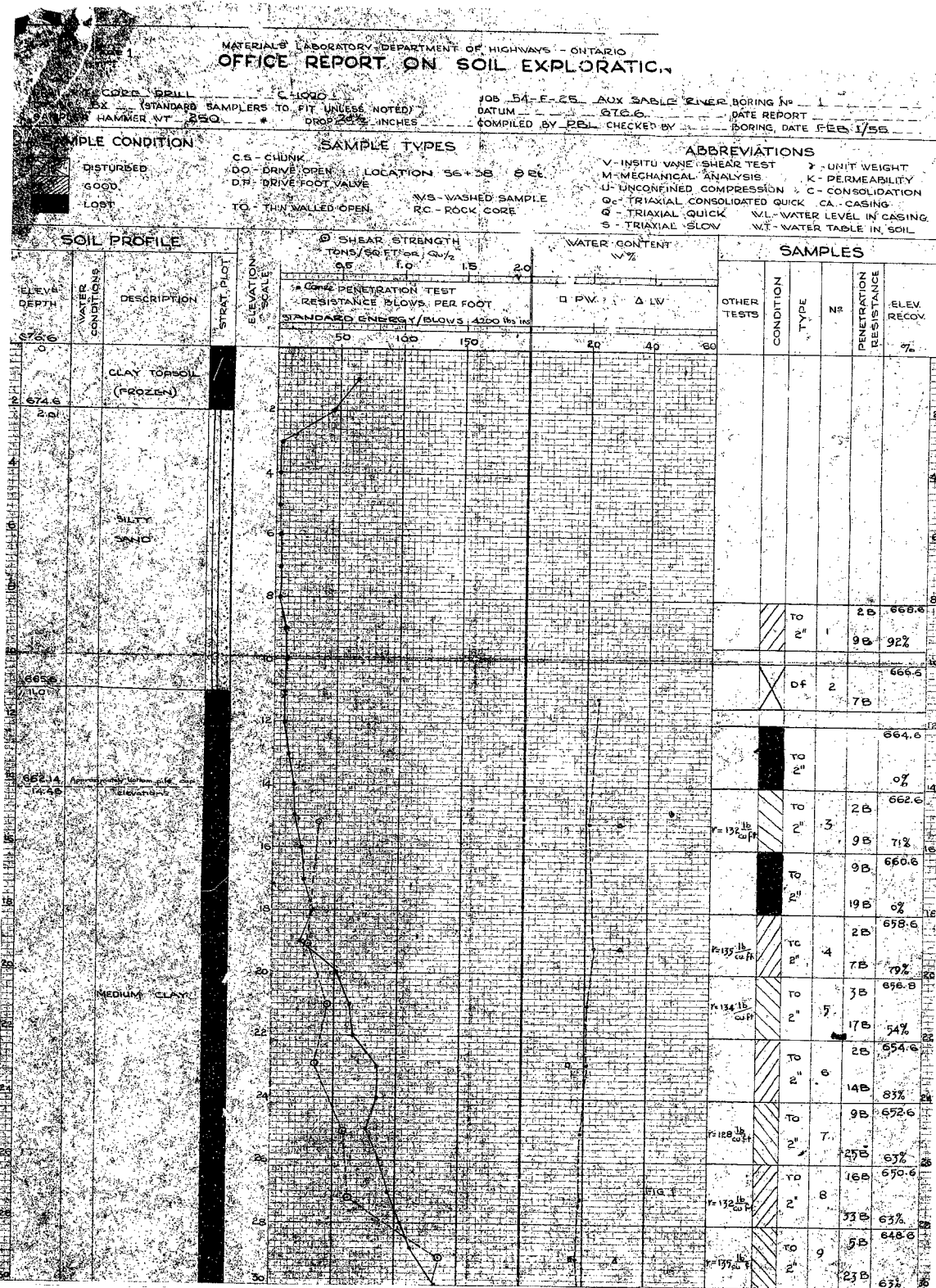
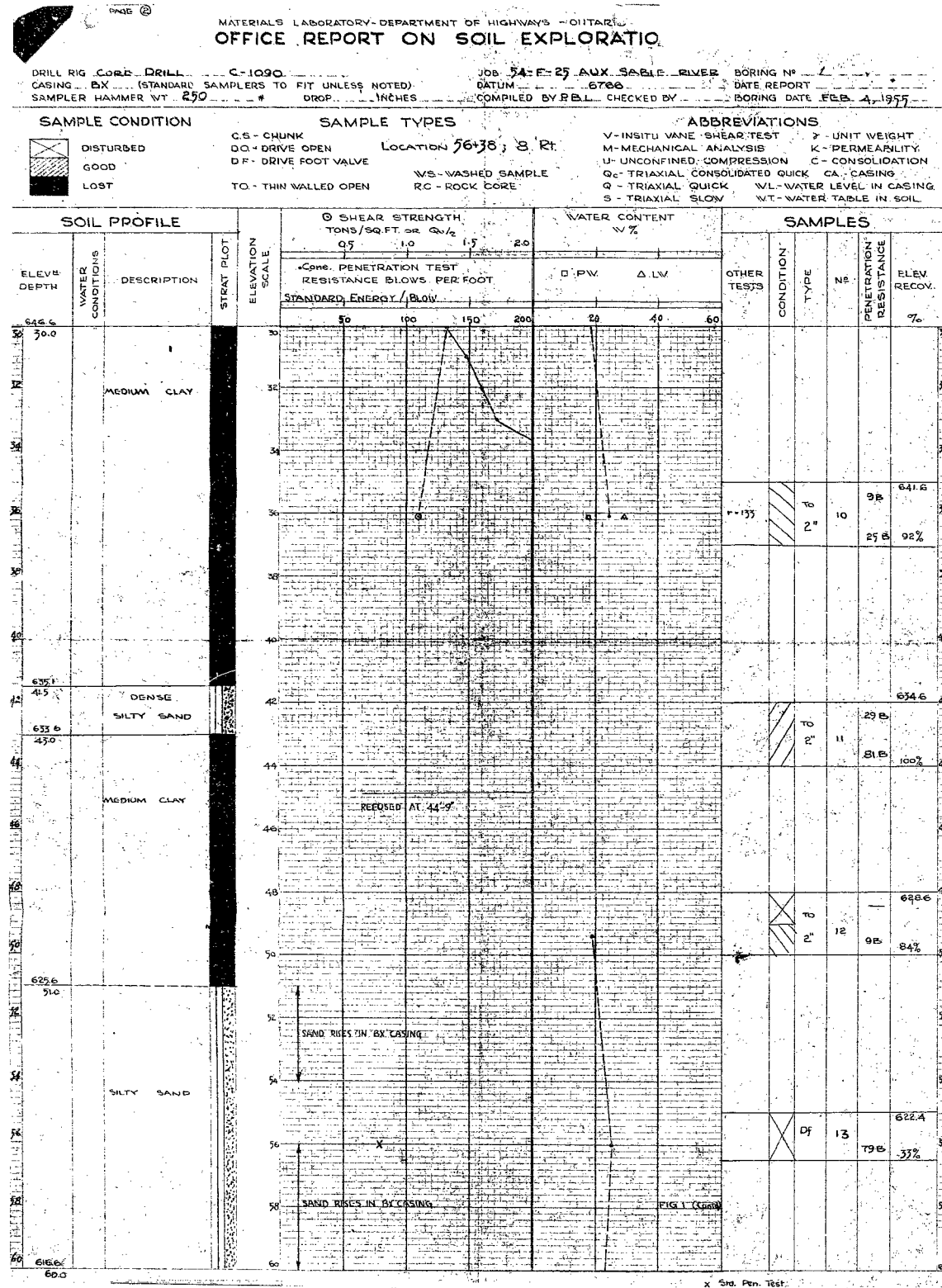


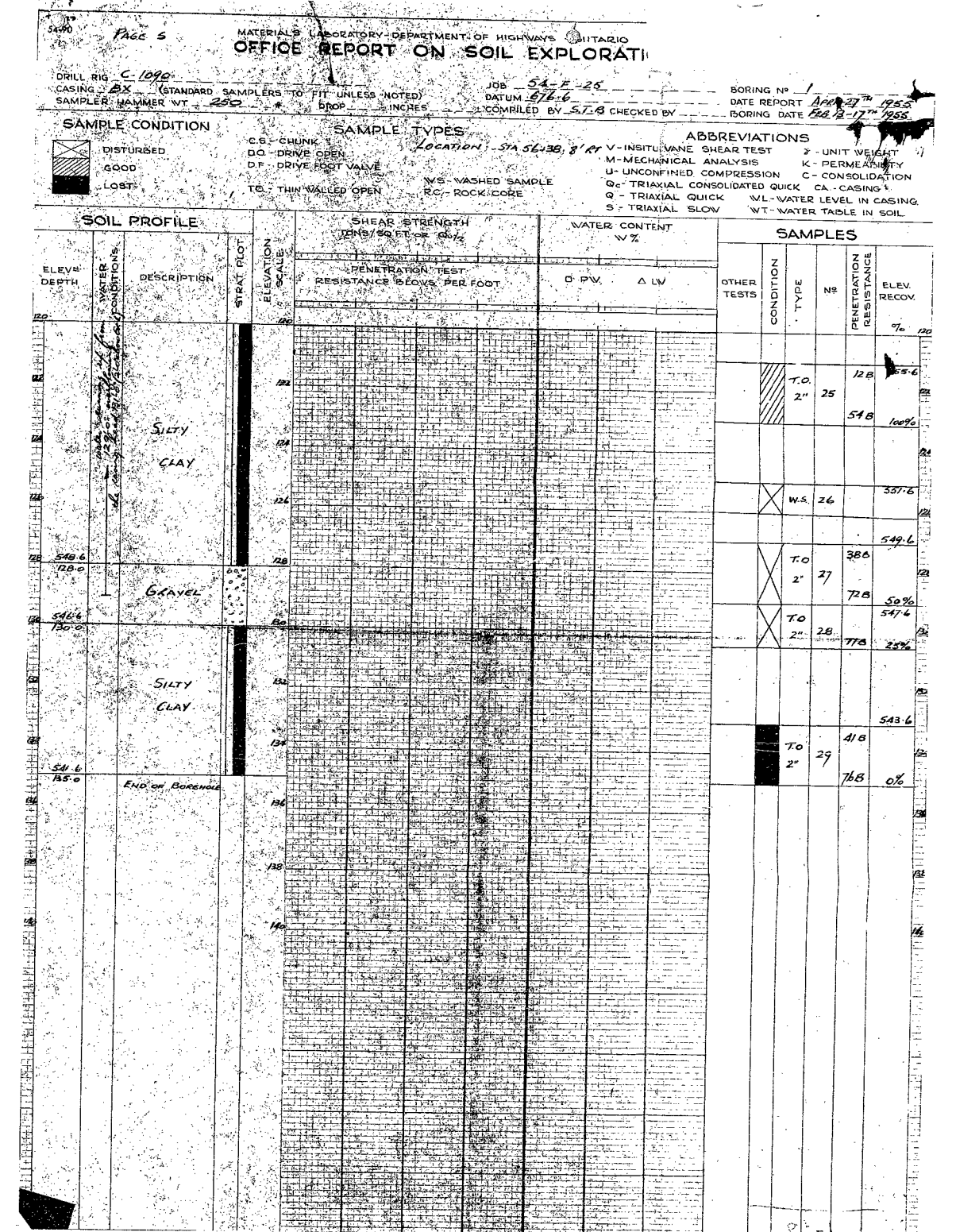
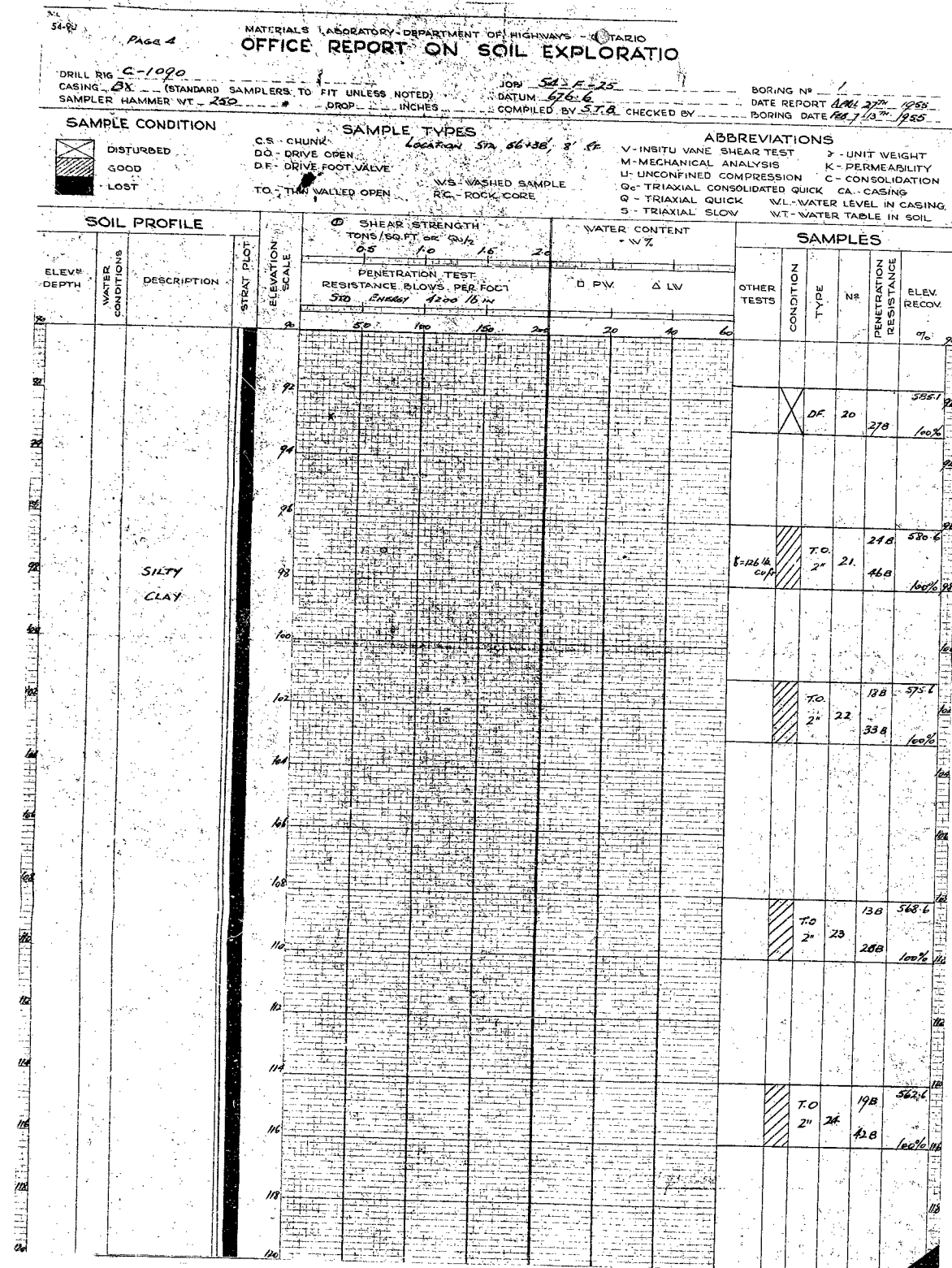
54-F-25

Hwy. # 81

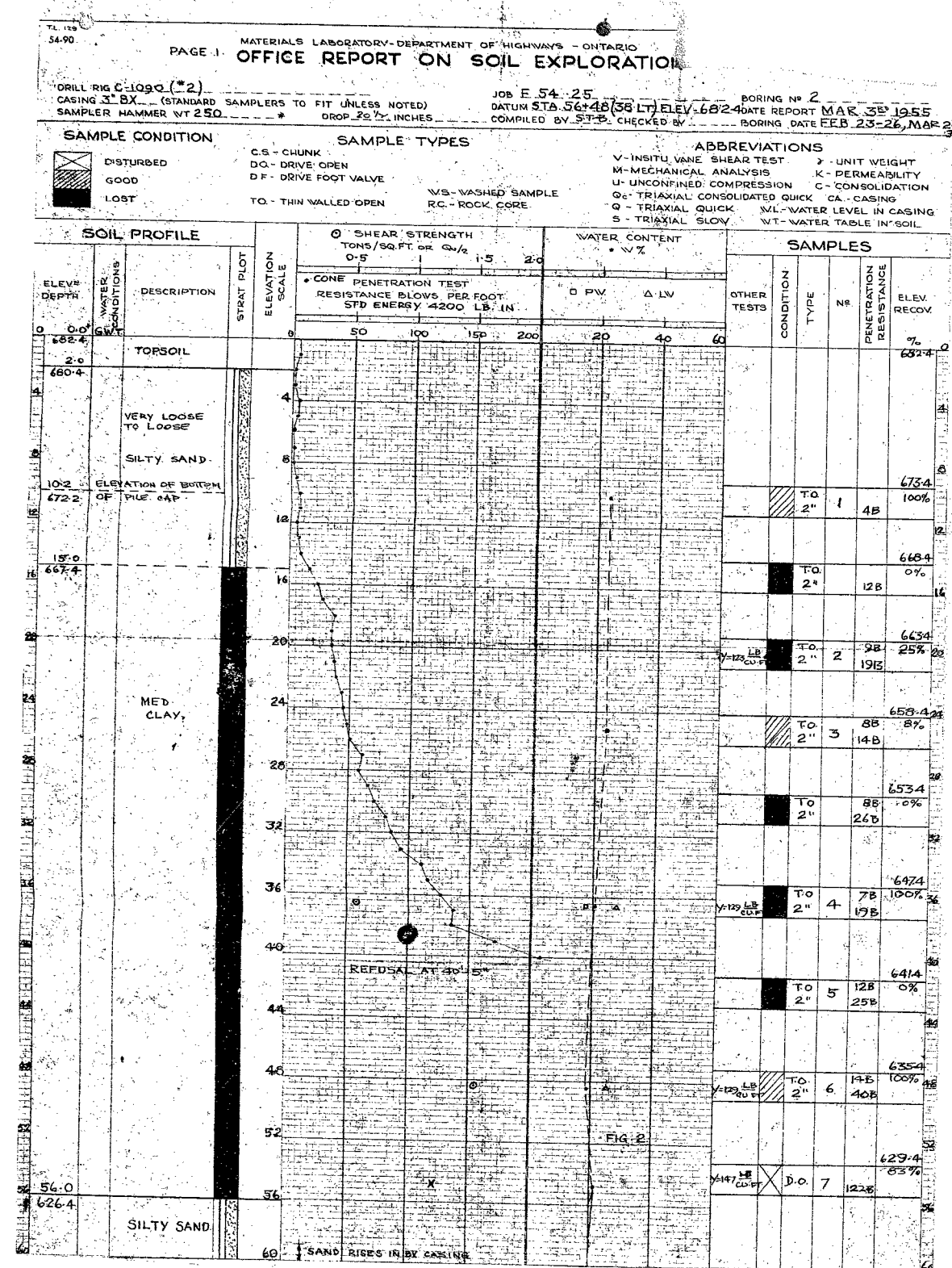
AUSABLE RIVER BRIDGE











TL 129  
54-90

PAGE 1

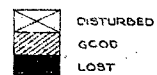
MATERIALS LABORATORY - DEPARTMENT OF HIGHWAYS - ONTARIO  
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG C-1000  
CASING 1 1/2" BX (STANDARD SAMPLERS TO FIT UNLESS NOTED)  
SAMPLER HAMMER WT 250 \* DROP 20 1/2" INCHES

JOB F-54-25  
DATUM STA. 35+21.10 R/L ELEV 6687  
COMPILED BY S.T.B. CHECKED BY

BORING NO. 4  
DATE REPORT 18th March 1955  
BORING DATE 19th March 1955

SAMPLE CONDITION



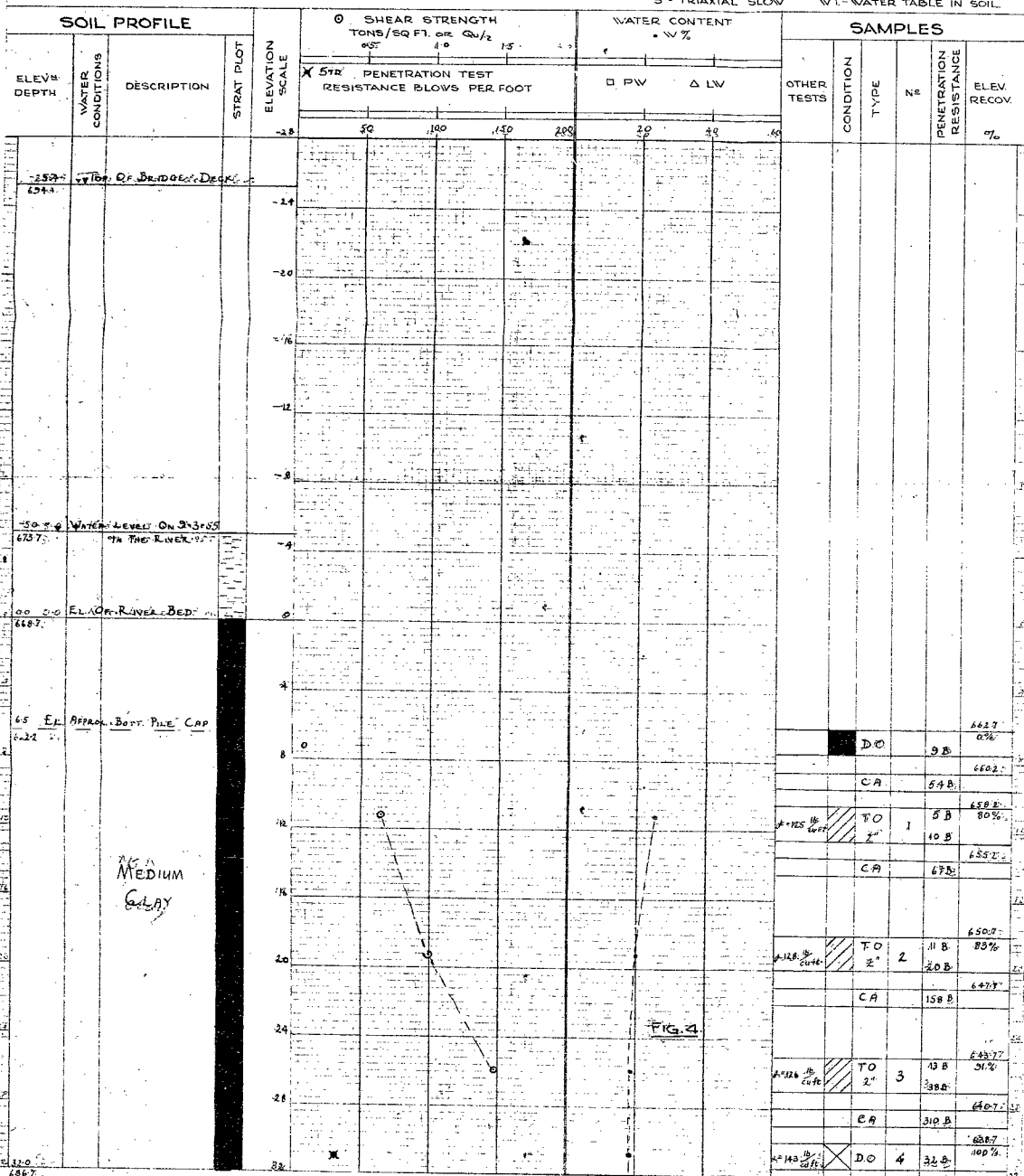
SAMPLE TYPES

CS - CHUNK  
DO - DRIVE OPEN  
DF - DRIVE FOOT VALVE  
TO - THIN WALLED OPEN

WS - WASHED SAMPLE  
RC - ROCK CORE

ABBREVIATIONS

V - INSITU VANE SHEAR TEST  
M - MECHANICAL ANALYSIS  
U - UNCONFINED COMPRESSION  
Q - TRIAXIAL CONSOLIDATED QUICK  
S - TRIAXIAL SLOW  
K - PERMEABILITY  
C - CONSOLIDATION  
CA - CASING  
WL - WATER LEVEL IN CASING  
WT - WATER TABLE IN SOIL



TL 129  
54-90

PAGE 2

MATERIALS LABORATORY - DEPARTMENT OF HIGHWAYS - ONTARIO  
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG C-1000  
CASING 1 1/2" BX (STANDARD SAMPLERS TO FIT UNLESS NOTED)  
SAMPLER HAMMER WT 250 \* DROP 20 1/2" INCHES

JOB F-54-25  
DATUM STA. 35+21.10 R/L ELEV 6687  
COMPILED BY S.T.B. CHECKED BY

BORING NO. 4  
DATE REPORT 18th March 1955  
BORING DATE 19th March 1955

SAMPLE CONDITION



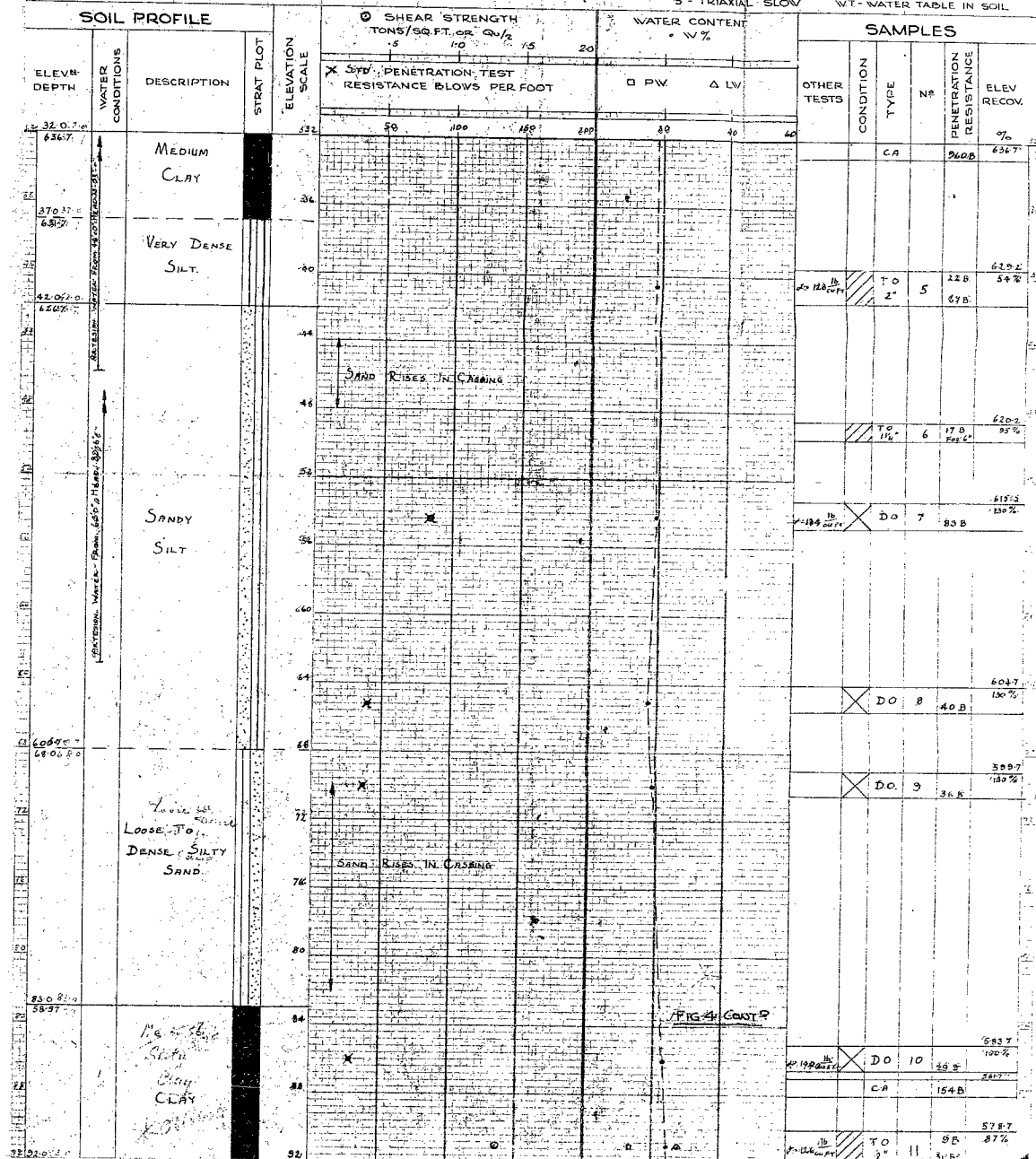
SAMPLE TYPES

CS - CHUNK  
DO - DRIVE OPEN  
DF - DRIVE FOOT VALVE  
TO - THIN WALLED OPEN

WS - WASHED SAMPLE  
RC - ROCK CORE

ABBREVIATIONS

V - INSITU VANE SHEAR TEST  
M - MECHANICAL ANALYSIS  
U - UNCONFINED COMPRESSION  
Q - TRIAXIAL CONSOLIDATED QUICK  
S - TRIAXIAL SLOW  
K - PERMEABILITY  
C - CONSOLIDATION  
CA - CASING  
WL - WATER LEVEL IN CASING  
WT - WATER TABLE IN SOIL



Page 3.

MATERIALS LABORATORY-DEPARTMENT OF HIGHWAYS - ONTARIO  
OFFICE REPORT ON SOIL EXPLORATION

DRILL BIG 5-1098 (2)

CASING 3 1/2 DIA. (STANDARD SAMPLERS TO FIT UNLESS NOTED)  
SAMPLER HAMMER WT 250 # DROP \_\_\_\_\_ INCHES

10B. ~~FF-524-1-63~~

DATUM Sta. 55+20, 10' BT, S  
COMPILED BY STA. CHECKED BY


BORING NO.

DATE REPORT 18<sup>th</sup> MARCH 1955

## 2. BOUNDED DATA

## ABBREVIATIONS

### SAMPLE CONDITION


 DISTURBED  
 GOOD  
 LOST

## SAMPLE TYPES

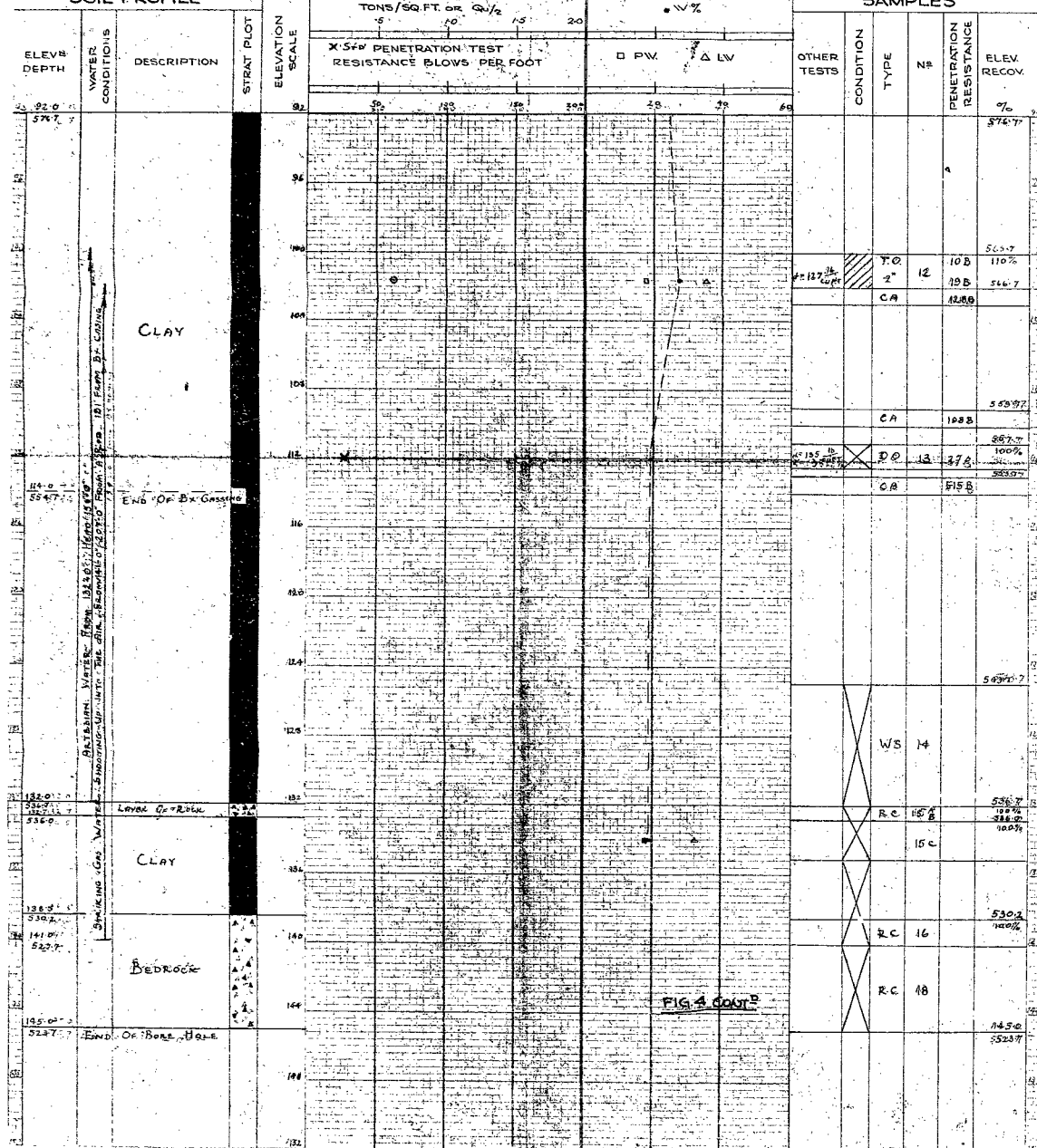
C.S - CHUNK  
D.O - DRIVE OPEN  
D.F - DRIVE FOOT VALVE  
T.O - THIN WALLED OPEN

## ABBREVIATIONS

ABBREVIATIONS

V-INSITU VANE SHEAR TEST	γ-UNIT WEIGHT
M-MECHANICAL ANALYSIS	K-PERMEABILITY
U-UNCONFINED COMPRESSION	C-CONSOLIDATION
Qc-TRIAXIAL CONSOLIDATED QUICK	CA-CASING
S-TRIAXIAL QUICK	WL-WATER LEVEL IN CASING
Q-TRIAXIAL SLOW	WT-WATER TABLE IN SOIL

### SOIL PROFILE



T.L. 124  
64.00

MATERIALS LABORATORY - DEPARTMENT OF HIGHWAYS - ONTARIO  
OFFICE REPORT ON SOIL EXPLORATION

DRILL Pic. C = 10980

CASING 3" BX (STANDARD SAMPLERS TO FIT UNLESS NOTED)  
SAMPLER HAMMER WT 250

JOB F-55A-15

DATUM Sta. 54+74 (24.0' R.A.) E.L. 190.0  
COMPILED BY S.I.B. CHECKED BY

BORING NO. 1

DATE REPORT 25<sup>th</sup> MARCH 1955

DATE \_\_\_\_\_

## ABBREVIATIONS

### SAMPLE CONDITION

DISTURBED  
GOOD  
LOST

## SAMPLE TYPES

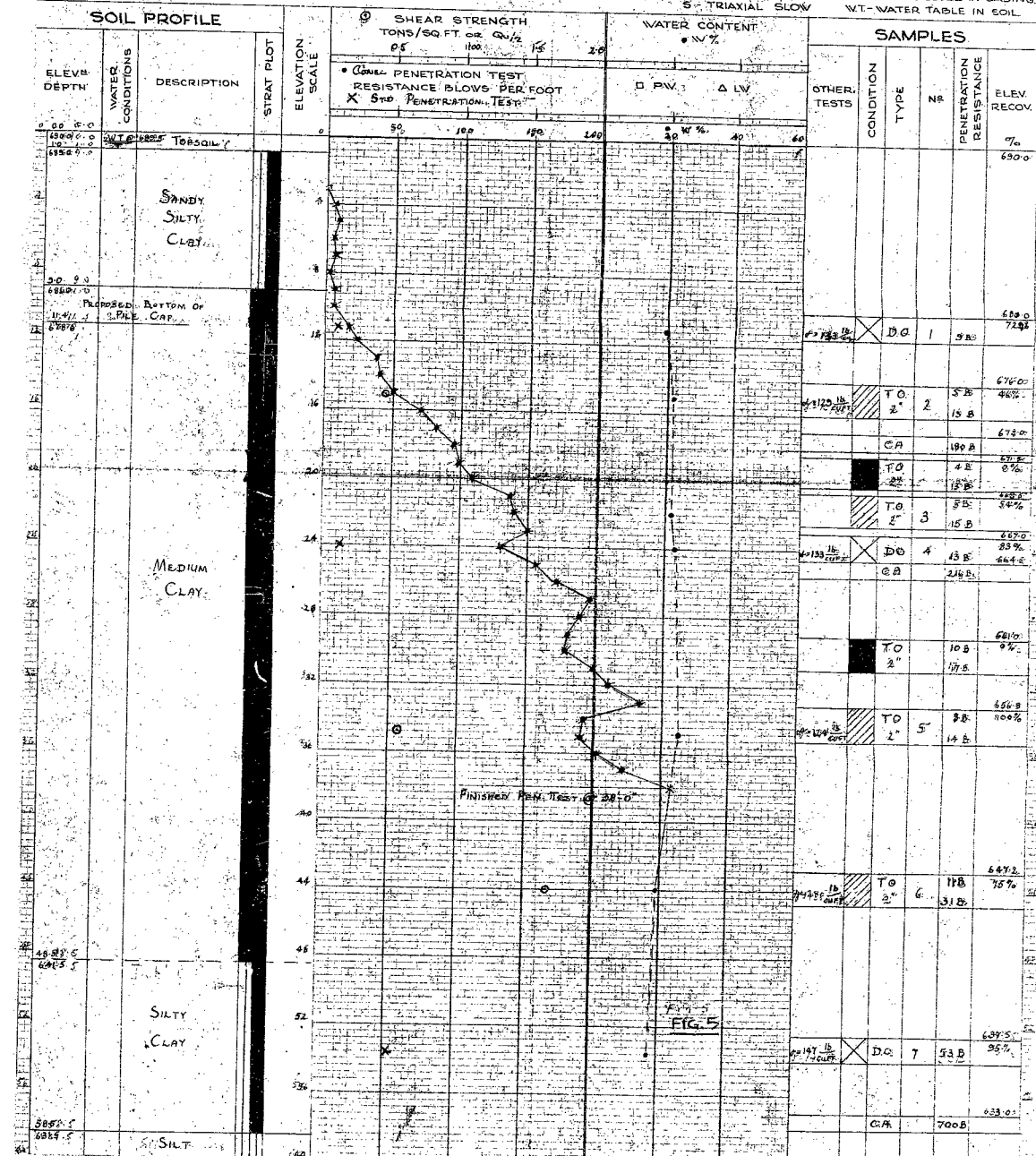
C.S - CHUNK  
D.O - DRIVE OPEN  
D.F - DRIVE FOOT VALVE  
T.O - THIN WALLED OPEN

## ABBREVIATIONS

ABBREVIATIONS

V-INSITU VANE SHEAR TEST	γ - UNIT WEIGHT
M-MECHANICAL ANALYSIS	K - PERMEABILITY
U-UNCONFINED COMPRESSION	C - CONSOLIDATION
Qc- TRIAXIAL CONSOLIDATED QUICK	CA - CASING
Q - TRIAXIAL QUICK	WL - WATER LEVEL IN CASING
S - TRIAXIAL SLOW	WT - WATER TABLE (INCL)

### SOIL PROFILE



76  
54-90

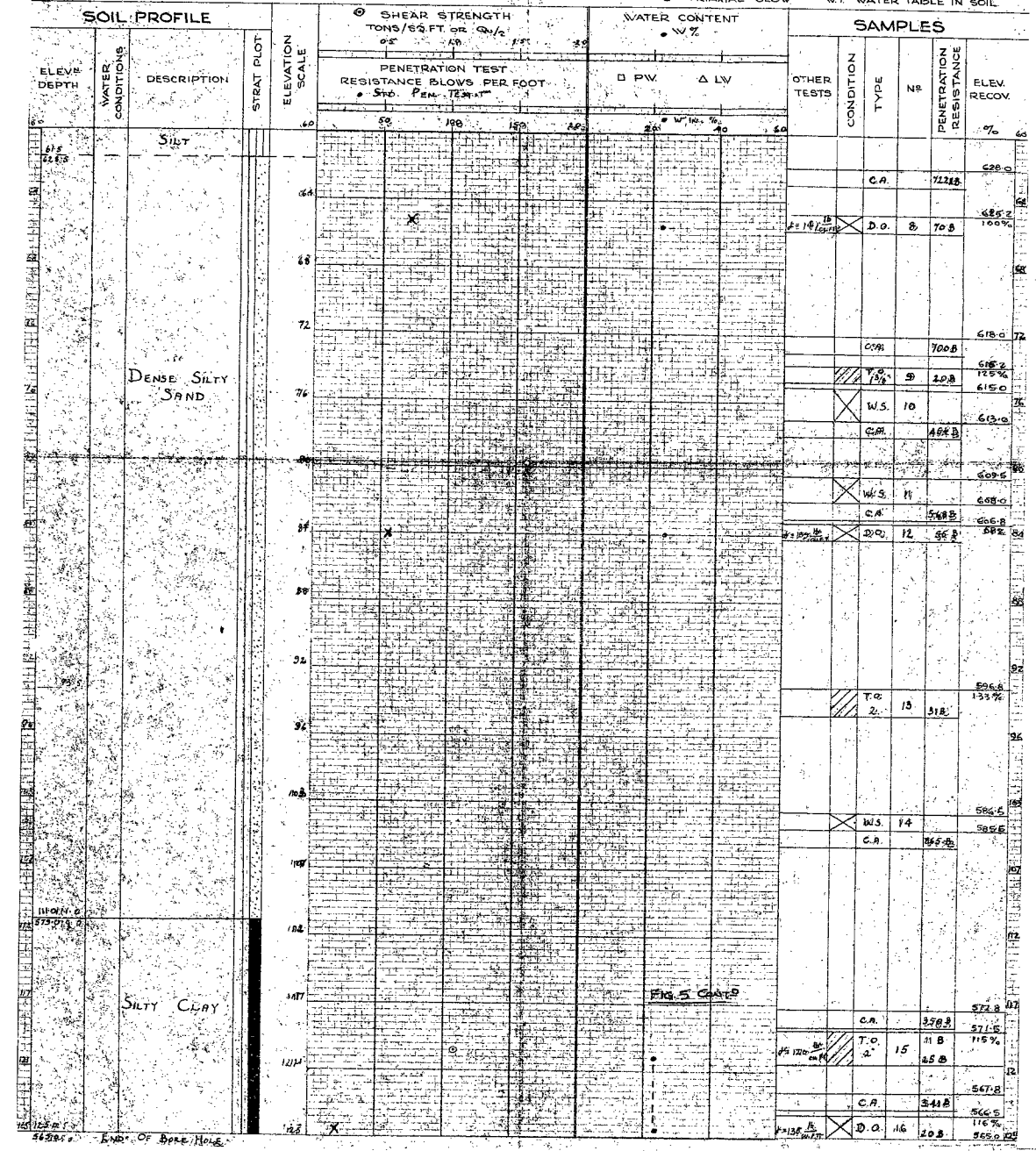
MATERIALS LABORATORY DEPARTMENT OF HIGHWAYS - ONTARIO  
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG C-1050 JOB FE 50-53 BORING NO. 5  
CASING 3" B.L. (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM 542.5474 (1874) E.L. 1000 DATE REPORT 24th MARCH 55  
SAMPLER HAMMER WT. 150 # DROP 20 1/2 INCHES COMPILED BY L.S. CHECKED BY --- BORING DATE 18th, 22nd, 23rd, 30th MARCH 55

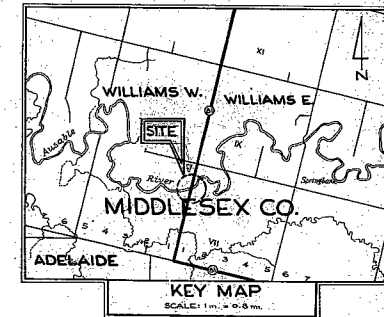
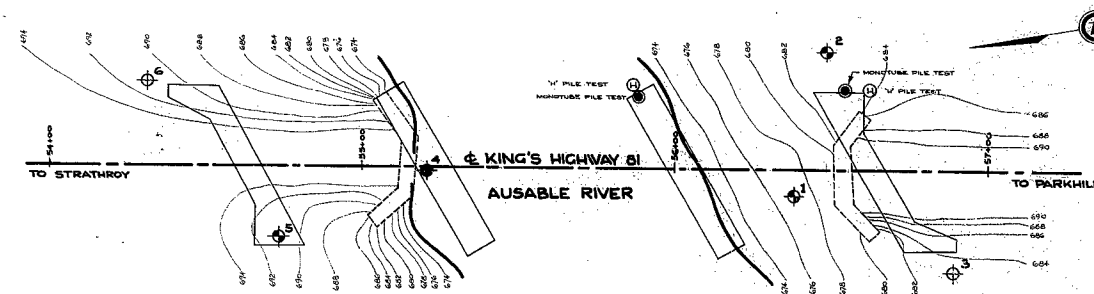
**SAMPLE CONDITION**  
DISTURBED  
GOOD  
LOST

**SAMPLE TYPES**  
C.S. - CHUNK  
D.O. - DRIVE OPEN  
D.F. - DRIVE FOOT VALVE  
TO - THIN WALLED OPEN  
W.S. - WASHED SAMPLE  
R.C. - ROCK CORE

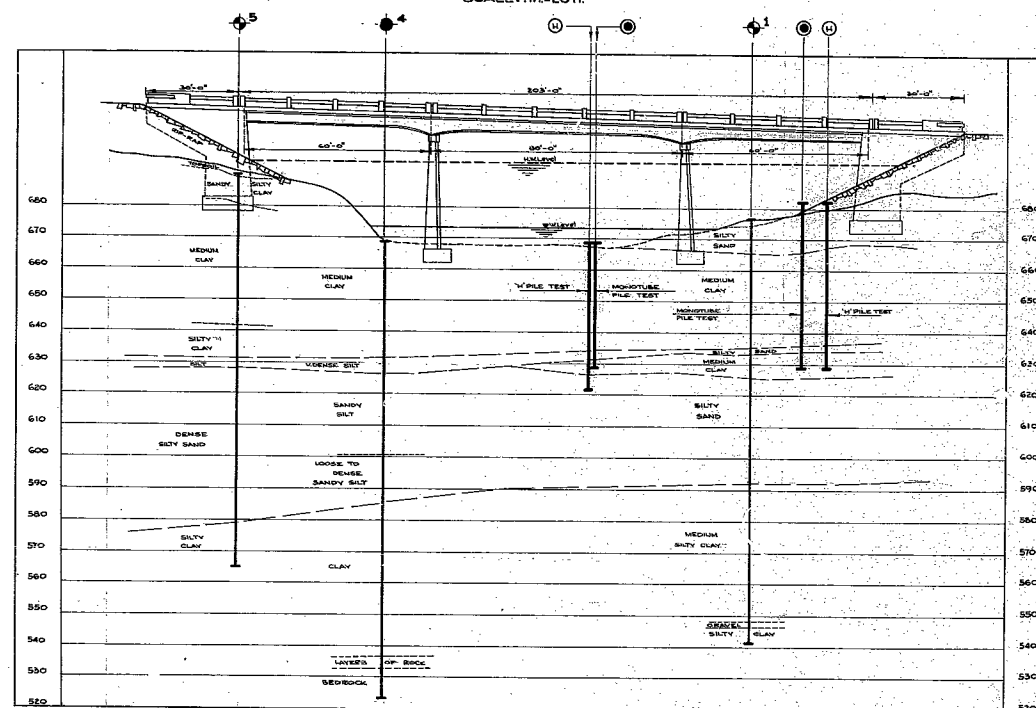
**ABBREVIATIONS**  
V - INSITU VANE SHEAR TEST  
M - MECHANICAL ANALYSIS  
U - UNCONFINED COMPRESSION  
Qc - TRIAXIAL CONSOLIDATED QUICK  
Q - TRIAXIAL QUICK  
S - TRIAXIAL SLOW  
γ - UNIT WEIGHT  
K - PERMEABILITY  
C - CONSOLIDATION  
CA - CASING  
WL - WATER LEVEL IN CASING  
WT - WATER TABLE IN SOIL







PLAN  
SCALE: 1 in. = 20 ft.



SECTION A-A  
SCALE: 1 in. = 20 ft.

LEGEND			
	PENETRATION HOLE		
	BORE HOLE		
	BORE & PENETRATION HOLE		
HOLE	ELEVATION	STATION	OFFSET
1	676.6'	56+30	2' RT.
2	682.4'	56+40	30' LT.
3	683.4'	56+90	30' RT.
4	668.7'	55+21	1' RT.
5	690.0'	54+74	23' RT.
6	691.0'	54+31	27' LT.

Geocis No. 40P4-3

DEPARTMENT OF HIGHWAYS - ONTARIO		
MATERIALS & RESEARCH SECTION		
<b>AUSABLE RIVER BRIDGE &amp; HIGHWAY NO. 81</b>		
ORIGINATED: W.B.L.	DISTRICT NO. 3	DATE: JUNE 1, 1961
DRAWN: W.L.F. REDKINDI-D.M.E.	W.P. NO.	JOB NO. P-54-25
CHECKED: R.L.L.	SCALE: 1 in. = 20 ft.	DRAWING NO.
APPROVED:		<b>F-54-25A</b>

40P4-3  
Geocis No.

Report on the  
Soils Investigation  
and Pile driving test for  
the foundations of the Anasible  
river bridge on highway 81.

Copies to:

Mr. A. Toye  
Bridge Engineer (2)

Mr. J. Walter  
Construction Engineer (1)

Mr. W. L. Fraser  
Division Engineer London (1)

Project 54-F-25.

Mr. G. Farantatos (1)

File (1)

### Introduction

The following report is concerned with a soils investigation and test piling for the foundations of the proposed Anasable river bridge on Highway 81.

The proposed renewal is a three span continuous girder raising the road level by approximately 14 feet, having two piers and two abutments.

The exploration was carried out to determine the subsoil conditions and the pile test was made to decide the length and type of pile to be used.

### Procedure

The soils investigation was begun on January 31st and completed by April 6; it comprised four boreholes varying in depth from 95 to 145 feet, and five cone penetration tests. All the soils information obtained is attached in appendix I under logs of each hole, together with a plan 34-P-25A showing the position of the holes and penetration tests relative to the proposed structure and all relevant levels.

Due to the difficulty in finding level ground, considerable work was involved in setting up the drill and winter shack. A case in point is borehole 1 which is located on the flattest ground between the existing abutment and the river bank. The drill was position<sup>ed</sup> so as to dispense with the tripod, by attaching the sheave wheel to the existing structure. Further it may be seen that borehole #4 is located in the river; due to the risk of spring flooding this hole was bored from the bridge deck.

### Soil Conditions

A general description of each hole is as follows:

#### Borehole #1

Beneath clay topsoil a bed of silty sand becoming gravel extended to a depth of eleven feet. Then followed a bed of clay 40 feet thick varying in texture from medium to silty clay.

At 51 feet depth this clay gave place to Silty sand which was found to be under a hydrostatic head throughout its entire depth to 84 feet below ground level.

The soil then reverted to silty clay for a further 51 feet to a total depth of 135 feet where the borehole was terminated.

#### Borehole #2

Conditions in this borehole were similar to those in borehole #1 commencing with loose silty sand for thirteen feet. Medium to silty clay followed from thirteen to fifty-four feet giving way to silty sand for sixteen feet and then reverting to silty clay down to the limit of the borehole at 95 feet total depth.

#### Borehole #4

In this hole zero depth refers to the river bed. The first thirty-seven feet were of medium clay giving way to a stratum of sandy silt forty-eight feet thick, under a hydrostatic head. At eighty-three feet the Soil changed to silty clay and continued to bed rock at a depth of 138.5 feet.

#### Borehole #5

Beneath one foot of topsoil a silty clay layer extends to a depth of nine feet. From this level a bed of medium clay was encountered for fifty feet becoming more Silty for the lower ten feet. Silty sand commenced at this level and extended for fifty-two feet where the soil reverted to silty clay for fourteen-feet to the limit of the boring at a total depth of 125.0.

#### Water Conditions

##### Borehole #1

A hydrostatic head was found to exist in the silty sand lying immediately beneath the upper medium clay layer causing water to overflow the casing 2'-9" above ground level. It was further observed that the borehole was making water at 42 feet depth without any noticeable head.



#### Borehole #2

Sand was found to be rising in the casing at depths of 59 feet and 72 feet. This effect was evidence of a hydrostatic head for which the free surface eventually rose to ground level.

#### Borehole #4

An artesian effect was found at depths of 46 and 63 feet in the silty sand having a maximum head of 19 feet above river bed. More water, and gas were tapped in the bedrock at a depth of 145 feet.

#### Borehole #5

After completion of the hole and withdrawal of the casing, water flooded the hole to within 6" below ground level.

#### Tests.

The undisturbed clay samples obtained have been tested to determine their cohesion properties; these were found to vary from 1000 to 3300 lbs. per square foot with an average value of 1300 lbs. per square foot. Further tests include density, liquid and plastic limit and classification.

#### Calculations

Using a value of 1300 lbs. per square foot as the adhesion between pile and clay, calculations were made to determine the length of pile embedment to develop a resistance of 150 tons, using a safety factor of 3; This figure was adopted as no loading test was made.

The results showed that for embedment in the clay alone an H pile of 12x12 section must penetrate for 58', and for a pipe pile of 12" diameter, to 76' embedment. The borings show that these conditions could not be met within the upper clay so that penetration into the silty sand would be necessary, resulting in end bearing in the silty sand or in the lower clay layer.

Due to the hydrostatic head in the silty sand its bearing properties were in doubt; the possibility existed that the material was above the critical voids ratio for the value of hydrostatic pressure it contained which would produce a quick condition.

In order to resolve this uncertainty and confirm that an adhesion of at least 1300 lbs. per square foot would develop after driving, a pile test was carried out on the 13 and 14 of June by the Birmingham Construction Company using a 500 Super Vulcan differential acting steam hammer. It comprised one H pile and one monotube pile each for the foundations of the north Abutment and North Pier. It was anticipated that little adhesion in the clay would operate during the actual driving, as this property develops with time, but a minimum value could be deduced by calculating the resistance during driving using the Hiley formulae. Knowing this resistance and the area of contact the adhesion could be found.

#### Results of Test piling

The two piles in the north abutment came to rest 53.5 feet below ground level, bearing in very firm material just above the silty sand layer.

In the north pier, the monotube pile came to rest at approximately the same elevation as the two piles in the north abutment, but the H pile penetrated the silty sand for 5 feet. At this point the splicing failed between the 50 feet and 18 feet sections due to the absence of butt welding and no further driving was possible. Calculated from the Hiley formulae, substituting the observed Energys per blow, from the calibrated Blows per minute, Blows per foot, and measured rebound, the ultimate resistance of the piles when driving ceased was as follows:

#### North Abutment

Monotube	128 tons
H pile	121.5 tons

North Pier

Monotube	151 tons
H pile	143.5 tons.

Discussion

The differences in penetration between the H piles is due to their difference of inertia. The greater the weight of a pile the less the energy reaching the foot. When driving ceased the H pile in the North Abutment weighed 5300 lbs. compared to 3604 lbs. for the H pile in the North Pier so that more energy was available for penetration in the latter case.

The measured values of rebound for the H piles were found to be less than the assumed values so that the resistances were much higher than expected. By comparison the rebounds for the monotubes were each greater than the assumed so that the dynamic resistances were less than predicted.

Graphs 1 to 4 in appendix I show the relationships between set and ultimate resistance for varying hammer speeds calculated according to Hiley using the measured values of rebound.

These show that the 68 foot H pile in the north pier could have reached the required resistance at a minimum speed of 100 blows per minute but at a penetration rate of 1200 blows per foot. Before the splicing broke the hammer was delivering 106 Blows per minute with a penetration of 336 blows per foot which gave an ultimate resistance of 143.5 tons.

When driving ceased on the 100 foot H pile it was penetrating at 13 blows per inch and the hammer was working at 116 Blows per minute giving a ultimate resistance of 121.5 tons.

In the case of the 80 feet monotube in the north abutment refusal occurred at a hammer speed of 112 blows per minute giving an ultimate resistance of 128 tons.

The 60 foot Monotube pile in the north pier reached an ultimate load of 151 tons for a penetration of 115 blows per inch.

Recommendation:

It is recommended that pipe piles be used weighing approximately 1000 lbs. for a 50 foot length, to develop an ultimate resistance of 150 tons. This allows a safe load of 50 tons using a safety factor of 3 as recommended by the national building code of Canada.

The maximum penetration of a test pile beneath a pile cap was 43 feet, in the north abutment, but due to the possibility of irregular soil conditions and damage to the pile heads, it is recommended that 50 foot lengths of pile be driven, and extra sections be available to cover any exceptional case.

The greater cross section of the pipe pile minimizes the risk of settlement which is critical factor in this case. Further, pipe piles are lighter to handle and can be more rapidly spliced.

In order to examine the inertia effects, graphs have been prepared for a 50' H section pile and a 50 ft. pipe pile using the rated energy per blow of the 500 Super Vulcan differential acting steam hammer.

The values of set and resistances for varying hammer speeds have been calculated according to the Hilley formulae and using assumed values of rebound, taken from Chellis pages 449 and 450. The curve for the pipe pile shows that an ultimate resistance of 150 tons will occur for hammer speeds greater than 95 blows per minute, and at the maximum observed speed of 115 blows per minute, the penetration must be 96 blows per foot.

The same ultimate resistance can be obtained with an H pile for hammer speeds greater than 95 blows per minute, but requiring more blows per foot of penetration than the pipe pile. At the maximum observed hammer speed of 115 blows per minute this penetration rate must be 141 blows per foot.



In order to allow for the possible differences between the assumed and actual values of rebound the pile driving should be supervised by an Engineer from the foundation section who could obtain graphs of the rebound and make field estimations for the conditions giving an ultimate resistance of 150 tons, and authorize additions to the 50 feet lengths if it became necessary. This factor is particularly important in view of the differences between actual and assumed values of rebound already noted.

Conclusions:

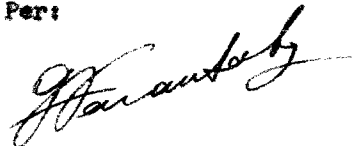
For the condition that the piles should have a safe load of 50 T. using a S.F. of 3 as recommended by the National building code of Canada, it is concluded that pipe piles of 12" diameter are best suited to this foundation, due to their lower inertia and greater cross section, which minimizes the risk of settlement.

Fifty feet sections of pile should be driven to allow for irregularities and extra lengths be available for exceptional cases in the soil conditions, damage to the pile heads and embedment in the pile cap.

An Engineer from the foundation section should be present during the driving to make graphs of pile rebound and field determinations for an ultimate resistance of 150 tons.

F. C. Brownridge  
Materials & Research Engineer

Per:



(G. N. Farantatos)

GNF:OD

APPENDIX I

120 BLOWS/MIN.

GRAPH #1

100 FT 'M' PILE (NORTH ABUTMENT)

500 VULCAN HAMMER

$C_2 + C_3 = 0.70$  ASSUMED VALUE

$C_2 + C_3 = 0.45$  ACTUAL VALUE USED

CHECKED:- P. B. L.

150 T

110 B/M

105 B/M

125 T

100 B/M

95 B/M

100 T

SET PER BLOW IN INCHES

80 T

0.02

0.03

0.04

0.05

0.06

0.07

0.08

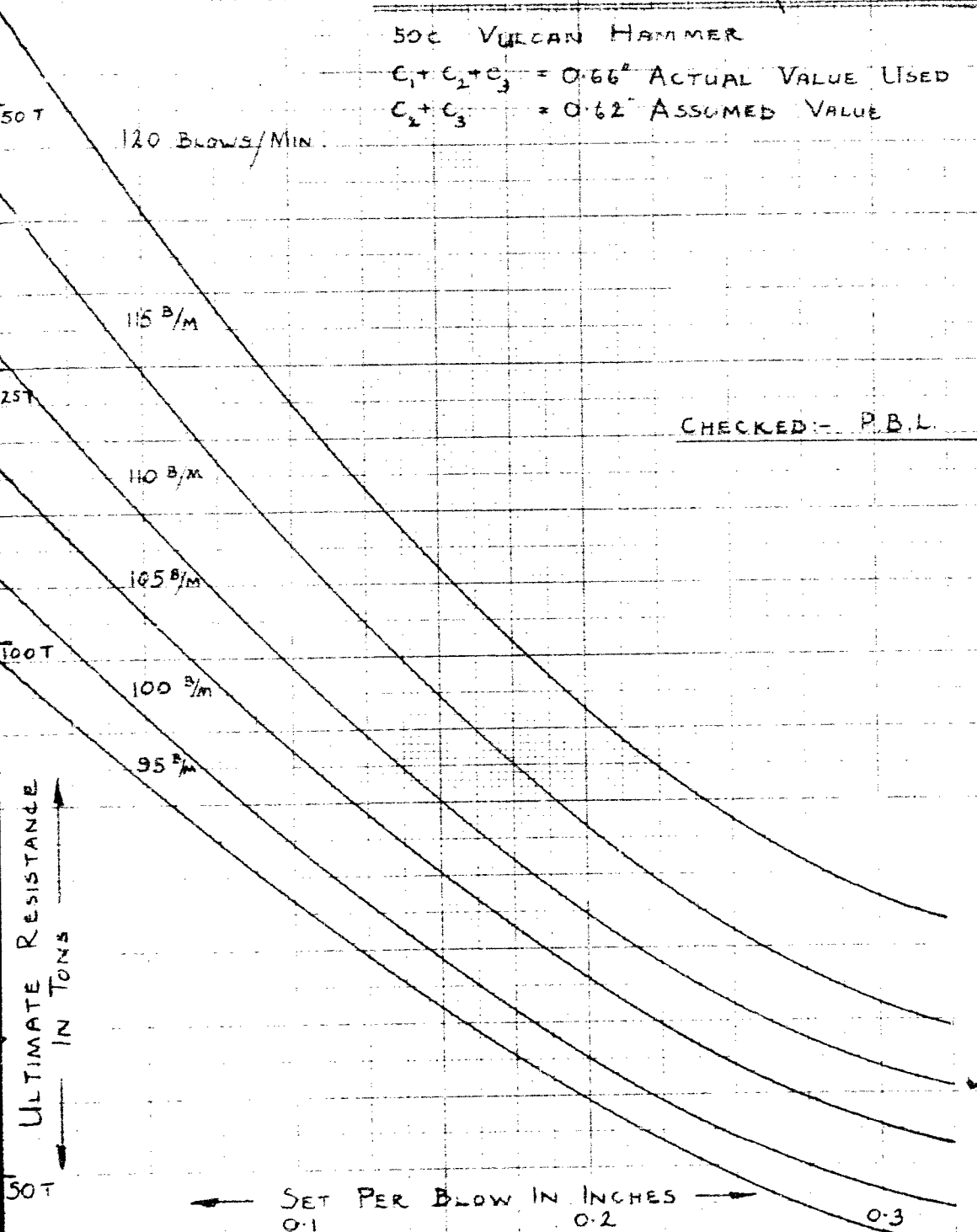
# GRAPH #2

80FT. MONOTUBE PILE (NORTH ABUTMENT)

500 VULCAN HAMMER

$C_1 + C_2 + C_3 = 0.66^2$  ACTUAL VALUE USED

$C_2 + C_3 = 0.62^2$  ASSUMED VALUE





# GRAPH # 3

68 FT 'H' PILE WEIGHING 3604 lbs.

$C_2 + C_3 = 0.4$  ACTUAL VALUES

50 c VULCAN HAMMER.

ULTIMATE RESISTANCE IN TONS

225 T

120 BLOWS/MIN

200 T

115 B/M

110 B/M

105 B/M

150 T

100 B/M

95 B/M

100

SET IN INCHES PER BLOW

CHECKED - P.B.L.

# GRAPH # 4

60 FT MONOTUBE (NORTH PIER)

500 VULCAN HAMMER

$C_1 + C_2 + C_3 = 0.62$  ACTUAL VALUES USED

$C_1 + C_2 + C_3 = 0.50$  ASSUMED VALUES

CHECKED:- P.B.L.

120 BLOWS/MIN.

115 B/M

110 B/M

105 B/M

100 B/M

95 B/M

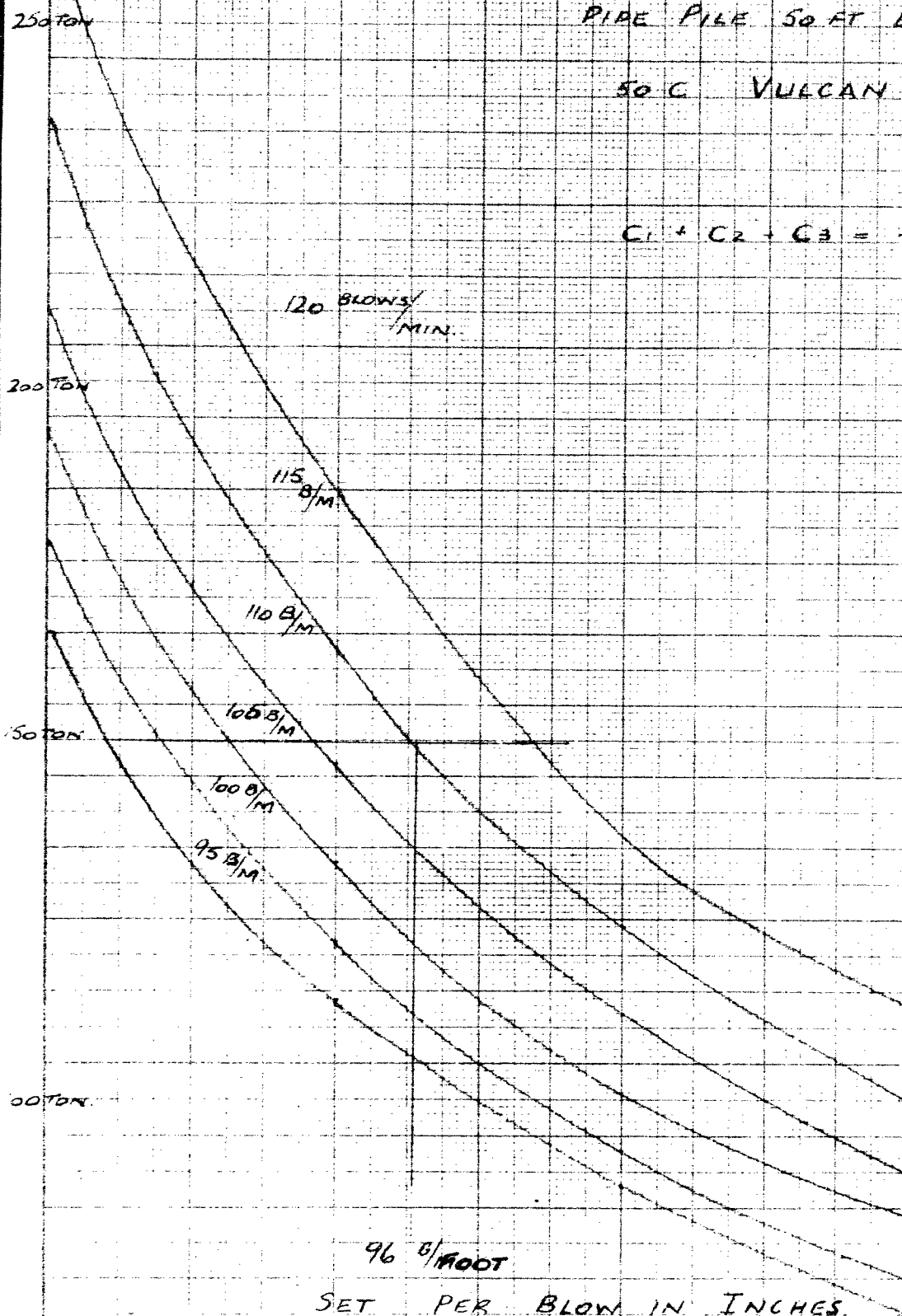
ULTIMATE RESISTANCE  
IN TONS

SET PER BLOW IN INCHES

0.01 0.03 0.05 0.07 0.09 0.11 0.13 0.15

PIPE PILE 50 FT LENGTH WEIGHING  
1000 LBS  
50 C VULCAN HAMMER

$C_1 + C_2 + C_3 = .44$  ASSUMED  
VALUES



50' FT H PILE WEIGHING 2650 lbs

50 C VULCAN HAMMER

$C_1 + C_2 + C_3 = .40$  ASSUMED VALUES

250 TONS  
120 BLOWS  
MIN

115 B/M.

200 TONS  
110 B/M.

105 B/M.

100 B/M.

95 B/M.

150 TONS

100 TONS

SET IN INCHES PER BLOW