#64-F-287M

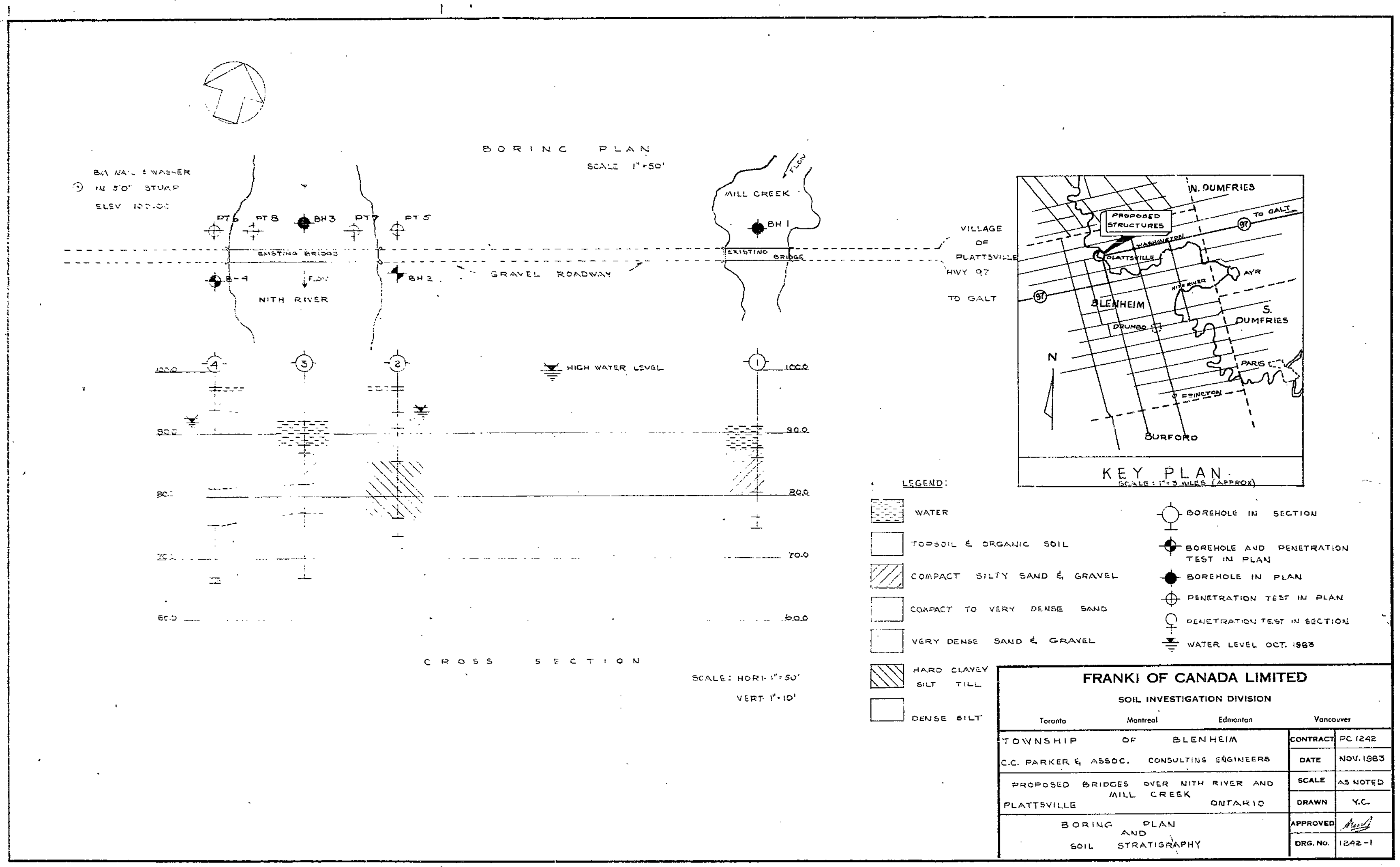
PLATTSVILLE

BRIDGE OVER

NITH RIVER

BLENHEIM

TWP.




**FRANKI OF CANADA LTD.** BORING RECORD

CONTRACT PC 1242 BORING 1 BORING DATE NOV. 4. 63

DATUM LOCAL DIAM. BX HAMMER 140 LBS. DROP 30 IN

LABORATORY TESTS

| SOIL PROFILE – DESCRIPTION | ELEV / DEPTH | ELEV. SCALE | SAMPLES – TYPE | NUMBER | % RECOVERY |
|---|---|---|---|---|---|
| MILL CREEK LEVEL (OCT. 24 63) | 91.0 | | | | |
| WATER | | 90.0 | | | |
| RIVER BOTTOM | 87.5 / 3.5 | | | | |
| DARK BROWN ORGANIC SILT, SAND & GRAVEL | 86.0 / 5.0 | 85.0 | DO | 1 | 65 |
| COMPACT TO DENSE BROWN SILTY FINE SAND | 80.5 / 10.5 | 80.0 | DO | 2 | 90 |
| DENSE TO VERY DENSE BROWN FINE TO COARSE SAND WITH A TRACE OF SILT | 76.5 / 14.5 | | DO | 3 | 100 |
| VERY DENSE GREY-BROWN SILTY COARSE SAND AND GRAVEL | 74.7 / 16.3 | 75.0 | DO | 4 | 100 |
| END OF BORING | | 70.0 | | | |

STANDARD PENETRATION RESISTANCE — DYNAMIC PENETRATION RESISTANCE — BLOWS PER FOOT 20 40 60 80

SHEAR STRENGTH LBS./SQ. FT. — WATER CONTENT % 5 10 15 20 — OTHER TESTS

**SAMPLE TYPES**

AS AUGER SAMPLE
DO DRIVE OPEN
DF DRIVE FOOT VALVE
SO SLEEVE OPEN
SF SLEEVE FOOT VALVE
TO THIN WALLED OPEN
TP THIN WALLED PISTON
WS WASHED SAMPLE

RC ROCK CORE
$K_F$ FIELD PERMEABILITY TEST
GROUND WATER LEVEL AT TIME OF BORING
REMARKS

O WATER CONTENT
ATTERBERG LIMITS
$\gamma$ IN SITU UNIT WEIGHT
M MECHANICAL ANALYSIS
$K_F$ PERMEABILITY FIELD
$K_L$ PERMEABILITY LAB.
$d_R$ RELATIVE DENSITY
s.g. SPECIFIC GRAVITY
P COMPACTION

UNCONFINED
UNDRAINED TRIAXIAL
IN SITU VANE
LAB VANE
CU CONSOLIDATED UNDRAINED
CUP CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS
d CONSOLIDATED DRAINED
C CONSOLIDATION

## FRANKI OF CANADA LTD. — BORING RECORD

CONTRACT PC 1242 — BORING 2 — BORING DATE [illegible]

DATUM LOCAL — DIAM. 4½ & BX — HAMMER 140 LBS. DROP 30 IN

**LABORATORY TESTS**

| Description | Elev / Depth |
|---|---|
| GROUND LEVEL | 97.5 / 0.0 |
| TOPSOIL | 0.5 |
| LOOSE TO COMPACT BROWN SANDY SILT WITH ORGANIC INTRUSIONS | 93.5 / 4.0 |
| COMPACT TO DENSE BROWN SILTY SAND AND GRAVEL | 91.0 / 6.5 |
| COMPACT TO VERY DENSE BROWN GRADED SAND AND GRAVEL WITH A TRACE OF SILT | 85.5 / 12.0 |
| HARD GREY CLAYEY SILT TILL | 76.5 / 21.0 |
| VERY DENSE GREY BROWN COARSE SAND AND GRAVEL WITH A LITTLE SILT. | 73.5 / 24.0 |
| END OF BORING | |

Elev. scale: 95.0, 90.0, 85.0, 80.0, 75.0, 70.0

| Type | Number | % Recovery |
|---|---|---|
| DO | 1 | 65 |
| DO | 2 | 95 |
| DO | 3 | 80 |
| DO | 4 | 0 |
| DO | 5 | 100 |
| DO | 6 | 100 |
| DO | 7 | 95 |

SAMPLE TYPES
AS AUGER SAMPLE
DO DRIVE OPEN
DF DRIVE FOOT VALVE
SO SLEEVE OPEN
SF SLEEVE FOOT VALVE
TO THIN WALLED OPEN
TP THIN WALLED PISTON
WS WASHED SAMPLE

RC ROCK CORE
$K_F$ FIELD PERMEABILITY TEST
GROUND WATER LEVEL AT TIME OF BORING
REMARKS

O WATER CONTENT — ● UNCONFINED
ATTERBERG LIMITS — ▲ UNDRAINED TRIAXIAL
γ IN SITU UNIT WEIGHT — IN SITU VANE
M MECHANICAL ANALYSIS — LAB VANE
$K_F$ PERMEABILITY FIELD — CU CONSOLIDATED UNDRAINED
$K_L$ PERMEABILITY LAB. — CUP CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS
$d_R$ RELATIVE DENSITY — d CONSOLIDATED DRAINED
s.g. SPECIFIC GRAVITY — C CONSOLIDATION
P COMPACTION

## FRANKI OF CANADA LTD. — BORING RECORD

CONTRACT PC 1242 — BORING 3 — BORING DATE NOV 6.63

DATUM LOCAL — DIAM. BX — HAMMER 140 LBS. DROP 30 IN

**LABORATORY TESTS**

| Description | Elev / Depth |
|---|---|
| RIVER LEVEL (OCT. 24 63) | 92.0 |
| RIVER BOTTOM | 88.25 / 3.75 |
| LOOSE BLACK ORGANIC SANDY SILT AND GRAVEL | 5.0 |
| COMPACT BROWN SILTY SAND AND GRAVEL | 82.0 / 10.0 |
| COMPACT GREY FINE SAND WITH SOME SILT | 77.5 / 14.5 |
| VERY DENSE GREY BROWN SAND AND GRAVEL WITH A TRACE OF SILT | 66.75 / 25.25 |
| END OF BORING | |

Elev. scale: 90.0, 85.0, 80.0, 75.0, 70.0, 65.0

| Type | Number | % Recovery |
|---|---|---|
| DO | 1 | 25 |
| DF | 2 | 0 |
| WS | | |
| DF | 3 | 100 |
| DO | 4 | 65 |
| DO | 5 | 90 |
| DO | 6 | 95 |

FOR 4"
FOR 3"

SAMPLE TYPES
AS AUGER SAMPLE
DO DRIVE OPEN
DF DRIVE FOOT VALVE
SO SLEEVE OPEN
SF SLEEVE FOOT VALVE
TO THIN WALLED OPEN
TP THIN WALLED PISTON
WS WASHED SAMPLE

RC ROCK CORE
$K_F$ FIELD PERMEABILITY TEST
GROUND WATER LEVEL AT TIME OF BORING
REMARKS

## FRANKI OF CANADA LTD. — BORING RECORD

CONTRACT PC 1242 — BORING 4 — BORING DATE OCT. 30.63

DATUM LOCAL — DIAM. 4½ INCH & BX — HAMMER 140 LBS. DROP 30 IN

**LABORATORY TESTS**

| Description | Elev / Depth |
|---|---|
| GROUND LEVEL | 97.5 / 0.0 |
| TOPSOIL | 0.5 |
| COMPACT BROWN SAND & GRAVEL WITH SOME SILT | 94.0 / 3.5 |
| DENSE BROWN SAND AND GRAVEL | 81.5 / 16.0 |
| VERY DENSE GREY FINE TO COARSE SAND WITH A TRACE OF SILT | 75.5 / 22.0 |
| DENSE TO VERY DENSE GREY SILT AND SANDY SILT WITH LAYERS OF CLAYEY SILT | 66.5 |
| VERY DENSE GREY SILTY SAND & GRAVEL | 66.0 / 31.5 |
| END OF BORING | |

Elev. scale: 95.0, 90.0, 85.0, 80.0, 75.0, 70.0, 65.0

| Type | Number | % Recovery |
|---|---|---|
| DO | 1 | 80 |
| DO | 2 | 85 |
| DO | 3 | 90 |
| DO | 4 | 100 |
| DO | 5 | 95 |
| DO | 6 | 95 |
| DO | 7 | 85 |
| DO | 8 | 65 |
| DO | 9 | 95 |

150/11"

SAMPLE TYPES
AS AUGER SAMPLE
DO DRIVE OPEN
DF DRIVE FOOT VALVE
SO SLEEVE OPEN
SF SLEEVE FOOT VALVE
TO THIN WALLED OPEN
TP THIN WALLED PISTON
WS WASHED SAMPLE

RC ROCK CORE
$K_F$ FIELD PERMEABILITY TEST
GROUND WATER LEVEL AT TIME OF BORING
REMARKS

O WATER CONTENT — ● UNCONFINED
ATTERBERG LIMITS — ▲ UNDRAINED TRIAXIAL
γ IN SITU UNIT WEIGHT — IN SITU VANE
M MECHANICAL ANALYSIS — LAB VANE
$K_F$ PERMEABILITY FIELD — CU CONSOLIDATED UNDRAINED
$K_L$ PERMEABILITY LAB. — CUP CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS
$d_R$ RELATIVE DENSITY — d CONSOLIDATED DRAINED
s.g. SPECIFIC GRAVITY — C CONSOLIDATION
P COMPACTION

BA 1742

FRANKI OF CANADA, LIMITED

SOIL INVESTIGATIONS

214 MERTON ST. TORONTO
HU. 1-6426-7

# REPORT

# TO

# TOWNSHIP OF BLENHEIM

# ON

# SOIL CONDITIONS AND FOUNDATIONS

# PROPOSED PLATTSVILLE BRIDGE

# OVER RIVER NITH

# TOWNSHIP OF BLENHEIM, COUNTY OF OXFORD

# ONTARIO.

Distribution: 6 copies - C.C. Parker & Associated Limited,
Consulting Professional Engineers.

C.C.Parker Reference: 2083-20-10.

Our Reference: PC-1242.

December 23rd, 1963.

# I N D E X

INTRODUCTION

Franki of Canada Limited has been retained by C.C. Parker & Associates Limited, by letter of October 16th, 1963, to carry out a soil investigation for the Township of Blenheim. The investigation was to be carried out at the site of the proposed new structures over the Mill Creek and Nith River in Lot 19, Concessions ~~7~~ 12 and ~~8~~ 13, at the road crossing on the west side of the town of Plattsville.

The object of the investigation was to determine the soil conditions at the site as they affect the foundation design of the proposed new structure.

PROCEDURES

The field work was carried out between October 24th and November 7th, 1963, and consisted of 2 detailed auger boreholes with adjacent dynamic cone penetration tests and 2 detailed wash borings, at predetermined locations. Four additional dynamic cone penetration tests were carried out. The locations of the borings are shown on Drawing 1242-1 which also shows a section of the inferred soil stratigraphy. A detailed log for each boring and the results of the additional dynamic cone penetration tests are given on the Boring Records.

Soil testing on samples obtained was carried out in our laboratory and the results are shown on the Boring Records and on the Figures of the Appendix. Samples remaining after testing will be stored until June 15th, 1964, and then discarded unless other instructions are received.

Elevations referred to in this report are related to a local datum defined by a bench mark consisting of a nail and washer in a 5 foot stump on the west side of the Nith River, approximately 50 feet north of the road. The elevation of this bench mark was given as 100.0.

SITE CONDITIONS

The site is located on the flood plain of the Nith River. The topography of the site is level to undulating. The river meanders at the site. The proposed river structure is perpendicular to a relatively straight section of the river, while the proposed Creek structure will intersect the creek a few degrees off the perpendicular. The centre line of the proposed new structures in the same as the centre line of the present wooden structures.

From geological information, it is known that the site is generally covered by silts, sands and gravels overlying glacial till which extends to shaly dolomite bedrock.

SOIL CONDITIONS

GENERAL

The materials encountered were mostly granular, with variations in gradation and silt content both with depth and across the site. Clay till was encountered in one borehole, and silt was encountered in another. The soil conditions are discussed below in detail.

MILL CREEK

Borehole 1 was located in the Mill Creek channel half way between the north ends of the proposed structure. The boring was carried out with a raft mounted diamond drill boring machine. The upper 1.5 feet consists of organic silt, sand and gravel.

The organic material is underlain by 5.5 feet of silty fine sand. The 'N' values obtained were 28 and 38 blows per foot, indicating a compact to dense relative density. The permeability coefficient is about 5 x $10^{-3}$ cm. per second.

a grain size distribution curve is shown on Figure 1 of the Appendix.

The silty sand is underlain by 4 feet of fine to coarse sand with a trace of silt. The 'N' values obtained were 38 and over 100 blows per foot, indicating a dense to very dense relative density. The permeability coefficient is about 1 x $10^{-2}$ cm. per second. Two grain size distribution curves are shown on Figure 1 of the Appendix.

The sand is underlain by silty coarse sand and gravel, to the depth explored at 16.3 feet. The 'N' value obtained was over 100 blows per foot indicating a very dense relative density. The permeability coefficient is about 5 x $10^{-2}$ cm. per second.

EAST ABUTMENT

Borehole 2 was located at the south end of the proposed abutment, and an additional dynamic cone penetration test PT.5 was carried out at the north end.

The upper 6 inches is topsoil consisting of dark brown organic sandy silt.

The topsoil is underlain by 3.5 feet of brown sandy silt with organic intrusions. The results of the standard penetration and dynamic cone penetration tests indicate a loose to compact relative density.

The organic material is underlain by 2.5 feet of silty sand and gravel. The standard penetration and dynamic cone penetration tests indicate a compact to dense relative density. The permeability coefficient is about 5 x $10^{-3}$ cm. per second.

The silty sand and gravel is underlain by 5.5 feet

feet of graded sand and gravel with a trace of silt. The 'N' values obtained were 67 and 17 indicating erratic compact to dense relative densities. The dynamic cone penetration test at PT.5 indicated erratic relative densities to elevation 85, suggesting a condition similar to that at borehole 2. The cone penetration test adjacent to borehole 2 met refusal at about elevation 90, probably on the dense zone encountered at this elevation. The coefficient of permeability is about $2 \times 10^{-1}$ cm. per second. A grain size distribution curve is shown on Figure 2 of the Appendix.

The sand and gravel is underlain by 9 feet of clayey silt till which consists of silt and fine sand with a few stones up to one inch, in a clayey silt matrix. There are find sand lenses averaging about 2 inches apart. The liquid limit and plasticity index was 19% and 6.5% respectively, at a natural moisture content of about 11%, which is less than the plastic limit of 12.5%. The two unit weights determined were 146 p.c.f. and 134 p.c.f. An undrained triaxial compression test gave a shear strength of 4500 p.s.f. Sand lenses and stones in the other samples prevented further reliable strength testing. However, the consistency can be taken as hard, as substantiated by the 'N' values which ranged from 31 to 41 blows per foot. This material is practically impervious. A grain size distribution curve is shown on Figure 2 of the Appendix.

The clayey silt till is underlain by coarse sand and gravel with a little silt, extending to the depth explored at 24 feet. The 'N' value obtained was 62 blows per foot, indicating a very dense relative density.

WEST ABUTMENT

Borehole 4 was located at the south end of the proposed west abutment, and an additional dynamic cone penetration test PT.6, was carried out at the north end.

The upper 6 inches is topsoil consisting of dark brown organic sandy silt.

The topsoil is underlain by 3 feet of compact sand and gravel with some silt.

The silty material is underlain by about 12 feet of sand and gravel. The 'N' values obtained were about 40 blows per foot, indicating a dense relative density, which was substantiated by both dynamic cone penetration tests. The coefficient of permeability is about 1 x $10^{-1}$ cm. per second. A grain size distribution curve is shown on Figure 4 of the Appendix.

The sand and gravel is underlain by 6 feet of fine to coarse sand with a trace of silt. The 'N' values obtained were 50 and 75 blows per foot, indicating a very dense relative density. The coefficient of permeability is about 1 x $10^{-2}$ cm. per second. Two grain size distribution curves are shown on Figure 4 of the Appendix.

The sand is underlain by 11 feet of silt and fine sandy silt with local zones containing a little clay. The 'N' values obtained were 32 and 85 blows per foot, indicating a dense to very dense relative density. The coefficients of permeability are in the order of 1 x $10^{-4}$ to 1 x $10^{-6}$ cm. per second. Two grain size distribution curves are shown on Figure 5 of the Appendix.

The silt is underlain by silty sand and gravel to the depth explored which was 6 inches below the silt. The 'N' value obtained was 76 blows per foot, indicating a very dense relative density. The coefficient of permeability is about $1 \times 10^{-3}$ cm. per second. A grain size distribution curve is shown on Figure 4 of the Appendix.

RIVER PIERS

Borehole 3 was located at the centre of the proposed 80 foot span between the piers, 25 feet north of the centre line. Additional penetration tests PT.7 and PT.8 were carried out at the north ends of the proposed piers. Borehole 3 and the penetration tests were carried out from a wash boring diamond drill machine floated on a raft. The cone tests at the pier locations indicated high densities at the same elevation at which very dense material was encountered in borehole 3. Conditions in borehole 3 are therefore described as applicable to the two pier locations.

The river bottom is covered by about one foot of loose black organic sandy silt and gravel.

The organic material is underlain by 5 feet of silty sand and gravel similar in composition to that in borehole 1. The 'N' value obtained was 20 blows per foot, indicating a compact relative density. The coefficient of permeability is about $1 \times 10^{-2}$ cm. per second.

The silty sand and gravel is underlain by about 5 feet of fine sand with a little silt. The 'N' value obtained was 21 blows per foot, indicating a compact relative density. The coefficient of permeability is about $1 \times 10^{-2}$ cm. per second. A grain size distribution curve is shown on Figure 3 of the Appendix.

The fine sand is underlain by sand and gravel with a trace of silt, which extended to the depth explored at 26.5 feet. The 'N' values obtained ranged from 53 to over 100 blows per foot, indicating a very dense relative density. The coefficient of permeability is about $1 \times 10^{-3}$ cm. per second. A grain size distribution curve is shown on Figure 3 of the Appendix.

WATER CONDITIONS

At the time of the investigation the water level of the Nith River was at elevation 92. As would be expected with pervious materials forming the banks of the river, the water levels at the abutments were very close to the river level. The level in borehole 2 was at elevation 93, and the level in borehole 4 was at elevation 92.

The water level of Mill Creek was at elevation 91.

The high water level as recorded on C.C. Parker Drawing 2083-2 is 100.0. During high water the flood plain between the creek and the river is under water.

DISCUSSION

GENERAL

At present the road is carried over the Mill Creek and Nith River by wooden structures.

The structure over the creek will be replaced by a rigid concrete structure with a single 20 foot span. Although the abutment design loads are not known, they are estimated to be in the order of 15 kips per lineal foot. The allowable differential settlement between piers is not known but has been estimated to be no greater than 1/1000 of the span length, which would be only ¼ of an inch.

The present structure over the Nith River will be replaced by a 3 span continuous reinforced concrete girder bridge. The end spans will be 45 feet long, and the centre span will be 80 feet. The design loads are not known, but it is understood that the abutment loads will be in the order of

300 to 400 kips and the pier loads will be in the order of 700 to 800 kips. Although the allowable differential settlement between piers is not known, it has been estimated to be no greater than $^{1}/1000$ of the span length, which would be only ½ inch to 1 inch. The recommended bearing capacities take this settlement into account. These bearing capacities are expected to result in foundation areas within design dimensions.

Another factor in the foundation design, other than bearing capacities, is the amount of scour that may be expected. Besides being related to soil conditions, the scour is also dependent on the hydrologic properties of the drainage basin. The hydrologic properties of the site should therefore be studied. The foundation elevations recommended in this report have allowed for a depth of scour below river bottom equal to the height of river fluctuation which is about 10 feet. Therefore, scour susceptible materials above this depth are not considered as suitable foundation materials regardless of their bearing capacities.

It should be noted that in the Department of Highways Report W.P.20-58 W.J.F.58-31, covering the bridge site at the Nith River on Highway 97, on the south side of Plattsville, it is recommended that the minimum depth below the river bottom to the foundation level should be 15 feet. Since this depth of 15 feet was based on hydrologic conditions, it may well be that a depth of 15 feet will also apply at this site, establishing a maximum foundation elevation of 73.

The preliminary design calls for piled foundations for the river structure and spread footings for the creek structure. As discussed later in the report, end bearing piles are recommended for all the foundations, because of the depths of excavation below the water table and the dewatering difficulties associated with founding spread footings in pervious materials. The elevations shown on the preliminary design as "top of footing" have been taken as the elevation of the top of the pile caps. The pile caps have been assumed to be 3 feet thick. The elevations to which dewatering would be required for the pile caps would then be as follows:

| | | |
|---|---|---|
| Mill Creek Abutments | - | Elevation 84 |
| Nith River Abutments | - | Elevation 90 |
| Nith River Piers | - | Elevation 84.5 |

MILL CREEK

From the standpoint of bearing capacity, and frost protection, the structure could be founded at a maximum elevation of 84, using an allowable bearing capacity of 1.5 tons per square foot. Unless sufficient hydrologic information is available to establish the depth of scour, it should be assumed to be at least as great as the fluctuation in water level which is about 10 feet. Accordingly, the structure should not be founded in scour susceptible material above elevation 77. The compact to dense silty fine sand down to elevation 80 is particularly susceptible to scour, and the very dense fine to coarse sand down to elevation 76 is also susceptible to scour. It is recommended that the structure be founded no higher than elevation 77. The allowable bearing pressure at this elevation for spread fottings of the size required, is 2 tons per square foot.

EAST ABUTMENT

The granular materials above elevation 85 are not suitable for founding a structure of this size and importance, because of their unfavourable strength and settlement characteristics as indicated by their loose or erratic relative densities. From the standpoint of bearing capacity, the structure could be founded in the till below elevation 85 using an allowable bearing pressure of 3 tons per square foot.

The till is relatively resistant to scour, and the approach embankment will have to be protected against erosion, so that scour should not occur at the abutment. However, depending on hydrologic conditions, it may be advisable to found the structure at a lower elevation, since surface protection should not be considered as a completely dependable substitute for adequate depth.

WEST ABUTMENT

From the standpoint of bearing capacity, the structure could be founded at elevation 95 or lower, using an allowable bearing capacity of 2 tons per square foot. However, this material is susceptible to scour, so that the foundation level will depend on the effects of scour as determined from hydrologic conditions. Even though scour should not occur because of the protection provided for the approach embankment, it is recommended that the foundation be carried a minimum depth below river level, equal to the fluctuation in the river level, which would place the foundation at a maximum elevation of 77.

RIVER PIERS

The granular materials above elevation 77 has an allowable bearing pressure of 1 ton per square foot. The very dense sand and gravel below elevation 77 has an allowable bearing pressure of 2 tons per square foot. All the materials encountered, and particularly those above elevation 77, are susceptible to scour, so that the foundation level is dependent on hydrologic conditions as they affect scouring. It is considered that the maximum foundation level because of scour considerations would be at elevation 77.

DEWATERING & EXCAVATION

The utilization of spread footings would result in a costly dewatering system. The excavations in the channel are expected to be at least 10 feet deep, extending at least 15 feet below the river level. The excavations for the abutments are expected to be at least 15 feet below the present water level. The excavations would have to extend sufficiently far to prevent the bottom of the excavation from becoming quick at the perimeter. As a general rule, a penetration equal to the hydrostatic head could be considered satisfactory.

The coefficients of permeability for the various materials encountered are shown under "SOIL CONDITIONS". The pumping system capacities should be estimated from these coefficients. The only boring in which an impervious stratum was encountered was in borehole 2 at the East Pier location.

The extent of dewatering for pile foundations would be considerably less but would still involve closed sheeting. Based on the footing and pile cap elevations indicated by the preliminary design, the depths of excavation and depths below present water level would be as follows:

| | Depth of Excavation. | Depth of Water. |
|---|---|---|
| Mill Creek | 4 feet | 7 feet |
| East River Abutment | 8 feet | 3 feet |

| | Depth of Excavation. | Depth of Water. |
|---|---|---|
| West River Abutment | 8 feet | 2 feet |
| River Piers | 4 feet | 8 feet |

PILE FOUNDATIONS

Considering the depths of excavation and dewatering costs involved with spread footings, it is probable that pile foundations would be more economical. The maximum recommended foundation levels for the piles correspond to those recommended for spread footings. Because of scour considerations, the soil above recommended foundation elevations cannot be relied upon to contribute to either the vertical or horizontal support of the structure. Therefore, the piles must be end bearing and must be designed as unsupported columns.

Most types of end bearing piles could be utilised. The piles would have to be advanced to minimum depths corresponding to the foundation elevations established from scour considerations, and then driven to refusal. It is expected that displacement piles would reach refusal within a few feet of the maximum allowable foundation levels. H piles could extend to a considerable depth.

Some difficulty may be encountered in driving piles through the more dense gravelly materials such as were encountered in borehole 4 between elevations 81 and 94. This might necessitate advancing the pile to the maximum foundation elevation by some means such as jetting or churn drilling.

The preliminary design calls for 10.3/4 inch o.d. Steel Pipe Piles. Assuming the wall thickness to be ¼ inch, and that 3000 p.s.i. concrete is used, the piles would have a maximum capacity of 87 kips. The actual bearing capacity of the piles would have to be determined from the driving records and a dynamic pile driving formula such as the Hiley formula, in conjunction with the applicable building codes.

EMBANKMENTS

The maximum height of the approach embankments is about 15 feet. To ensure the stability of the embankments it is recommended that all the topsoil be removed, as well as any very organic parts of the underlying granular material. It is recommended that the side slopes be trimmed to 2 horizontal to 1 vertical, as shown on the preliminary design. The embankment fill should be placed in layers not exceeding 9 inches in thickness and each layer should be well compacted. It is suggested that when the embankment fill has been selected, a representative sample of this fill be subjected to a compaction test. It should then be specified that at least 95% of the optimum density be obtained in the field. It is also recommended that the portion of the slopes below high water level be protected with rip rap, and the portion of slopes above high water level be protected from superficial slope erosion by encouraging grass growth.

CONCLUSIONS & RECOMMENDATIONS

1) The area of the investigation is generally covered by granular materials of various composition and density, underlain by very dense silts and tills.

2) The river level at the time of the investigation was at elevation 92. The creek level was at elevation 91. The ground water level in the surrounding area was at slightly above river level.

3) The foundation elevations are dependent on the amount of

scour that may be expected. Therefore, a hydrologic study should be carried out.

4) The following recommended maximum foundations are based on general scour considerations, as discussed elsewhere.

| | |
|---|---|
| Mill Creek Bridge Abutments | Elevation 77 |
| Nith River East Abutment | Elevation 85 |
| Nith River West Abutment | Elevation 77 |
| Nith River Piers | Elevation 77 |

5) The structures may be founded on spread footings at the above elevations using an allowable bearing pressure of 3 tons per square foot for the east abutment, and 2 tons per square foot for the other foundations. Each footing should be continuous to minimize possible differential settlement.

6) The allowable bearing pressures for continuous spread footings have been kept conservative to minimize differential settlement within the continuous span river structure, and the rigid creek structure.

7) Dewatering and excavation for spread footings will involve extensive dewatering systems. Consideration may therefore be given to piled foundations.

8) End bearing piles designed as unsupported columns would have to be advanced to the maximum foundation elevations and then driven to refusal.

9) Dewatering in sand and gravel to depths below the present water level of up to 8 feet would be required for the pile caps placed at the elevations shown for the preliminary design.

10) The topsoil and any very organic granular material beneath the proposed embankment should be removed.

11) The embankment fill should be well compacted, and the proposed fill material be tested to enable filling

specifications to be made.

12) The slopes of the embankments should be trimmed to 2 : 1, and protected below high water level by rip rap, and above high water level by some means such as grass growth.

FRANKI OF CANADA LIMITED.

A.W. Millard, P.Eng.,
Divisional Soils Engineer.

AWM/mn.

# APPENDIX

Boring Records

Drawing 1241-2 - Boring Plan & Soil Stratigraphy

Figures - Laboratory Testing

# BORING RECORDS

The boring records on the following pages give a comprehensive picture of the soils information obtained from each boring. The explanation of the various headings is given below:

## SOIL PROFILE

Under this heading is given a short form description of the various soils encountered. The elevations given are referred to the Datum shown on the general heading.

In the description of the soil, the consistency of cohesive soils and the relative density of non-cohesive soils are described by the following terms:

| Consistency | Shear Strength pounds/sq.foot | Relative Density | Standard Penetration blows/foot |
|---|---|---|---|
| Very soft | less than 250 | Very loose | less than 4 |
| Soft | 250 -- 500 | Loose | 4 -- 10 |
| Firm | 500 -- 1000 | Compact | 10 -- 30 |
| Stiff | 1000 -- 2000 | Dense | 30 -- 50 |
| Very stiff | 2000 -- 4000 | Very dense | more than 50 |
| Hard | more than 4000 | | |

## STANDARD PENETRATION RESISTANCE
## DYNAMIC PENETRATION RESISTANCE

Under this heading are shown graphically the penetration resistances as a function of blows per foot. The dynamic penetration resistance is obtained by the continual driving of a standard 2-inch 60-degree cone and observing the blows required for each foot of penetration. The standard penetration resistance is obtained during driving of a standard 2-inch drive or split-spoon sampler and observing the blows required to advance the sampler 1 foot. For both tests the driving force consists of a 140-pound hammer dropping 30 inches.

## SAMPLES

Under this heading the samples taken are plotted to vertical scale in the first column. The second column shows the type of sampler used. The fourth column indicates the recovery as the percentage of the length over which the sampler is driven.

## LABORATORY TESTS

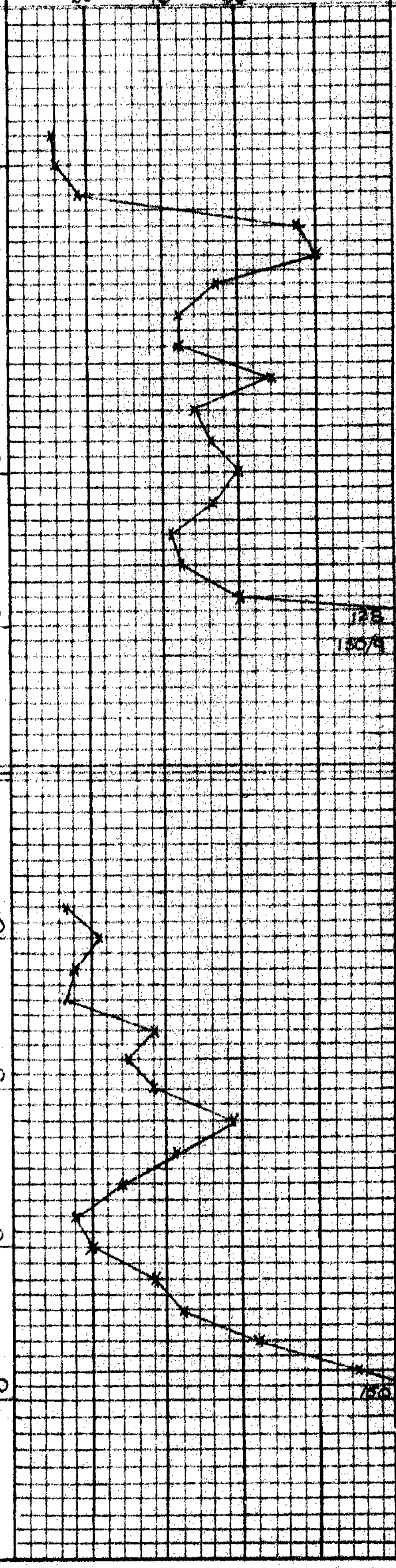
When laboratory tests on samples obtained are carried out, the results are given on the right hand side of the form. The symbols used for individual tests are explained in the legend.

**FRANKI OF CANADA LTD.** **BORING RECORD**

CONTRACT PC 1242 PT ~~BORING~~ 5 & 6 BORING DATE OCT. 28.63

DATUM LOCAL DIAM. — HAMMER 140 LBS. DROP 30 IN.

| SOIL PROFILE: DESCRIPTION | STRAT. PLOT | ELEV DEPTH | ELEV. SCALE | STANDARD PENETRATION RESISTANCE ●—● / DYNAMIC PENETRATION RESISTANCE x—x / BLOWS PER FOOT 20 40 60 80 | SAMPLES: DEPTH | TYPE | NUMBER | % RECOVERY |
|---|---|---|---|---|---|---|---|---|
| PT 6 | | | | | | | | |
| GROUND LEVEL | | 97.0 | | | | | | |
| | | | 95.0 | | | | | |
| | | | 90.0 | | | | | |
| | | | 85.0 | | | | | |
| | | | 80.0 | 125 | | | | |
| END OF PENETRATION | | 79.25<br>17.75 | | 150/9 | | | | |
| PT 5 | | | | | | | | |
| GROUND LEVEL | | 97.0 | | | | | | |
| | | | 95.0 | | | | | |
| | | | 90.0 | | | | | |
| | | | 85.0 | | | | | |
| END OF PENETRATION | | 80.0<br>17.0 | 80.0 | 150 | | | | |



**SAMPLE TYPES**

AS AUGER SAMPLE
DO DRIVE OPEN
DF DRIVE FOOT VALVE
SO SLEEVE OPEN
SF SLEEVE FOOT VALVE
TO THIN WALLED OPEN
TP THIN WALLED PISTON
WS WASHED SAMPLE

RC ROCK CORE
$K_F$ FIELD PERMEABILITY TEST
GROUND WATER LEVEL AT TIME OF BORING
REMARKS

**FRANKI OF CANADA LTD.** **BORING RECORD**

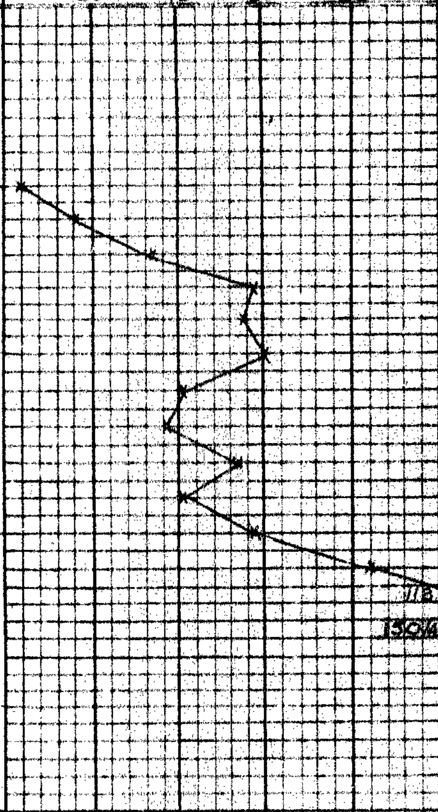
CONTRACT PC 1242 PT ~~BORING~~ 7 & 8 BORING DATE NOV 7.63

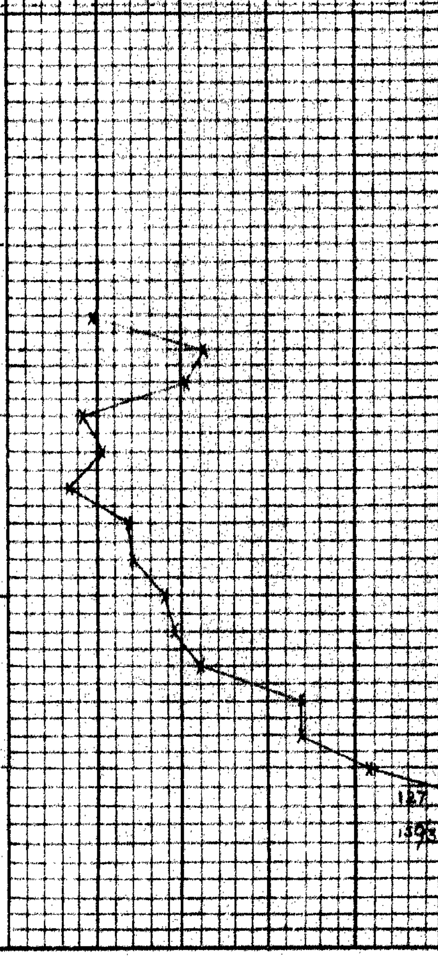
DATUM LOCAL DIAM. ——— HAMMER 140 LBS. DROP 30 IN

STANDARD PENETRATION RESISTANCE ●—●
DYNAMIC PENETRATION RESISTANCE x—x
BLOWS PER FOOT: 20, 40, 60, 80

SAMPLES: DEPTH | TYPE | NUMBER | % RECOVERY

| SOIL PROFILE – DESCRIPTION | STRAT. PLOT | ELEV / DEPTH | ELEV. SCALE |
|---|---|---|---|
| PT 7 EAST PIER | | | |
| RIVER LEVEL (OCT 24 63) | | 92.0 | |
| RIVER BOTTOM | | 90.5 / 1.5 | 90.0 |
| | | | 85.0 |
| | | | 80.0 |
| END OF PENETRATION | | 77.65 / 14.35 | |
| | | | 75.0 |
| PT 8 WEST PIER | | | |
| RIVER LEVEL (OCT 24 63) | | 92.0 / 0.0 | |
| | | | 90.0 |
| RIVER BOTTOM | | 89.0 / 3.0 | |
| | | | 85.0 |
| | | | 80.0 |
| | | | 75.0 |
| END OF PENETRATION | | 73.7 / 18.3 | |




**SAMPLE TYPES**

AS AUGER SAMPLE
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TP THIN WALLED PISTON
WS WASHED SAMPLE

RC ROCK CORE
$K_F$ FIELD PERMEABILITY TEST
GROUND WATER LEVEL AT TIME OF BORING
REMARKS

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GRAIN SIZE DISTRIBUTION

APPENDIX

FIGURE 1

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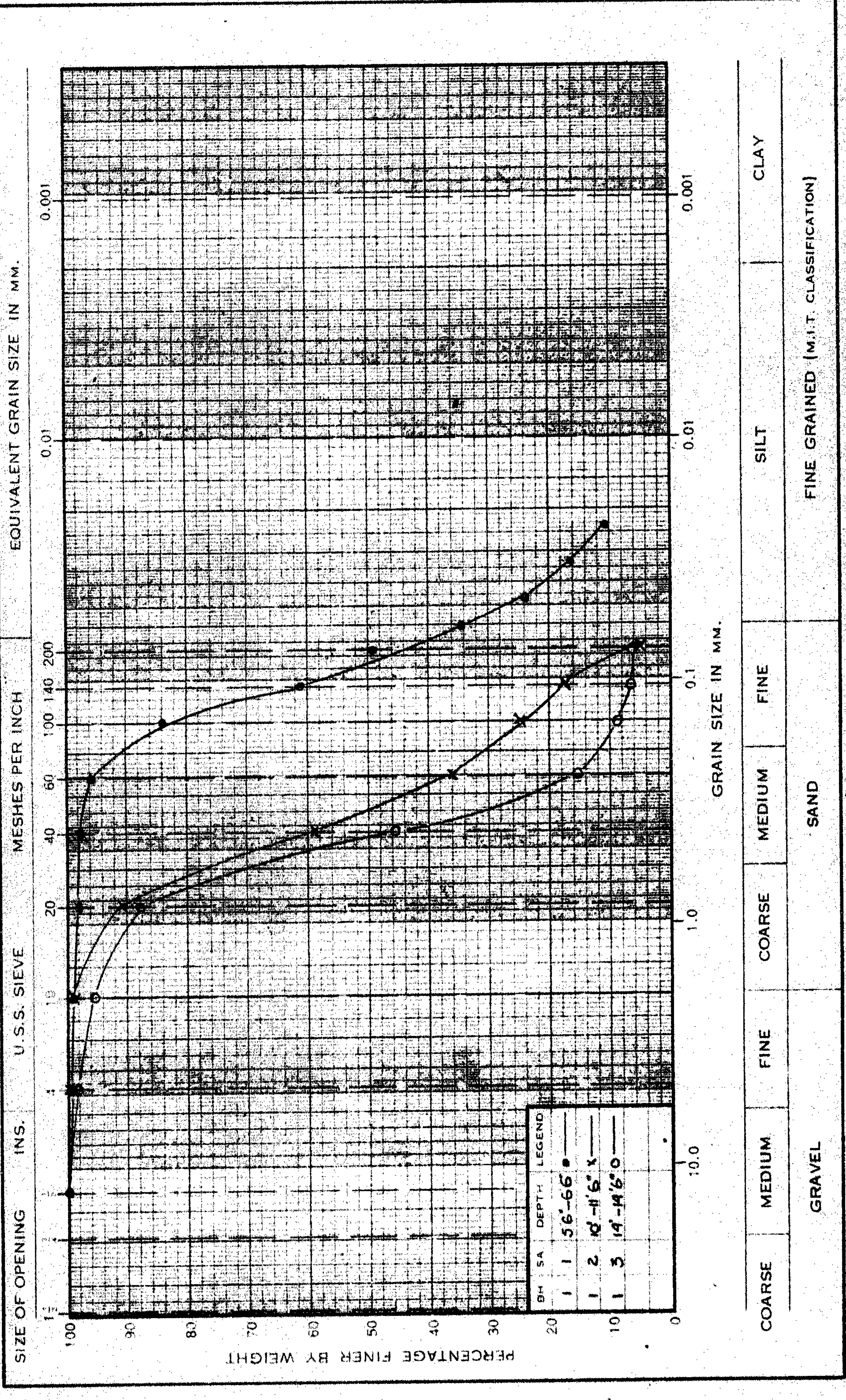


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FIGURE 2

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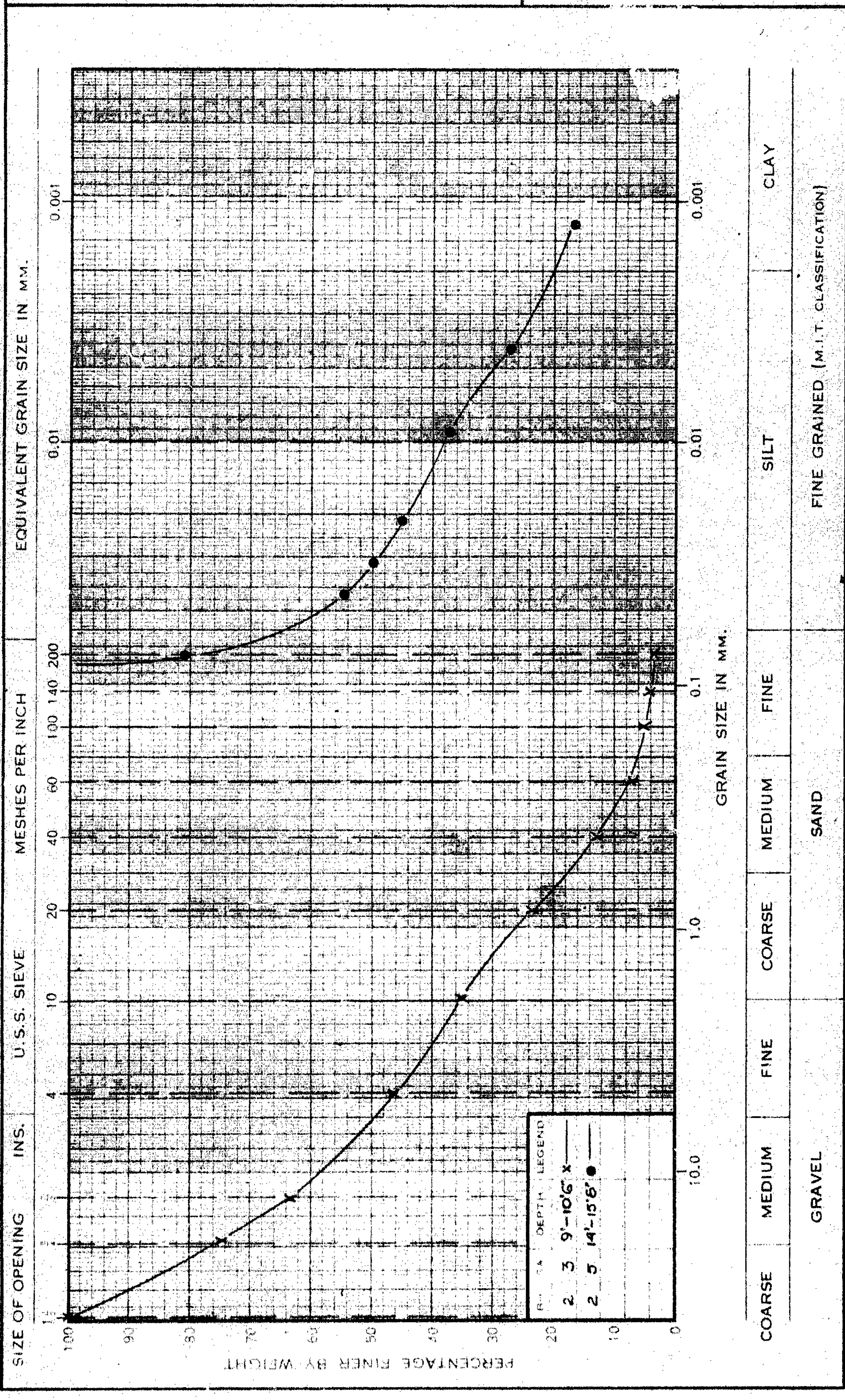


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**APPENDIX**

**FIGURE** 3

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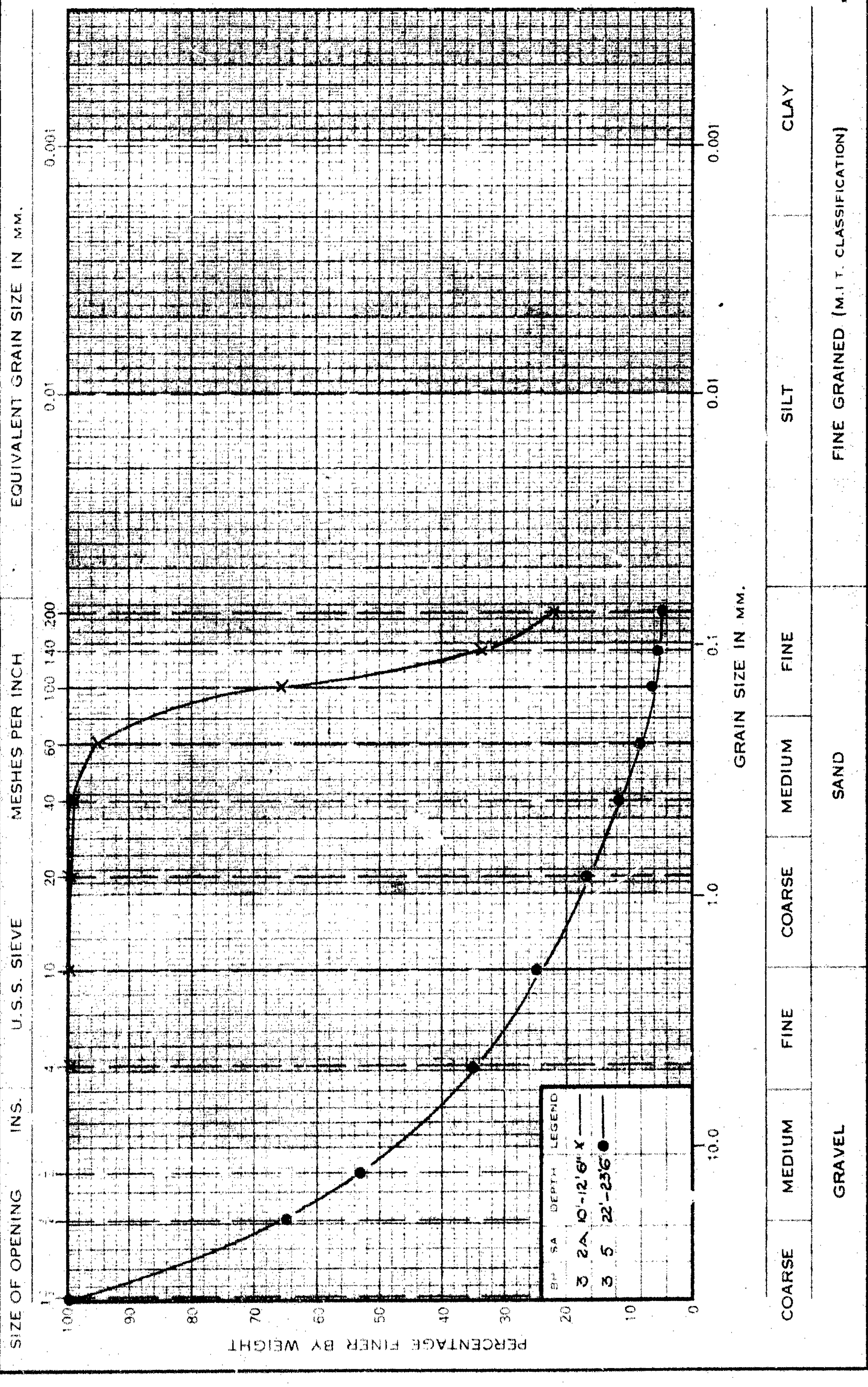


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**APPENDIX**

**FIGURE** 4

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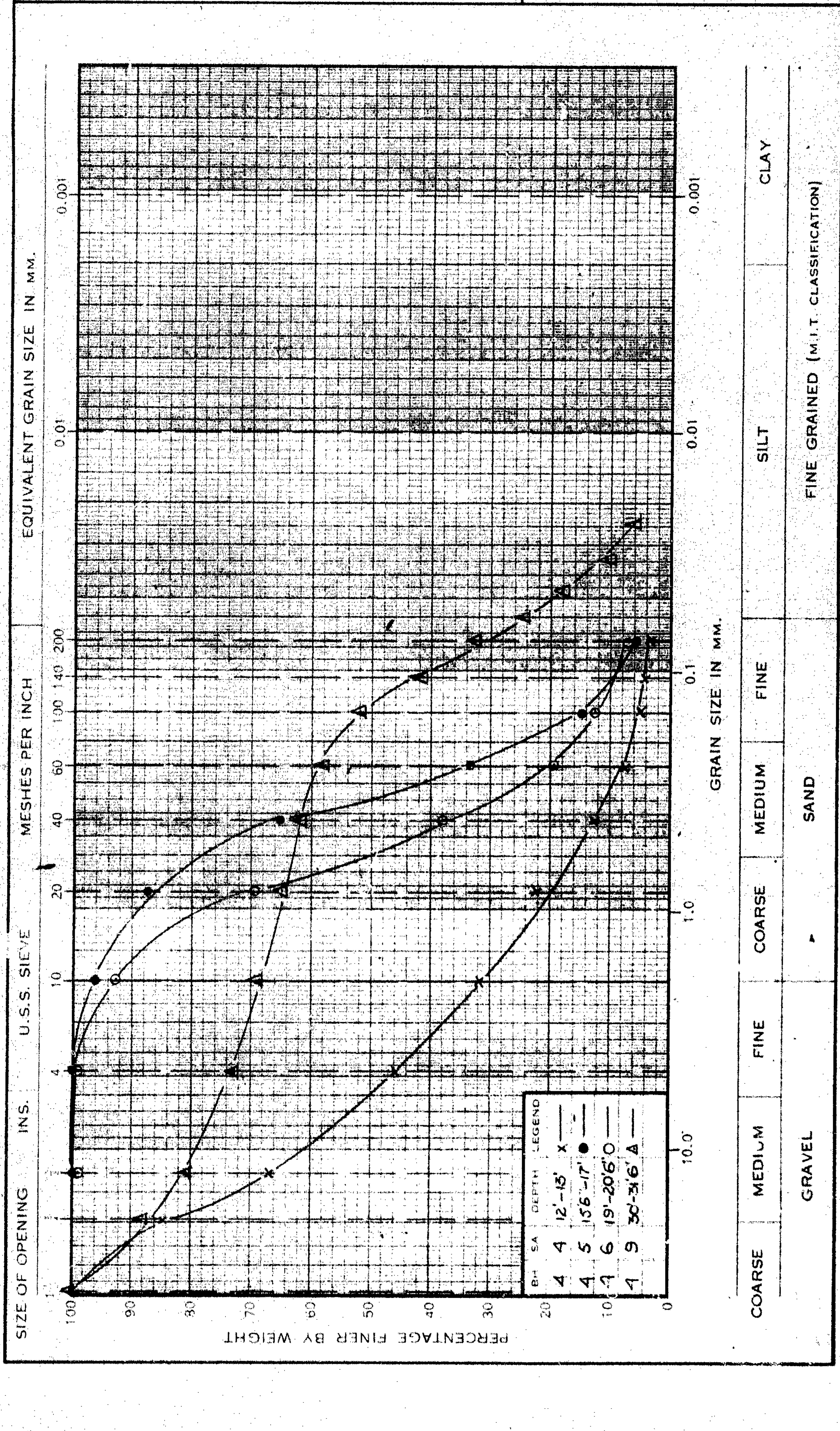


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FIGURE 5

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