchmark - "nail on south face of hydro pole, 3 feet \pm above grade, on north side of road at station 0 + 69 east, El. 1163.60."

III SUBSURFACE CONDITIONS

Details of the stratification at each borehole are shown on the data sheets and a general picture of the subsurface conditions is given by the profile on enclosure 2.

The deposit of loose clayey silty sand fill encountered in borehole 1 is apparently related to the construction of the existing bridge or road embankment. A natural deposit of fine grey sand containing traces of organic matter between 11 feet and 13 feet is presumably the sediment of the original glacial river or of the present river in an earlier position. Its meandering course will tend to shift it in a westerly direction at this point as it erodes the west bank of the spillway.

A comparison of the penetration graph for cone 5 with the stratification at borehole 1 suggests almost identical conditions at these two locations. Cone 1 has apparently encountered a boulder or other obstruction at a depth of 10 feet, and serves only as an indication of the presence of such obstructions which could affect, for example, piling operations.

The very stiff to hard grey silty clay which comprises the ground moraine in the region was encountered at approximately the same elevation in boreholes 1, 2 and 3, and was explored from a higher level at borehole 4. The material could be described as a "till", in that it is a dense, heavily pre-consolidated glacial deposit, with a moisture content near the plastic limit. It contains little or no gravel.

Boreholes 1, 2 and 3 reached the underside of the clay layer and were terminated in a very dense deposit of granular material. This stratum contains all sizes of granular particles from fine sand to cobbles (3" to 8") from which short lengths of core were recovered.

IV FOUNDATIONS

In considering the best type of foundation for the structure it is noted that:-

- (i) the grey silty clay stratum offers a high bearing capacity for spread footings, and will be reasonably insensitive to disturbance during construction.
- (ii) in the vicinity of borehole 1 the highest permissible level for footings is about El. 1146, which is 14.6 feet below the present grade and 6 feet below the prevailing water table at the time of this investigation.
- (iii) much of the fill at the east end of the present bridge will probably be moved during construction of the new bridge or the new road embankment. To excavate to El. 1146 may, therefore, be quite practicable.
- (iv) the results of cone 1 indicate that cobble or boulder-size obstructions may affect the driving of piles near El. 1150.
- (v) If piles are used at only one of the four support points i.e. the east abutment, they will probably be timber piles because of cost.

The foregoing observations lead to the conclusion that the use of spread footings will be quite practical and economical for the central piers and west abutment. For the east abutment either a spread footing or a piled foundation is practicable, although if timber piles are used some damage may be caused by obstructions around El. 1150. As a safeguard the piles should be fitted with steel shoes. Cost should be the deciding factor for the choice of foundation at this location.

The following gross soil pressures are recommended for the design of spread footings:

Borehole No.	Elevation (feet)	Maximum gross soil pressure (p.s.f.)
1	1146 to 1142	5000
	1142 to 1132	7000
2	1143 to 1132	7000
3	1143 to 1132	7000
4	1165 to 1135	7000

The above values allow for a safety factor of 3 or more against failure of the soil. Provided the footings are poured on a clean undisturbed grade, the total settlement is not expected to exceed *one inch*.

The stability of the footing for the west abutment in relation to the adjacent slope should be checked once a tentative design arrangement has been made. Provided that the footing is far enough from the face of the slope to allow the normal stress pattern to develop without appreciable distortion (say at 60° to the vertical) the footing should be quite stable. The effect of erosion on the future stability of the footing should also be considered, but this question merits special hydrological study beyond the scope of this investigation.

Timber piles of 12-inch nominal diameter driven to El. 1140 have a *theoretical safe working load* of 15 tons (cohesion 5000 p.s.f., adhesion 1300 p.s.f.). For deeper penetration the working load increases by approximately 1 ton per foot. Irrespective of these theoretical values, the piles should be driven to a satisfactory set in accordance with the Hiley formula. From the consistently high penetration values in the gravel and sand layer at boreholes 2 and 3 it is deduced that steel pipe piles with a working load of 40 tons would reach a set within 3 to 4 feet below the top of this stratum.

Whatever type of foundation is finally chosen, the settlement at all 4 support points should be small, so that a continuous span structure will perform satisfactorily.

V CONSTRUCTION

The footings for the central piers and (if a footing is used) the east abutment, will be located some distance below the normal water table. Dewatering can be carried out by enclosing the excavation within sheet piles and removing seepage by pumping.

To preserve the hydraulic stability of the bottom of the excavation, the piles should be driven to such a depth that the weight of soil between the footing grade and the pile tips is greater than the excess water pressure. For this purpose the density of the clay may be taken as 130 p.c.f.

A thin concrete blanket spread on the floor of excavation will help to prevent disturbance by construction personnel or equipment.

Timber piles, if used, should be fitted with steel shoes. Constant supervision of the pile driving is recommended to ensure that in the event of damage the piles are replaced.

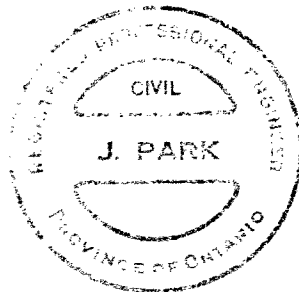
VI SUMMARY

1. The site is located to one side of a U-shaped spillway which has cut through a deep stratum of very stiff to hard clay. Below the clay layer at El. 1130±, there is a very dense deposit of cobbles, gravel and sand. The road embankment has been made up with sandy fill at the east end of the existing bridge.
2. The central piers and west abutment of the new bridge should be supported in the clay stratum on spread footings designed in accordance with the recommended soil pressures tabulated on page 4.
3. The east abutment can be supported on a footing or on piles. Both solutions are technically practicable and the choice will be governed by cost.
4. If timber piles are used at the east abutment, some of the piles may suffer damage from boulders or cobbles in the fill.
5. Settlements should be small, so that a continuous span structure will perform satisfactorily.

VII 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. 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.
4. Terzaghi and Peck: Soil Mechanics in Engineering Practice, John Wiley and Sons, New York, 1948.
5. The Application of Theories of Elasticity and Plasticity to Foundation Problems, Leo Jurgenson, Sc.D., Boston Society of Civil Engineers, May, 1934.
6. The Ultimate Bearing Capacity of Foundations by G.G. Meyerhof, Geotechnique, Vol. II, 1950 and 1951.
7. M.J. Tomlinson, The Adhesion of Piles Driven in Clay Soils, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, 1957.



DOMINION SOIL INVESTIGATION LIMITED

A handwritten signature in cursive script, appearing to read "James Park".

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

Encl.
JP/mc

LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE.

SOIL COMPONENTS AND GROUND WATER CONDITIONS.

BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY	ORGANICS	BEDROCK	GROUND WATER LEVEL	DEPTH OF CAVE-IN
$\phi > 8"$	3"	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.

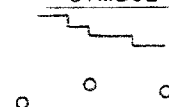
DYNAMIC PENETRATION RESISTANCE : to drive a 2" ϕ , 60° cone attached to the end of the drilling rods into the ground, expressed in blows per foot

STANDARD PENETRATION RESISTANCE, -N- : to drive a 2" outside dia, split spoon sampler 1 foot into the ground, expressed in blows per foot.

EXTRAPOLATED -N- VALUE

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

SYMBOL:



322

SOIL PROPERTIES.

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

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

k Coeff. of permeability
C Shear strength
 ϕ Angle of int. friction
C' Cohesion
 ϕ' Angle of int. friction

in terms of total stress
in terms of effective stress

UNDRAINED SHEAR STRENGTH.

- DERIVED FROM -

TRIAXIAL COMPRESSION TEST
UNCONFINED TEST

LABORATORY VANE TEST
FIELD

POCKET PENETROMETER TEST

Strain at failure is represented by direction of stem
20%
15%
10%
5%

St = 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 %

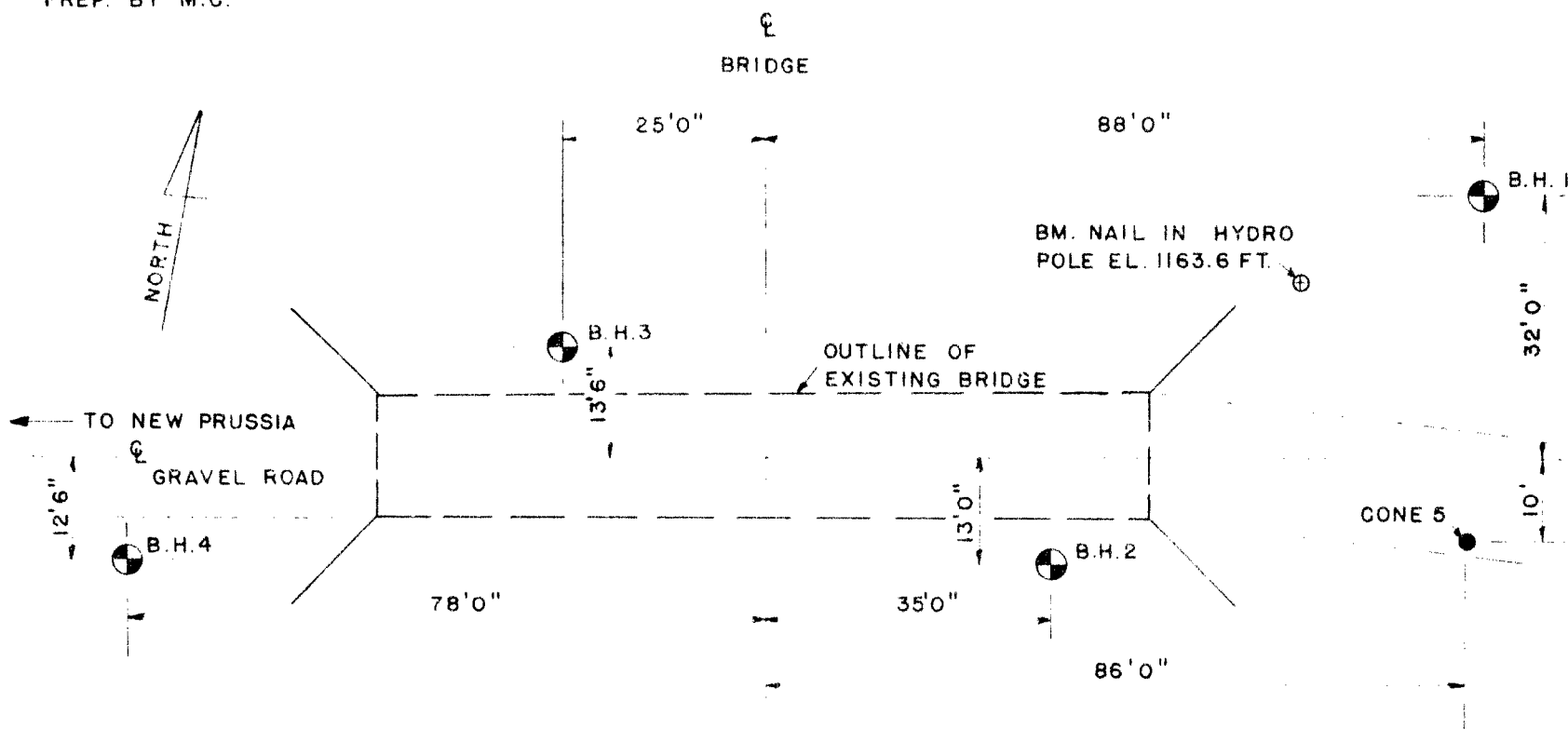
COHESIVE SOILS :

C lbs/sq ft

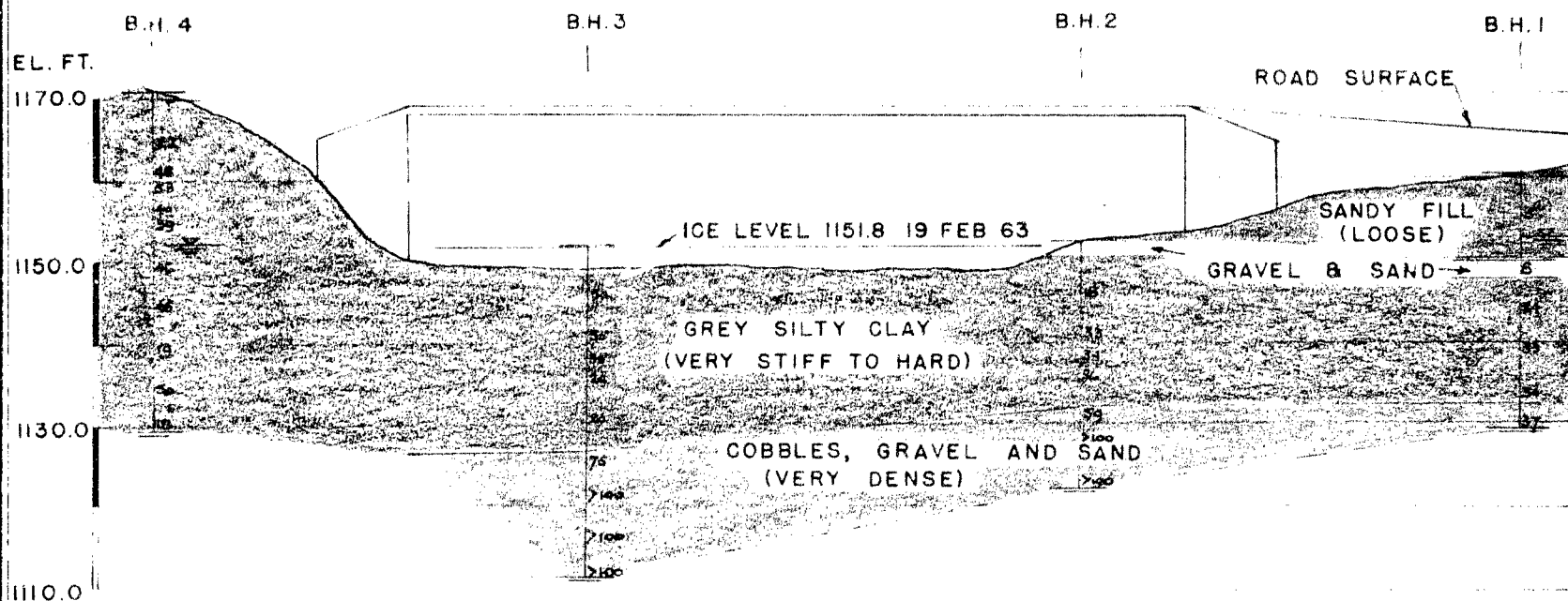
Very soft less than 250
Soft 250 - 500
Firm 500 - 1000
Stiff 1000 - 2000
Very stiff 2000 - 4000
Hard over 4000

JOB NO. 3-2-L5
 PREP. BY M.C.

ENCLOSURE 2



LOCATION OF BOREHOLES
 SCALE - 1 INCH TO 20 FEET



SUBSURFACE PROFILE
 SCALE - 1 INCH TO 20 FEET

NOTE - FIGURES AT BOREHOLES
 DENOTE STANDARD PENETRATION
 RESISTANCE.

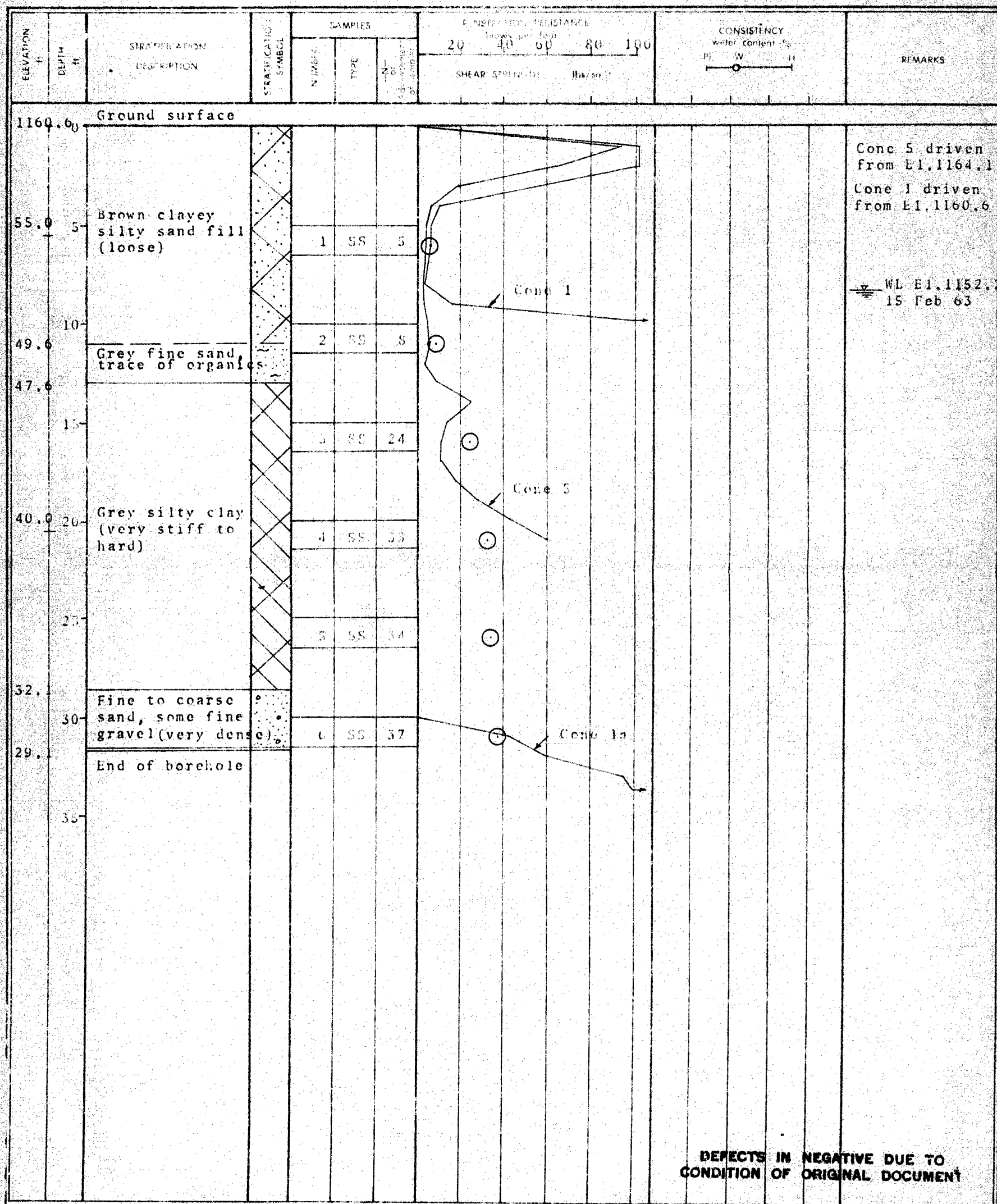
GEOTECHNICAL DATA SHEET FOR BOREHOLE 1

FORM REFERENCE NO. 5-2-15

CLIENT: Messrs. McCargar, Filer & Bachern
 ADDRESS: Nith River Bridge
 Borehole LOCATION: See enclosure 2
 DATUM ELEVATION: 1163.6 feet (nail on hydro pole, see enclosure 2)

DATE OF LOGGING: 15/14 Feb 63
 DIAMETER OF BOREHOLE: 3X (3-inch)
 DATE: 15/14 Feb 63

ENCLOSURE NO. 3



DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT

OUR REFERENCE NO 3-2-15

METHOD OF BORING TESTING

DIAMETER OF FOREHOLE. 3" (3-1/4 inch)

ENCLOSURE NO. 4

DATE: 14/15 Feb 63

DATUM ELEVATION: 1163.6 feet (nail on hydro pole, see enclosure 2)

VERTICAL SCALE: 1 IN. TO 5 FT.

GEOTECHNICAL DATA SHEET FOR BOREHOLE

OUR REFERENCE NO. 3-2-15

CLIENT Messrs. McCargar, Piler & Bachborn

METHOD OF BORING Washboring

ENCLOSURE NO. 5

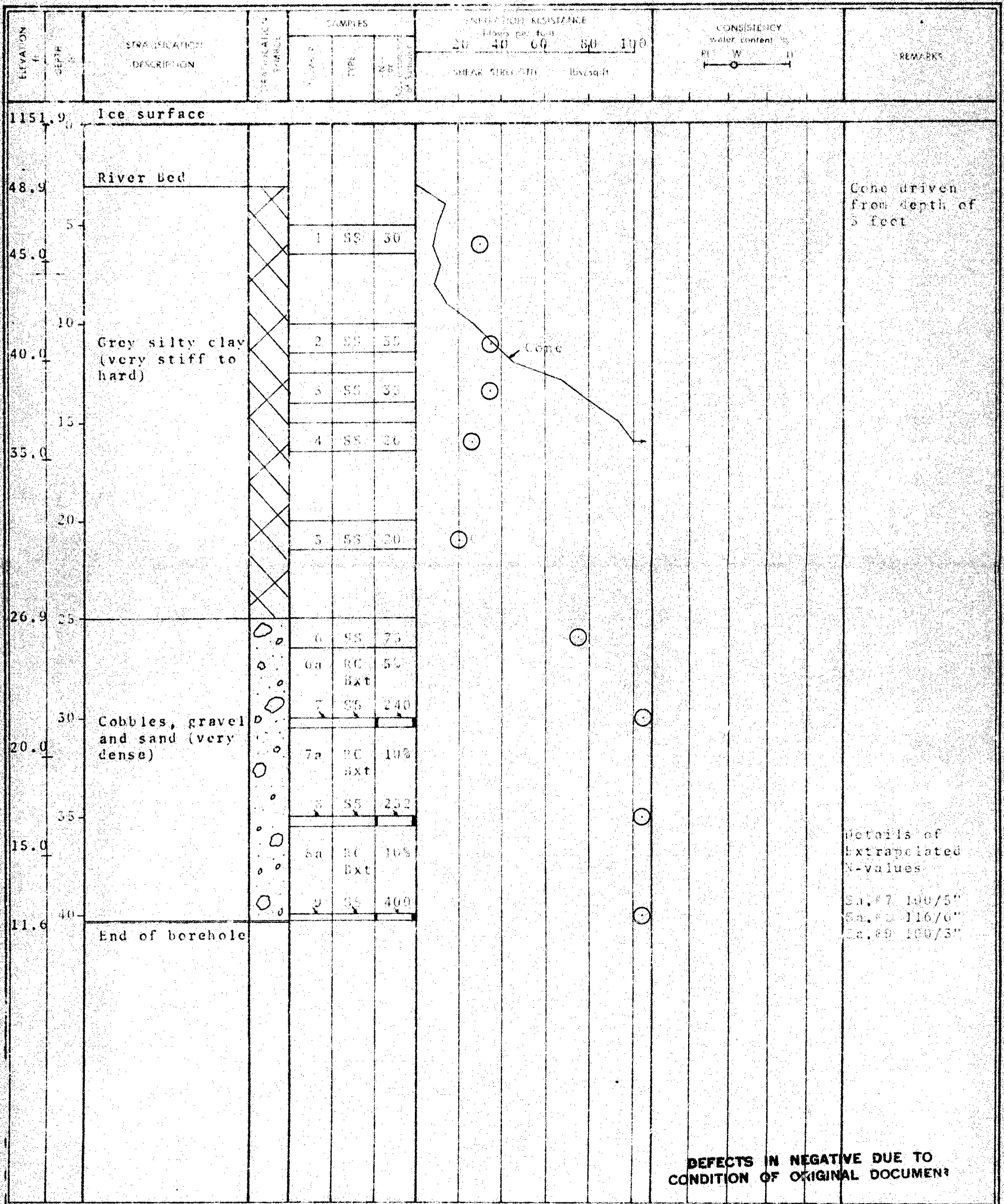
PROJECT With River Bridge

DIAMETER OF BOREHOLE 8" (3-inch)

DATE 18 Feb 63

Borehole LOCATION See enclosure 2

BATUM, SILLARON 1163.6 feet (nail on hydro pole, see enclosure 2)



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

GEOTECHNICAL DATA SHEET FOR BOREHOLE

OUR REFERENCE NO. A-2-15

CLIENT Messrs. McCargar, Eiler & MacLean

PROJECT With River Bridge

BOREHOLE LOCATION See enclosure 2

BATHY ELEVATION 1163.6 feet (nail on hydro pole, see enclosure 2)

METHOD OF BORING SPARK TEST

DIAMETER OF BOREHOLE 3/4" (1-1/8")

DATE 10/20/64

ENCLOSURE NO. 0

ELEVATION ft.	DEPTH ft.	STRATIFICATION DESCRIPTION	STRATIFICATION SYMBOL	SAMPLES			PENETRATION RESISTANCE blows per foot				CONSISTENCY water content %		REMARKS
				NUMBER	TYPE	NO. OF ASTM ASSIGNMENT	20	40	60	80	100	PL	
1171.0		Ground surface.					1000	2000	3000	4000	5000		
		organics											
65.0	5			1	SS	20							
				2	SS	45							
59.0	15	brown grey		3	SS	50							
				4	SS	40							
55.0	15	Silty clay (very stiff to hard)		5	SS	52							
50.0	20			6	SS	46							
45.0	25	trace of fine gravel		7	SS	40							
40.0	30			8	SS	15							
				9	SS	15							
35.0	35			10	SS	50							
29.5	42	End of borehole		11	SS	15							

WL El. 1154.7
1 hour after completion of hole

Vane pushed to 42'0" - encountered shear resistance in excess of 6000 p.s.f.

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT