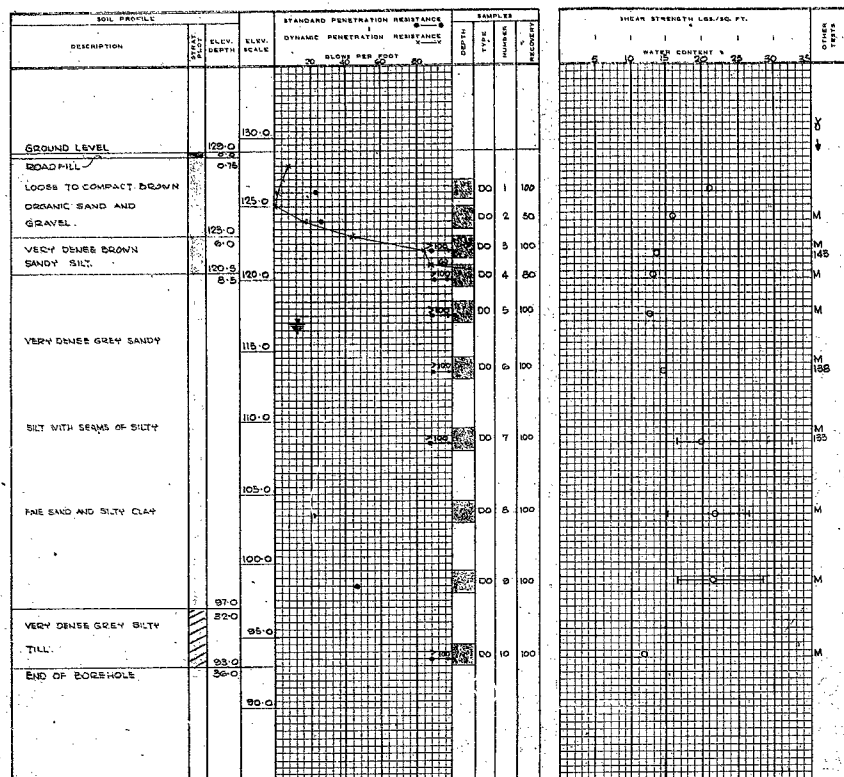


63-F-219M

NITH RIVER BRIDGE

CONTRACT PC 1188 BORING 1 BORING DATE MARCH 29 1963
 DATUM LOCAL DIAM. 4" HAMMER 140 LBS. DROP 30 IN



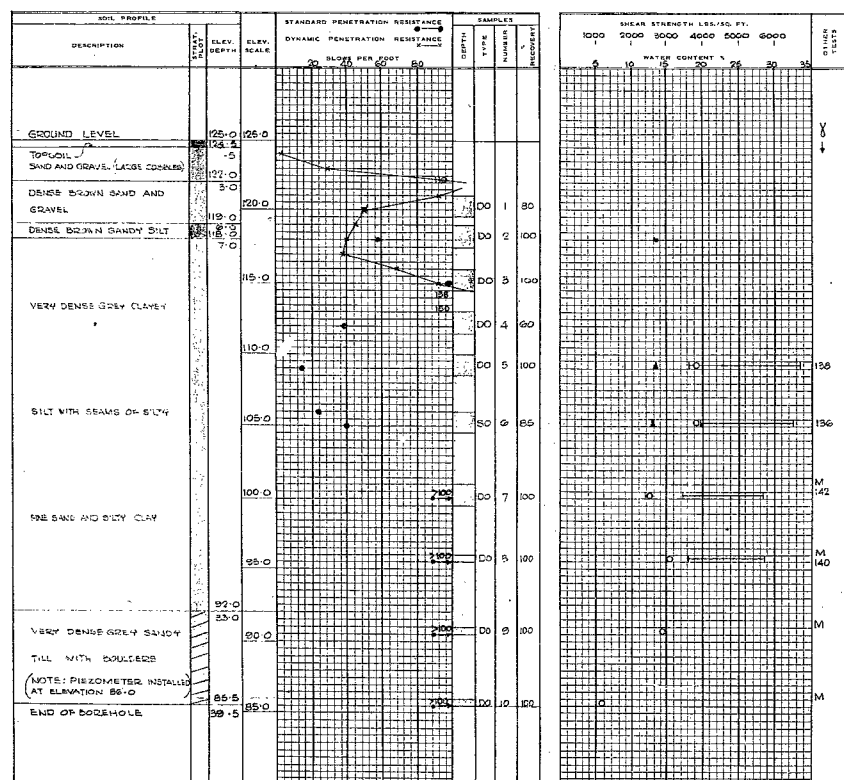
SAMPLE TYPES

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

RC ROCK CORE
 K_f FIELD PERMEABILITY TEST
 AT TIME OF BORING
 REMARKS

WATER CONTENT
 UNSATURATED TRIAXIAL
 IN SITU VANE
 MECHANICAL ANALYSIS
 PERMEABILITY FIELD
 PERMEABILITY LAB.
 RELATIVE DENSITY
 SPECIFIC GRAVITY
 CONSOLIDATION

CONTRACT PC 1188 BORING 4 BORING DATE MARCH 29 1963
 DATUM LOCAL DIAM. 4" HAMMER 140 LBS. DROP 30 IN



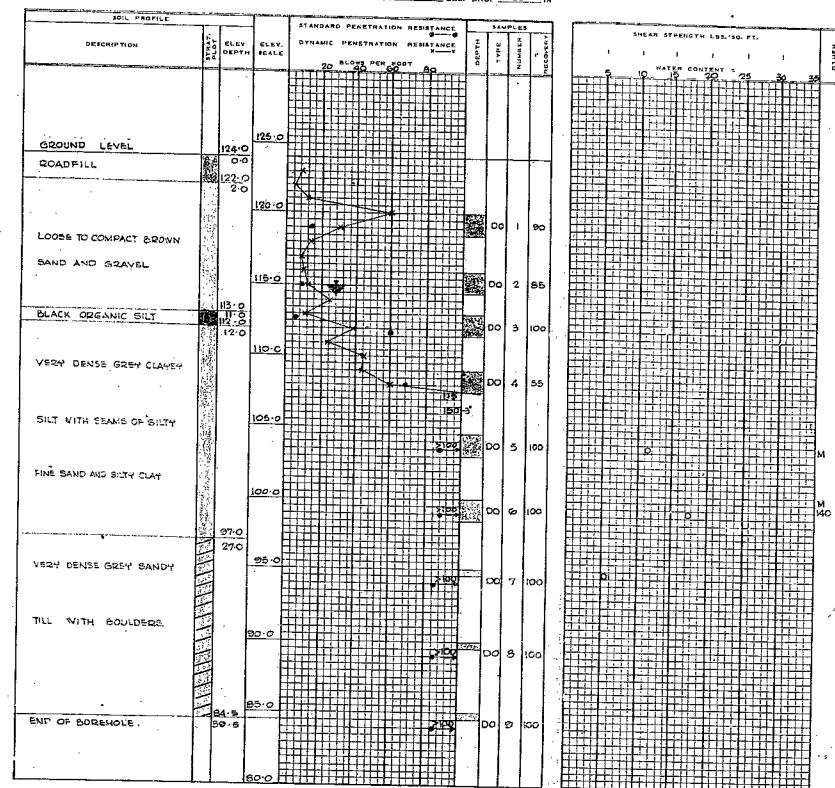
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 DP DRIVE FOOT VALVE
 SO SLEEVE OPEN
 SF SLEEVE FOOT VALVE
 TP THIN WALLED OPEN
 TS THIN WALLED PISTON
 WS WASHED SAMPLE

RC ROCK CORE
 K_f FIELD PERMEABILITY TEST
 AT TIME OF BORING
 REMARKS

WATER CONTENT
 UNSATURATED TRIAXIAL
 IN SITU VANE
 MECHANICAL ANALYSIS
 PERMEABILITY FIELD
 PERMEABILITY LAB.
 RELATIVE DENSITY
 SPECIFIC GRAVITY
 CONSOLIDATION

CONTRACT PC 1188 BORING 5 BORING DATE MARCH 28, APRIL 5 1963
 DATUM LOCAL DIAM. 4" HAMMER 140 LBS. DROP 30 IN



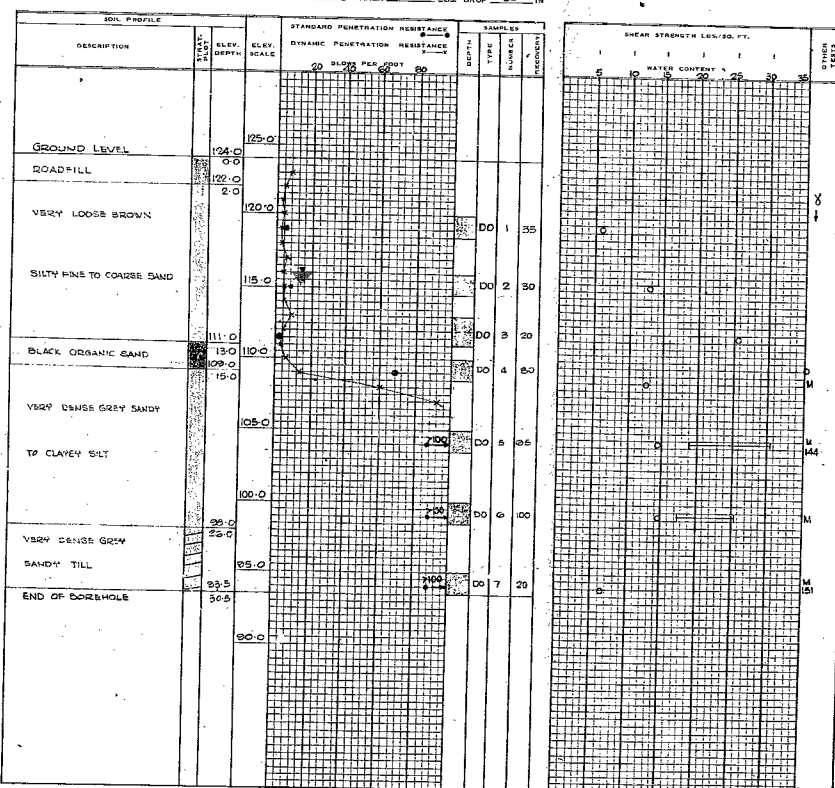
SAMPLE TYPES

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 DO DRIVE OPEN
 DP DRIVE FOOT VALVE
 SO SLEEVE OPEN
 SF SLEEVE FOOT VALVE
 TP THIN WALLED OPEN
 TS THIN WALLED PISTON
 WS WASHED SAMPLE

RC ROCK CORE
 K_f FIELD PERMEABILITY TEST
 AT TIME OF BORING
 REMARKS

WATER CONTENT
 UNSATURATED TRIAXIAL
 IN SITU VANE
 MECHANICAL ANALYSIS
 PERMEABILITY FIELD
 PERMEABILITY LAB.
 RELATIVE DENSITY
 SPECIFIC GRAVITY
 CONSOLIDATION

CONTRACT PC 1188 BORING 8 BORING DATE APRIL 4, 1963
 DATUM LOCAL DIAM. 4" HAMMER 140 LBS. DROP 30 IN

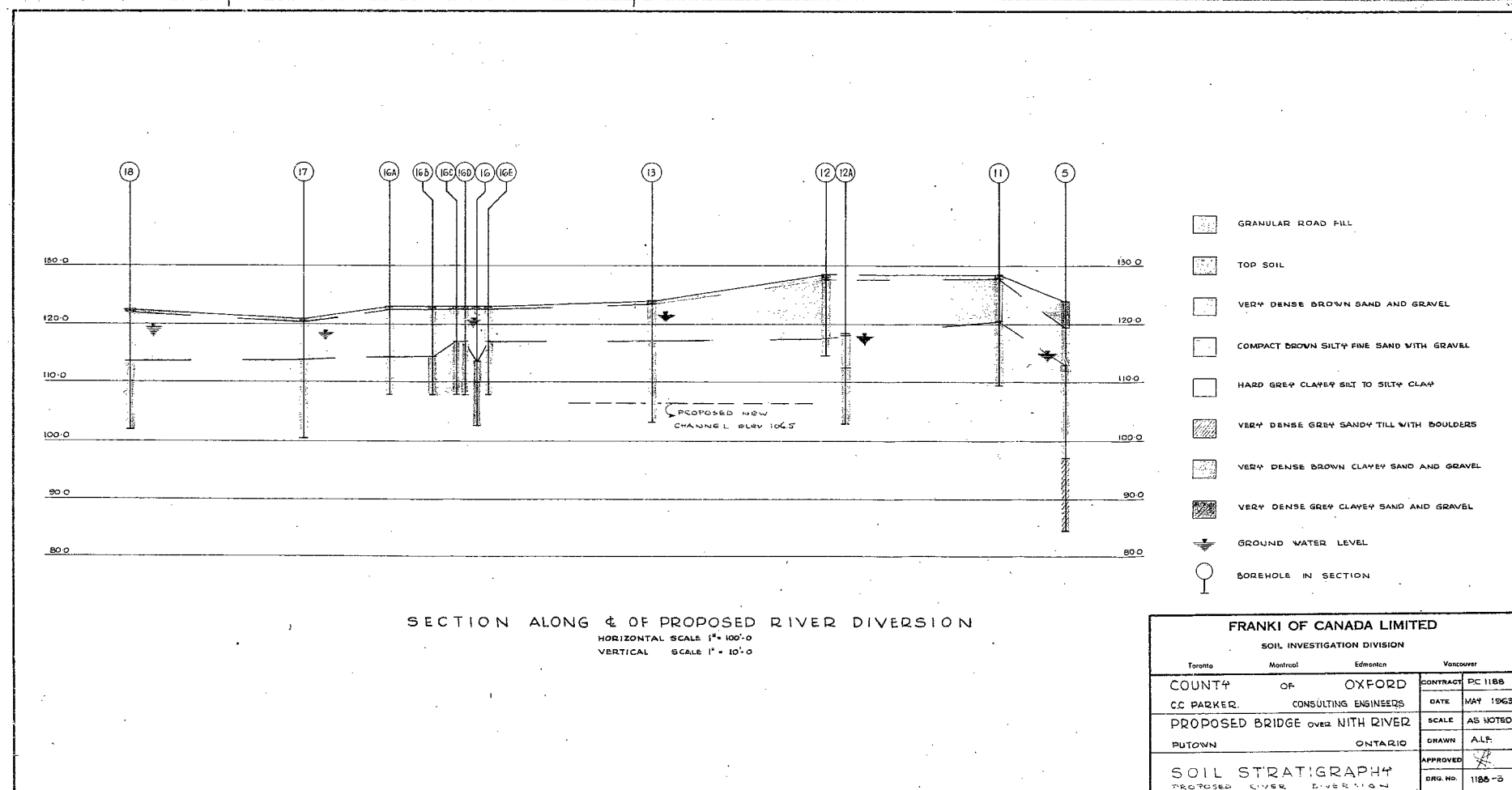
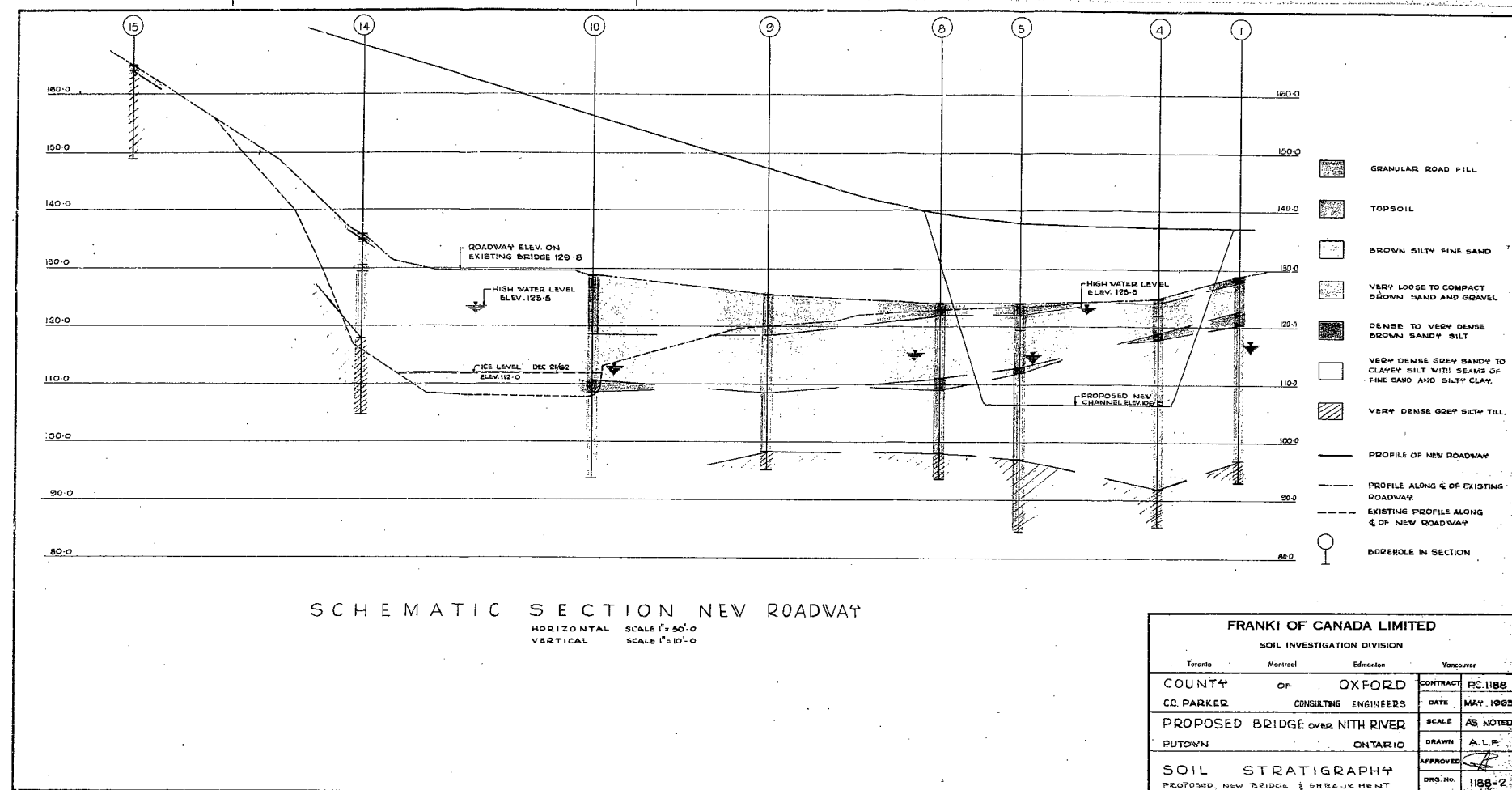


SAMPLE TYPES

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

RC ROCK CORE
 K_f FIELD PERMEABILITY TEST
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 UNSATURATED TRIAXIAL
 IN SITU VANE
 MECHANICAL ANALYSIS
 PERMEABILITY FIELD
 PERMEABILITY LAB.
 RELATIVE DENSITY
 SPECIFIC GRAVITY
 CONSOLIDATION



FRANKI OF CANADA, LIMITED
SOIL INVESTIGATIONS

BA 1660
23-193

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

R E P O R T

to

COUNTY OF OXFORD

on

63-F-219M

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED BRIDGE OVER NITH RIVER

TOWNSHIPS OF BLENHEIM & SOUTH DUMFRIES

ONTARIO

Distribution: 6 copies -- C. C. Parker and Associates Ltd.
Consulting Professional Engineers

PC 1188

May 9, 1963

I N D E X

Introduction	page 1
Description of the Site	page 1
Summarized Soil Conditions	page 2
Summarized Water Conditions	page 3
Discussion	
a. General	page 3
b. Bridge Foundations	page 4
c. Stability of Bridge Abutments	page 6
d. Proposed Embankment	page 8
e. Proposed River Diversion	page 9
Conclusions and Recommendations	page 10

Appendix I

Procedures
Soil Conditions
Water Conditions
Boring Records

Drawing 1188-1	-	Boring Plan
Drawing 1188-2	-	Soil Stratigraphy along new roadway
Drawing 1188-3	-	Soil Stratigraphy along diversion

Appendix II - Figures - Laboratory Testing

INTRODUCTION

Franki of Canada Limited has been retained by C. C. Parker and Associates Limited, Consulting Professional Engineers, on behalf of the County of Oxford, by letter of April 1, 1963, to carry out a soil investigation at the site of the proposed new bridge over the Nith River, near a community locally known as Putown, about 3 miles west of Paris, Ontario.

The object of the investigation was to determine and interpret the soil conditions at the site as they affect the foundation of the new bridge, the stability of the approach embankment and the excavation of a proposed river diversion.

The borehole locations and detailed Boring Records are given in Appendix I to this report. The results of laboratory testing are shown on the Boring Records and on the Figures of Appendix II.

A description of the procedures followed and detailed accounts of the soil and water conditions encountered are given in Appendix I.

DESCRIPTION OF THE SITE

The site is located about 3 miles west of Paris and about 1.25 miles north of Falkland, Ontario. The valley is locally widened, and the Nith River flows in the northern part of the valley, following the toe of the slope. The slope is locally very steep and in the vicinity of the site several slope failures have occurred and are presently occurring, leaving a vertical face. At the site, the slope is wooded and stable.

The present river crossing consists of an old steel structure. The north approach is cut in the slope, the southern approach is on fill, crossing the flood plain.

SUMMARIZED SOIL CONDITIONS

The area of the proposed bridge is covered by up to 2 feet of road fill, underlain by a stratum of brown sand and gravel of erratic density and with organic inclusions, extending to a depth ranging from about 5 to 10 feet. The sand and gravel are underlain by grey, generally very dense, sandy to clayey silt with layers or lenses of silty fine sand and silty clay. Below about elevation 97, the silt is underlain by a stratum of grey very dense silty to sandy till.

The area of the existing bridge has similar soil conditions. To the north of the bridge the granular stratum is thin or non-existent and the elevation of the till stratum is higher.

The area of the proposed river diversion is covered by about 8 inches of topsoil, underlain partly by very dense clean sand and gravel and partly by compact silty fine sand with gravel. The granular strata extent to elevations ranging from 120 to 115 and are underlain by hard grey clayey silt to silty clay, except in borehole 16 where the granular material overlies very dense clayey sand and gravel.

Detailed descriptions of the soil conditions encountered and the results of laboratory testing are given in Appendix I.

SUMMARIZED WATER CONDITIONS

The water level of the Nith River at the time of the investigation was subject to frequent and rapid fluctuations. The average river level was at elevation 112.5. To the south of the existing bridge the ground water level ranged from elevation 113 in borehole 10 to elevation 117 in borehole 1.

In the area of the proposed river diversion the ground water level ranged from elevation 117 in borehole 12 A to elevation 121 in borehole 13. The ground water in this area is possibly perched.

More detailed accounts of the ground water conditions are given in Appendix I.

DISCUSSION

a. General

At the present time, the Nith River follows the toe of the steep slope. In order to accommodate the required gradient of the new road and presumably also to keep the river away from the railway tressel west of the present bridge, it has been proposed to divert the Nith River into a new channel. In this way, it will be possible to construct the new bridge in the dry. The new channel will then be opened and the embankment can be placed.

The proposed new bridge will be a three-span continuous steel structure, about 260 feet long. The centre span will be 108 feet long, the approach spans 76 feet. The width of the bridge deck will be about 35 feet. At the present time, no details on the substructure or on loading are available.

Assuming that reinforced concrete abutments and piers will be used, the abutment and pier loads could be of the order of 600 and 400 tons, respectively.

The proposed river diversion will be excavated to about elevation 107 and will be about 250 feet wide. A total excavation of the order of 165,000 cubic yards will be required.

The proposed gradient of the proposed new road is 6 percent. In order to accommodate this gradient, considerable fill is required of the order of 120,000 cubic yards. The maximum height of the embankment will be about 52 feet in the area of the present river crossing.

b. Bridge Foundations

The bottom of the proposed river diversion is at elevation 106.5. The piers will be founded below the bottom of the diversion and one factor that requires consideration is the amount of scour that may be expected. Besides being related to the soil conditions, the scour is also dependant on the hydrologic properties of the drainage basin. Experience within the relatively short period of the investigation has shown, that the river is subject to very rapid changes in discharge. It is therefore recommended that the hydrology of the area be studied. When it becomes apparent that scour may be a problem it may alternatively be considered to line the channel locally.

The soil conditions at the pier locations are determined by boreholes 4 and 5 and penetration tests 3 and 6. Elevation 106.5 is in the grey silt stratum. At and below this elevation in borehole 5 the silt is very dense. In borehole 4, however, the silt contained relatively soft clay layers or lenses

and relatively loose silty fine sand layers or lenses. Under normal conditions, using nominal protection against ice and scour, the maximum foundation level would be at about elevation 102. It is considered that this elevation is suitable for the foundation of pier footings. For a structure of the type proposed, i.e. a continuous steel structure, the foundation considerations include not only the maximum allowable bearing capacity, but also differential settlement. It is not known, how much differential settlement can be tolerated, but it is estimated that it would be no more than $L/1000$, where L is the span. Using this expression, a differential settlement of 1.5 inch could be allowed between the piers. Based on the above considerations, it is recommended that the pier foundations be placed at or below elevation 102, using an allowable bearing pressure of 6000 pounds per square foot. It is further recommended that in order to minimize differential settlement within the pier, the pier be founded on one continuous footing. Using the above bearing pressure and the estimated maximum pier load of 600 tons, a footing of the order of 200 square feet would be required, which is probably well within design dimensions. Under the above conditions, total settlement should be small and differential settlement should be well within allowable limits.

The soil conditions at the proposed abutment locations are determined by boreholes 1 and 8 and penetration tests 2 and 7. At the location of borehole 8, loose granular material with organic inclusions extends to about elevation 109. This material is not suitable for foundation purposes. Below elevation 109, footings may be placed using an allowable bearing pressure of 6000 pounds per square foot with the same settlement considerations as before. Using the estimated abutment load of

400 tons, a footing area of about 150 square feet would be required, which is probably within design dimensions. On the south side, in the area of borehole 1, loose granular material extends to elevation 120. Below this elevation is very dense silt. At about elevation 100, lesser densities were obtained, but these densities are probably conservative and should in addition have no significant influence if footings are kept high. It is therefore recommended that the south abutments be founded at about elevation 119, under an allowable bearing pressure of 6000 pounds per square foot.

It must be re-emphasized at this point that the allowable bearing pressures given above are on the conservative side. They have been kept low because of the settlement considerations that must be given to a continuous structure. In addition, they would probably be within the anticipated design dimensions.

c. Stability of Bridge Abutments

No design information on the abutments is at present available. They may be of the closed front type, or of the spill-through type. As discussed under "Soil Conditions" in Appendix I, the silt stratum contains layers or lenses of relatively soft silty clay. The stability of the abutments was examined under the assumption that the clay layers would be continuous. In the present design, a channel side slope of 1 vertical to 1-3/4 horizontal has been indicated. Using this slope and the approximate abutment location within the slope, a factor of safety against a sliding wedge type of failure through a clay layer is of the order of 3. At the time of the investi-

gation, it was evident that the river is subject to very rapid and significant changes in water level. Considering the case of rapid draw-down in the sliding wedge analysis, the factor of safety is reduced by about 20 percent.

Using the same slope and the approximate location of the abutment, and considering a circular type of failure, the factor of safety obtained is of the order 1.0. This value is obtained using an angle of frictional resistance of 30 degrees and the abutment footing loaded under 6000 pounds per square foot. Both assumptions are probably conservative. Considering the eccentricity of the abutment load and the probable size of the loaded area, the actual evenly distributed load would probably be more of the order of 4000 to 4500 pounds per square foot. The angle of frictional resistance, depending partly on the quality of the proposed embankment, could probably be safely taken as 35 degrees. The actual factor of safety would, therefore, be closer to 1.3, which is within allowable limits.

However, since no information on the bridge design is at present available, it is tentatively recommended that the channel slope be taken not steeper than 1 vertical to 2 horizontal and that the edge of the abutment footing be not closer than 12 feet from the slope surface. It is further recommended that when the design details become available, we be supplied with a detailed drawing and loading information, so that a final analysis of the abutment stability can be made.

It is also recommended that the slope in the area of the abutments be protected against both toe and surface erosion. Protection against scour at the toe may be obtained by rip rap. Surface protection can be achieved by

stone or by other means.

d. Proposed Embankment

The maximum height of the proposed embankment is about 52 feet in the area of the present river crossing. To the north of the crossing the site is covered by dense silt and dense till. It has been assumed that the river bed is underlain by similar material but it is recommended that this be confirmed in the summer, when the river level is low, or after the river has been diverted. To the south of the crossing the site is covered by roadfill, underlain by granular material of erratic and generally loose relative density, containing organic layers and inclusions, probably a flood plain deposit. Outside the present road this material occurs at the surface.

To ensure the stability of the embankment, it is recommended that all topsoil be removed, as well as the very organic parts of the underlying granular material. It is further recommended that the side slopes of the embankment be trimmed to 1 vertical to 2 horizontal. Under normal circumstances a steeper slope of 1 to 1-1/2 would be stable, however, because soil conditions are bound to be erratic in a flood plain, the shallower slope is recommended. 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 percent of the optimum density be obtained in the field. It is also recommended that surficial slope erosion be prevented by providing drainage of the roadway and encouraging grass growth on the slope.

The slope of the valley is about 80 feet high and locally very steep. Surface failures have occurred and are occurring in the immediate vicinity of the existing bridge. It is therefore recommended that where the proposed embankment adjoins the existing slope, this slope be filled to obtain a gradual change into the embankment. This should stabilize the slope and prevent local failures which in turn could trigger embankment failure.

e. Proposed River Diversion

The proposed new channel will be excavated to about elevation 107 or to a maximum depth of about 20 feet. About 50 percent of the excavation will be in granular material which probably contains boulders. The remainder of the excavation will be in hard clayey silt to silty clay. The upper granular material is water bearing. The average water level is at elevation 120 and it is believed that this water level may be perched on the underlying silt stratum. If it is replenished continuously however, it will complicate excavation. It is therefore suggested the site be drained prior to general excavation, by the provision of drainage trenches.

Although the quantity of excavated material is only about 15 percent of the quantity required for the embankment fill, attention was given to use this material. Figure 9 of Appendix II, shows the grainsize distribution curves for the granular material and for the clayey material and Figure 10 shows the compaction characteristics obtained for both materials. The maximum wet density for the granular material was 158 pounds per cubic foot at an optimum moisture content of 10 percent.

This is excellent fill material, especially since the natural moisture content of this material is close to the optimum. The maximum wet density for the clayey material was 142 pounds per cubic foot at an optimum moisture content of 11 percent. The natural moisture content was of the order of 16 percent. It will be difficult and probably impractical to dry the clay before placing. This material is very sensitive to the moisture content. Only a few percent under or over the optimum will result in unsatisfactory compaction and/or workability. This clayey material therefore, is not generally a suitable compaction material, although it could be used under suitable climatic conditions.

CONCLUSIONS AND RECOMMENDATIONS

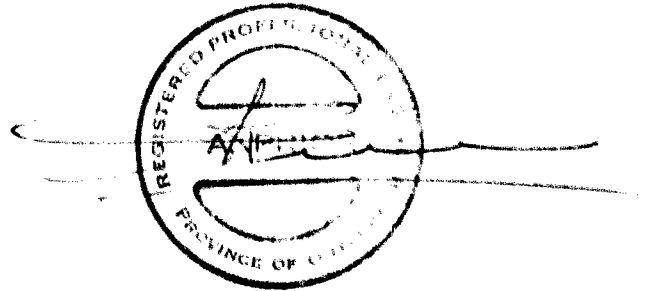
1. The area of the investigation is generally covered by granular materials of various composition and density, underlain by generally very dense silts resting on very dense silty to sandy till.
2. The average river level at the time of the investigation was at elevation 112.5. The ground water level in the surrounding area was close to this level, averaging at about elevation 115 and with a maximum elevation of 121.
3. The proposed bridge may be founded on spread footings.
4. The elevations of the pier foundations are partially determined by hydrologic and hydraulic conditions and it is recommended that a study of the area be made to determine the amount of scour that may be expected. If scour were a possible problem, the channel could be locally lined.

5. The recommended maximum foundation elevation for the piers is at elevation 102, where for the probable size of footing required, an allowable bearing pressure of 6000 pounds per square foot may be used.
6. It is recommended that each pier be founded on a continuous footing, to minimize possible differential settlement.
7. The footing for the north abutment should be placed at an elevation not higher than 109, at which elevation an allowable bearing pressure of 6000 pounds per square foot may be used.
8. The footing for the south abutment may be placed at elevation 119, using an allowable bearing pressure of 6000 pounds per square foot.
9. The allowable bearing pressures have been kept conservative to minimize differential settlement within the continuous span structure. The pressure given is expected to be well within design dimensions.
10. In the absence of structural and loading information, preliminary stability analyses were carried out on the abutments and side slopes. It is tentatively recommended that the side slope in the abutment area be made not less than 1 vertical to 2 horizontal and that the edge of the abutment footing be not closer than 12 feet from the slope surface.
11. It is recommended that when design details become available, further and detailed stability analyses be carried out.

12. The slope in the area of the abutment should be protected against toe and surface erosion.
13. For analysis of the stability of the proposed embankment, it has been assumed that the soil conditions beneath the river bottom could be interpolated from those determined on the banks. It is recommended that this be confirmed at a later date, when the river bed becomes accessible.
14. All topsoil beneath the proposed embankment should be removed and it is recommended that the side slopes be trimmed to 1 vertical to 2 horizontal.
15. The embankment fill should be well compacted and it is recommended that the proposed fill material be tested to enable filling specifications to be made.
16. The new roadway should be drained and the side slopes of the embankment protected against surface erosion.
17. A gradual change from the embankment slope into the existing valley slope should be made.
18. Excavation in the upper granular material for the proposed river diversion will probably encounter boulders. The remainder of the excavation is in hard silty clay to clayey silt.
19. The granular material in the area to be excavated is water-bearing, and it may be advisable to drain the area by a system of ditches.

20. The excavated granular material may be used as fill elsewhere.
The clayey material can only be used under suitable weather
conditions.

FRANKI OF CANADA LIMITED



A. Prior, P. Eng.,
Divisional Soils Engineer.

AP/lh

APPENDIX I

Procedures page I

Soil Conditions

a. Proposed Bridge page II

b. Proposed Embankment page IV

c. Proposed River Diversion page V

Water Conditions page VI

PROCEDURES

The fieldwork was carried out between March 27 and April 17, 1963, and consisted of 20 detailed boreholes, 7 of which with an adjacent dynamic cone penetration test and 4 additional dynamic cone penetration tests.

The locations of the borings are shown on Drawing 1188-1. Boreholes 1, 4, 5 and 8 and penetration tests 2, 3, 6 and 7 were put down at the approximate abutment and pier locations of the proposed new bridge. These borings were put down from the existing roadway to save the time and expenses involved in moving the equipment up and down the embankment.

Boreholes 9, 10, 14 and 15 were put down at the location of the existing bridge, in which area the proposed new embankment, after closing of the river channel, will have a maximum height. No boreholes were put down in the river at this time because of the present flow conditions in the river. Boreholes in the river, if required, could be conveniently put down, when the river diversion has been completed.

Boreholes 11, 12, 13, 16, 17 and 18 were put down along the centre line of the proposed diversion. When in borehole 16 conditions were encountered different from those in the other boreholes, boreholes 16 A, B, C, D and E were put down to determine where the change occurred.

A detailed log for each boring is given on the Boring Records in this Appendix. A stratigraphic cross section along the centre line of the proposed roadway is given on Drawing 1188-2 and a section along the centre line of the proposed river diversion on Drawing 1188-3.

Soil testing on samples obtained was carried out in our laboratory. The results are shown on the Boring Records and on the Figures of Appendix II. Samples remaining after testing will be stored until November 15, 1963 and then discarded unless other instructions are received.

Elevations referred to in this report are relative to a local datum consisting of the top of the deck of the existing bridge. The elevation of this datum was 129.8 which compares approximately with geodetic elevation 807.

SOIL CONDITIONS

a. Proposed Bridge

The borings in the area of the proposed bridge were carried out from the existing roadway. This part of the area is covered by 1 to 2 feet of granular fill, underlain by 5 to 11 feet of brown sand and gravel with a small silt content of the order of 10 percent. The composition of this stratum is heterogeneous. It is layered and cross-bedded and the grain-size distribution within each layer varies. A typical average grain-size distribution curve is shown on Figure 1 of Appendix II. The relative density of the stratum is also variable, as indicated by the results of the standard and dynamic penetration values obtained. The standard penetration tests gave 'N' values ranging from 2 to 50 blows per foot and the dynamic penetration values varied similarly. The higher values may have been caused by coarse sizes, the lower values by organic layers or lenses encountered within the stratum. Generally, it appears that the density of the stratum in the southern part of the site is greater than that in the northern part. In

average the relative density is considered to range from loose to compact. The natural moisture content obtained ranged from 6 to 21 percent in the clean granular part of the stratum. On organic layers within the stratum, moisture contents of 26 and 36 percent were obtained.

The sand and gravel is underlain by a stratum of silt, extending to about elevation 97. In boreholes 5 and 8, a layer of black organic sandy silt about 2 feet in thickness was encountered between the sand and gravel stratum and the silt. The silt stratum is grey in colour, except in boreholes 1 and 4, where the upper 1 to 3 feet were oxidized to brown. The stratum is not of homogeneous composition, but varies both in horizontal and vertical direction from a sandy to a clayey silt. Typical grainsize distribution curves are shown on Figures 1, 2, 3, 4, 5, 6, 7 and 8 of Appendix II. From these Figures it may be seen that generally in borehole 1, the silt is mainly sandy, whereas in the remainder of the boreholes the clay content is relatively large. The silt stratum contains occasional embedded gravel and frequently layers and seams of silty fine sand and silty clay. The thickness of these seams and layers varies from a fraction of an inch to occasionally 1 or 2 feet. These layers appear to be more predominant in borehole 1 and 4 than in boreholes 5 and 8. Their effect can be seen in the penetration values obtained on samples 4, 5 and 6 of borehole 4, and samples 8 and 9 of borehole 1. In general, the 'N' values obtained were of the order of 100 blows per foot, indicating the very dense nature of the silt. Figure 11 of Appendix II shows the natural moisture contents of the silt stratum as a function of elevation. It can be

seen that the average moisture content obtained is of the order of 13 percent. This Figure also shows the results of Atterberg limit testing on the silty clay layers within the stratum. The average liquid limit obtained was about 30 with a corresponding plasticity index of 13 at a natural moisture content of about 18 percent, close to the plastic limit. The shear strength obtained on typical samples of clay layers was about 2,600 pounds per square foot. The average moisture content of the clayey layers is about 137 pounds per cubic foot and of the silt stratum as a whole, about 141 pounds per cubic foot.

The silt stratum is underlain by a stratum of grey glacial till extending to the depth explored. This stratum was proved for a maximum depth of 13 feet in borehole 5. The till consists of sand and gravel in a matrix which varies from sandy silt to silty sand. Typical grainsize curves for this material are given on Figures 4, 5 and 8 of Appendix II. The till is very dense, as indicated by the results of standard penetration tests, which gave 'N' values well in excess of 100 blows per foot. The natural moisture content of the stratum ranged from 5 to 12 percent. The in-situ unit weight is of the order of 150 pounds per cubic foot.

b. Proposed Embankment

The area of the proposed Embankment is covered by boreholes 9, 10, 14 and 15. The soil conditions in these boreholes are generally similar to those described under (a). The brown sand and gravel in boreholes 9 and 10 is of erratic

and generally loose relative density and frequently contains seams of organic material. In borehole 14 the thickness of the sand and gravel stratum is less than south of the river, is of greater density and separated from the grey silt by a 1-foot layer of brown silty fine sand. In borehole 15 no sand and gravel and silt were encountered, but the road fill is directly underlain by very dense silty to sandy till. At the time of the investigation the Nith River had a high water level, which in addition was subject to very rapid changes. It was therefore not feasible to carry out any boreholes through the river bed in an economical way. Boreholes in this area could be carried out later this summer, or when the river diversion has been completed. For the time being, it seems reasonable to assume for the purpose of this report, that the river bed would consist of silt and/or silty till.

c. Proposed River Diversion

The area of the proposed River Diversion is covered by boreholes 11, 12, 13, 16, 17 and 18. About 8 inches of topsoil covers the site. From borehole 12 eastwards, the topsoil is underlain by brown sand and gravel which based on the 'N' values obtained, is generally in a very dense state. This material is generally free of silt sizes and contains gravel sizes up to several inches in diameter. It is probable that cobble and boulder sizes will also be encountered during excavation. From borehole 13 westwards the topsoil is underlain by brown, silty, occasionally very silty, fine sand with a variable amount of gravel sizes. 'N' values obtained in this material were of the order of 20 to 30 blows per foot, indicating the relative

density to be compact.

The granular strata except in borehole 16, are underlain by grey clayey silt to silty clay to the depth explored. This material is not unsimilar to the silt described under (a) but generally is more clayey than in the area of the proposed bridge. Based on the 'N' values obtained and on visual examination, the stratum is considered to be of hard consistency.

In borehole 16 the grey clayey silt to silty clay was not encountered. Instead, the silty fine sand is underlain by a stratum of brown, changing to grey clayey fine to coarse sand and gravel of a very dense nature. In order to check the horizontal extent of this material, boreholes 16 A to E were put down by augering only. None of these boreholes encountered similar material, however, and it is therefore concluded that the clayey sand and gravel is a very local deposit.

The granular and clayey strata will be excavated to elevation 106.5 to accommodate the proposed river diversion. With a view to using the excavated material as fill elsewhere, Standard Proctor Compaction Tests were carried out on both the brown sand and gravel and grey clayey silt to silty clay. Figure 9 of Appendix II shows the grainsize distribution curve for the sand and gravel east of borehole 12. This curve only includes the material passing the No.4 sieve. The moisture content versus density curve resulting from the compaction test on this material is given on Figure 10. It can be seen that a maximum wet density of

VII.

158 pounds per cubic foot can be obtained at an optimum moisture content of about 10 percent. This compares with a natural moisture content of 8 percent for the material tested for compaction and varying across the site from about 5 to 11 percent. The grainsize distribution curve for a typical sample of the clayey silt to silty clay is given on Figure 9 of Appendix II and the compaction curve on Figure 10. For this material a maximum wet density of 142 pounds per cubic foot at an optimum moisture content of about 11 percent was obtained. This compares with a natural moisture content of about 16 percent for the material tested.

WATER CONDITIONS

The ground water conditions were observed throughout the period of the investigation. The observations generally indicated that the water level readings were influenced by surface water as a result of weather conditions at the time of the investigation.

In the area of the proposed bridge, water level observation pipe was installed in boreholes 1, 5 and 8 and in addition a piezometer was installed in borehole 4 within the silt stratum, sealed off from the sand and gravel stratum by several compacted bentonite seals. The water level obtained in the stand pipes has been shown on the Boring Records and on the section on Drawing 1188-2. The water levels fluctuated with the river level and the relatively stabilized level at the end of the investigation ranged from elevation 117 in borehole 1 to elevation 115 in borehole 5. The piezometer in borehole 4 was flushed after installation and filled with water to de-air the

system. The water level in the piezometer did not move over a period of 10 days, indicating the very low permeability of the silt stratum.

In the area of the proposed embankment, stand pipes were installed in boreholes 10 and 14. No water was encountered in borehole 14, probably due to the very low permeability of the silt. In borehole 10, ground water was measured in the sand and gravel at elevation 113, which is, as may be expected, close to river level, which was in average at elevation 112.5 during the period of the investigation.

In the area of the proposed river diversion, ground water was encountered in the granular strata at elevations varying between elevation 117 in borehole 12 A to elevation 121 in borehole 13. These levels are from 5 to 9 feet above river level and probably represent perched water level at the time of the investigation.

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:

<u>Consistency</u>	<u>Shear Strength pounds/sq.foot</u>	<u>Relative Density</u>	<u>Standard Penetration blows/foot</u>
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 RESISTANCEDYNAMIC 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.

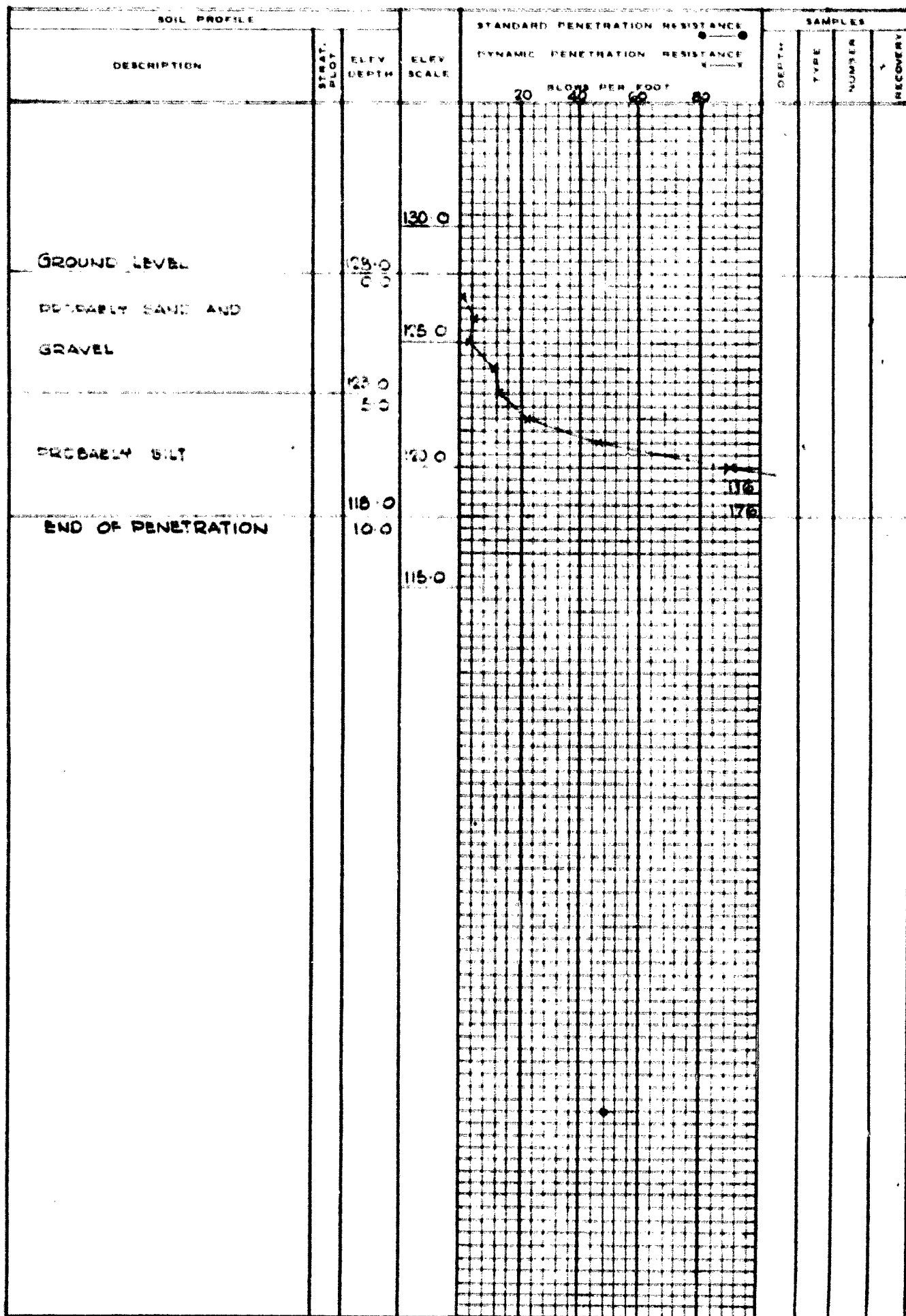
SAMPLES

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.

CONTRACT PC 1188 BORING PENETRATION TEST 2 BORING DATE MARCH 29 1963
 DATUM LOCAL DIAM. — HAMMER 140 LBS. DROP 30 IN



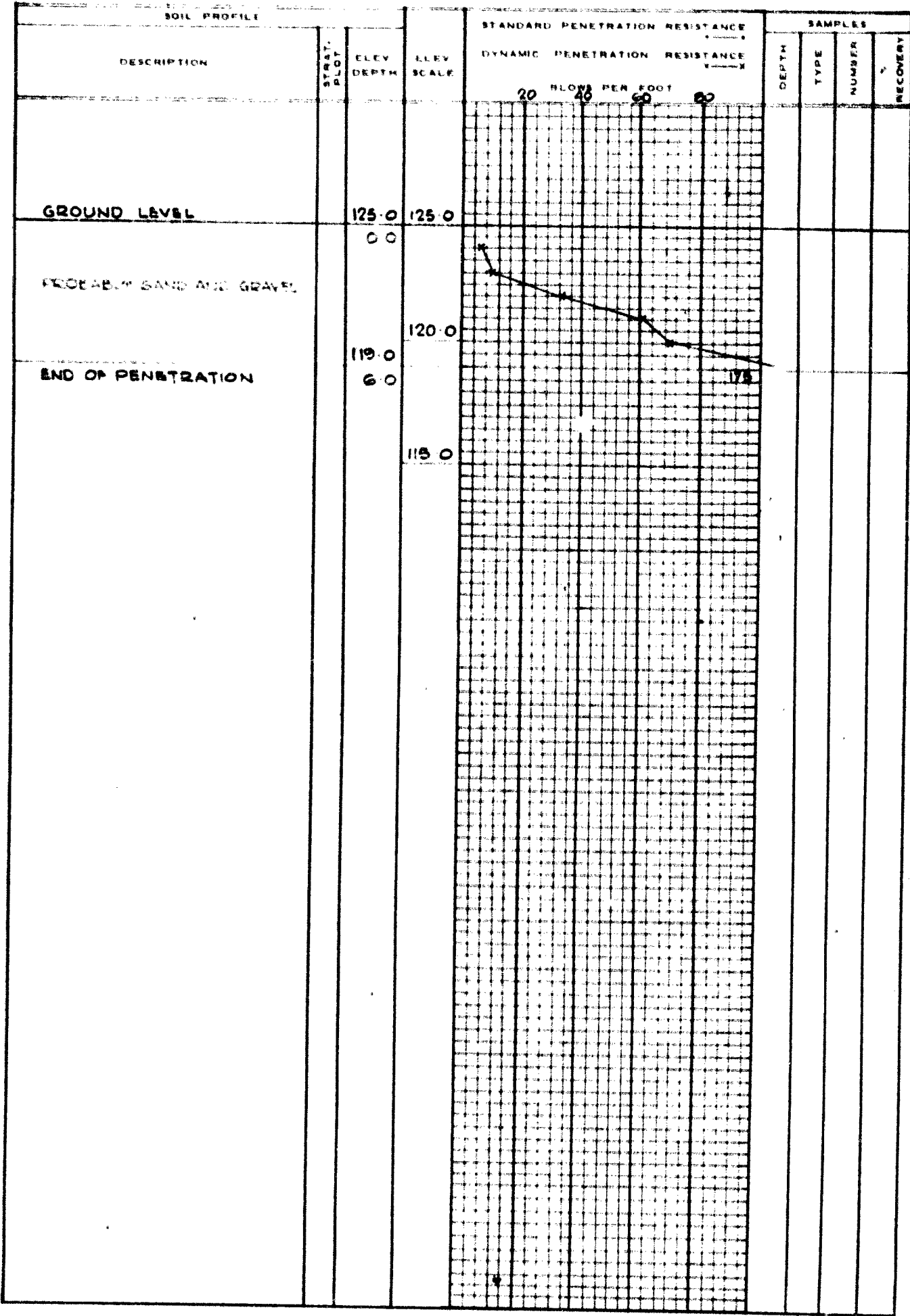
SAMPLE TYPES

AS AUGER SAMPLE
 DO DRIVE OPEN
 OF 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

CONTRACT PC 1188 BORING PENETRATION TEST 8 BORING DATE MARCH 29 1963
DATUM LOCAL DIAM. _____ HAMMER 140 LBS. DROP 2 IN



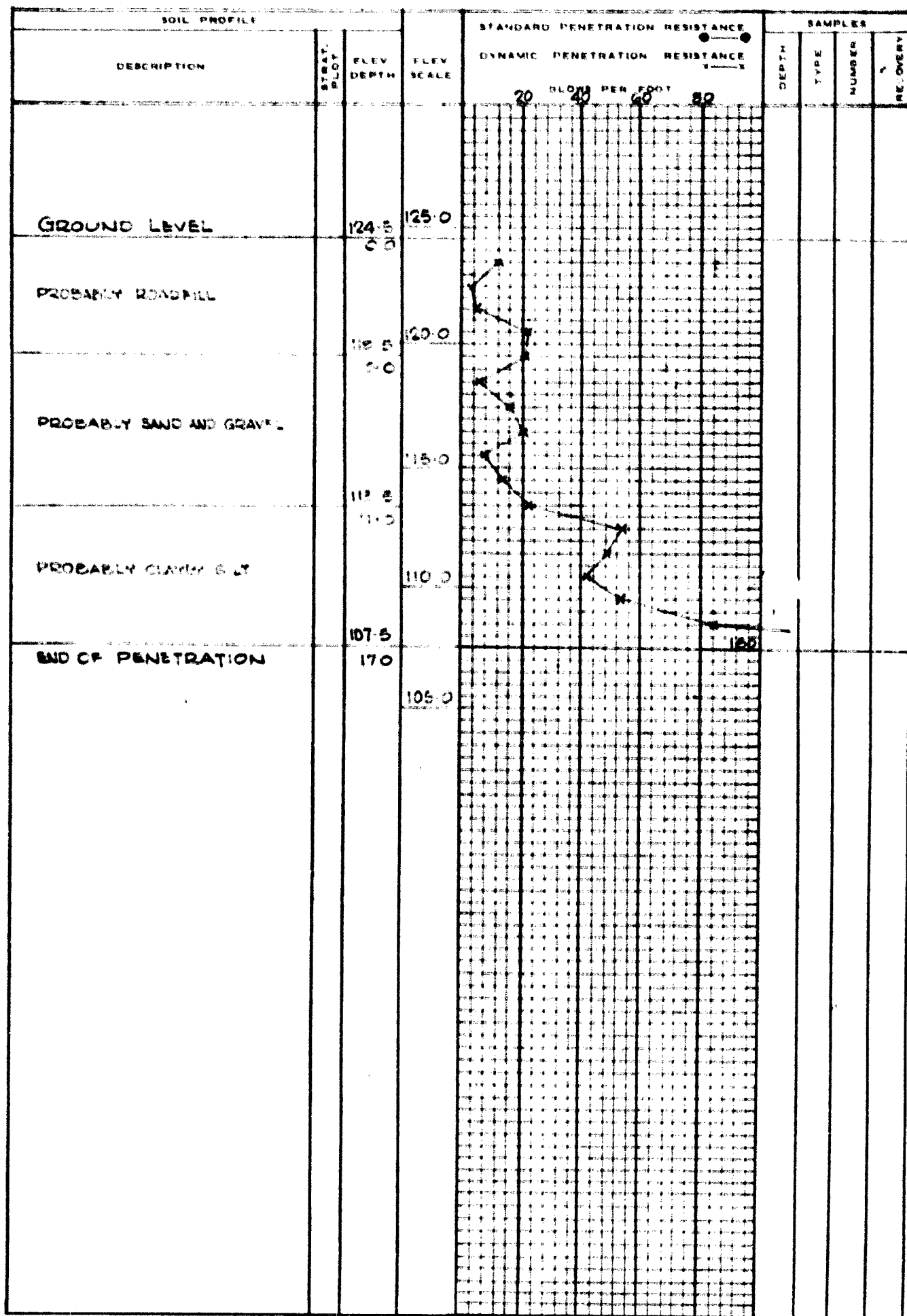
SAMPLE TYPES

- 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 _____

CONTRACT PC 1166 BORING PENETRATION TEST 6 BORING DATE APRIL 4 1963
 DATUM LOCAL DIAM. — HAMMER 140 LBS. DROP 30 IN.



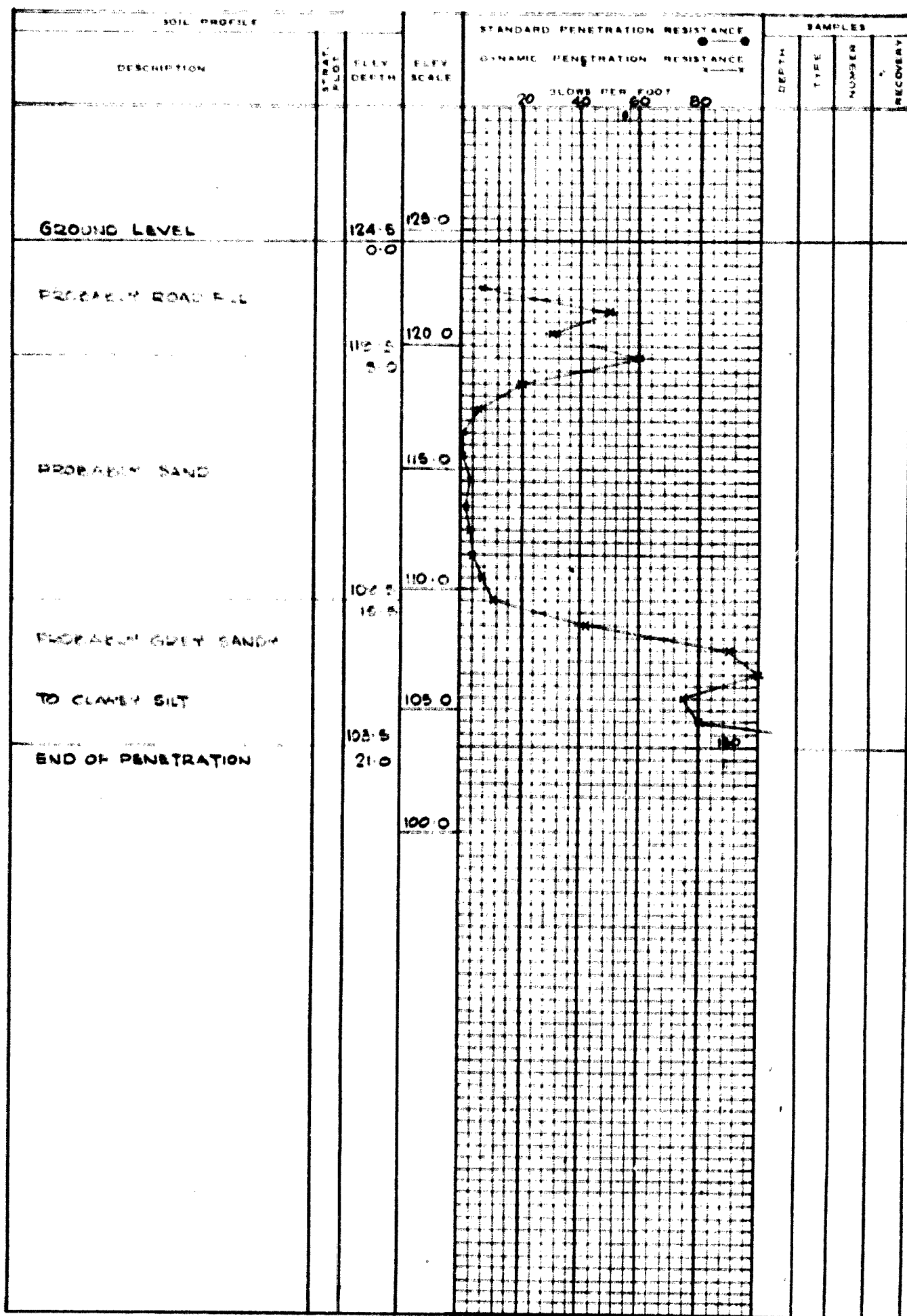
SAMPLE TYPES

AS AUGER SAMPLE
 DO DRIVE OPEN
 OF 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

CONTRACT PC 1188 BORING PENETRATION TEST 7 BORING DATE MARCH 28 1963
 DATUM LOCAL DIAM. _____ HAMMER 140 LBS. DROP 30 IN



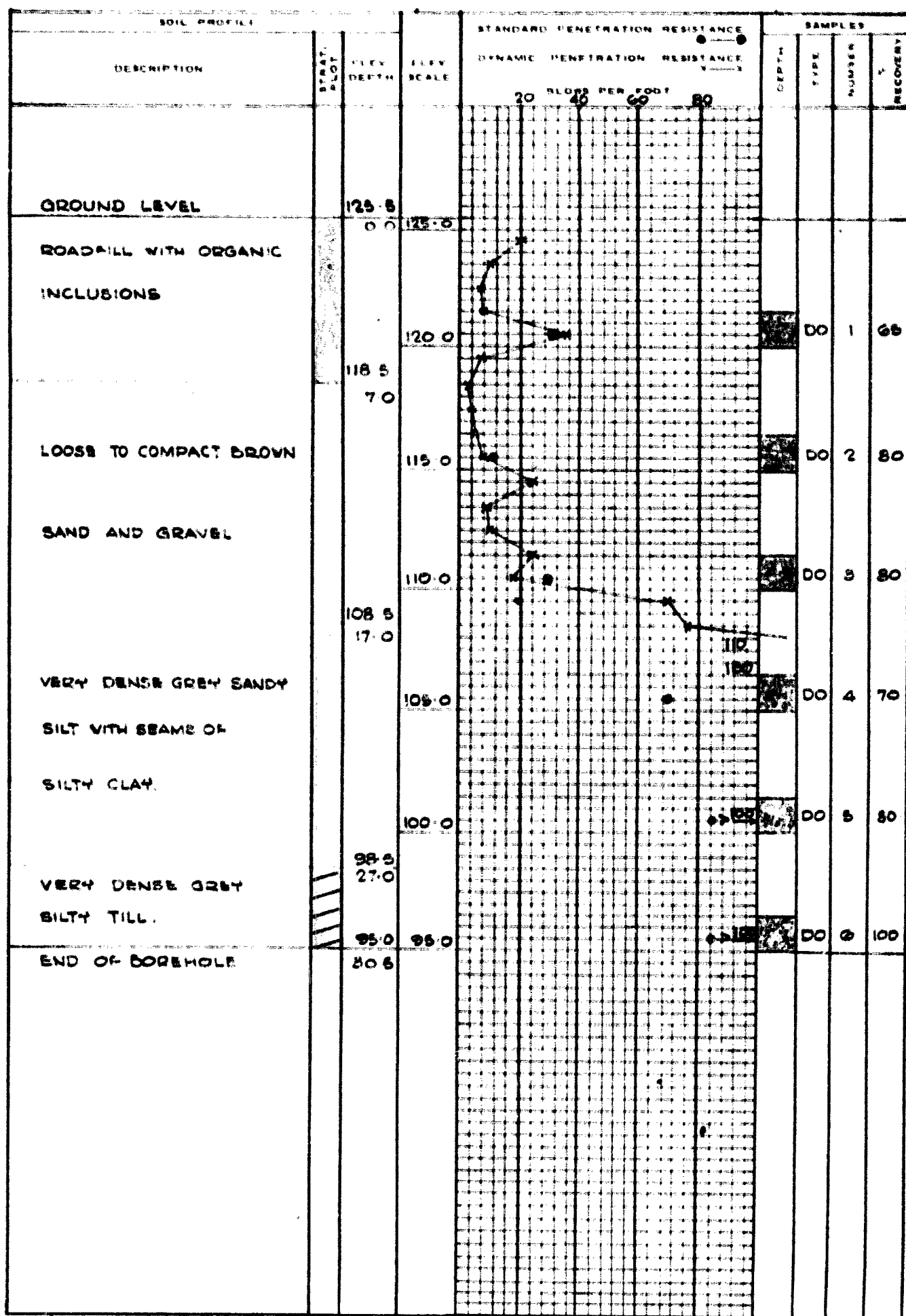
SAMPLE TYPES

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_p FIELD PERMEABILITY TEST
 ↓ GROUND WATER LEVEL
 AT TIME OF BORING

REMARKS

CONTRACT PC 1188 BORING 9 BORING DATE MARCH 28 1968
 DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN



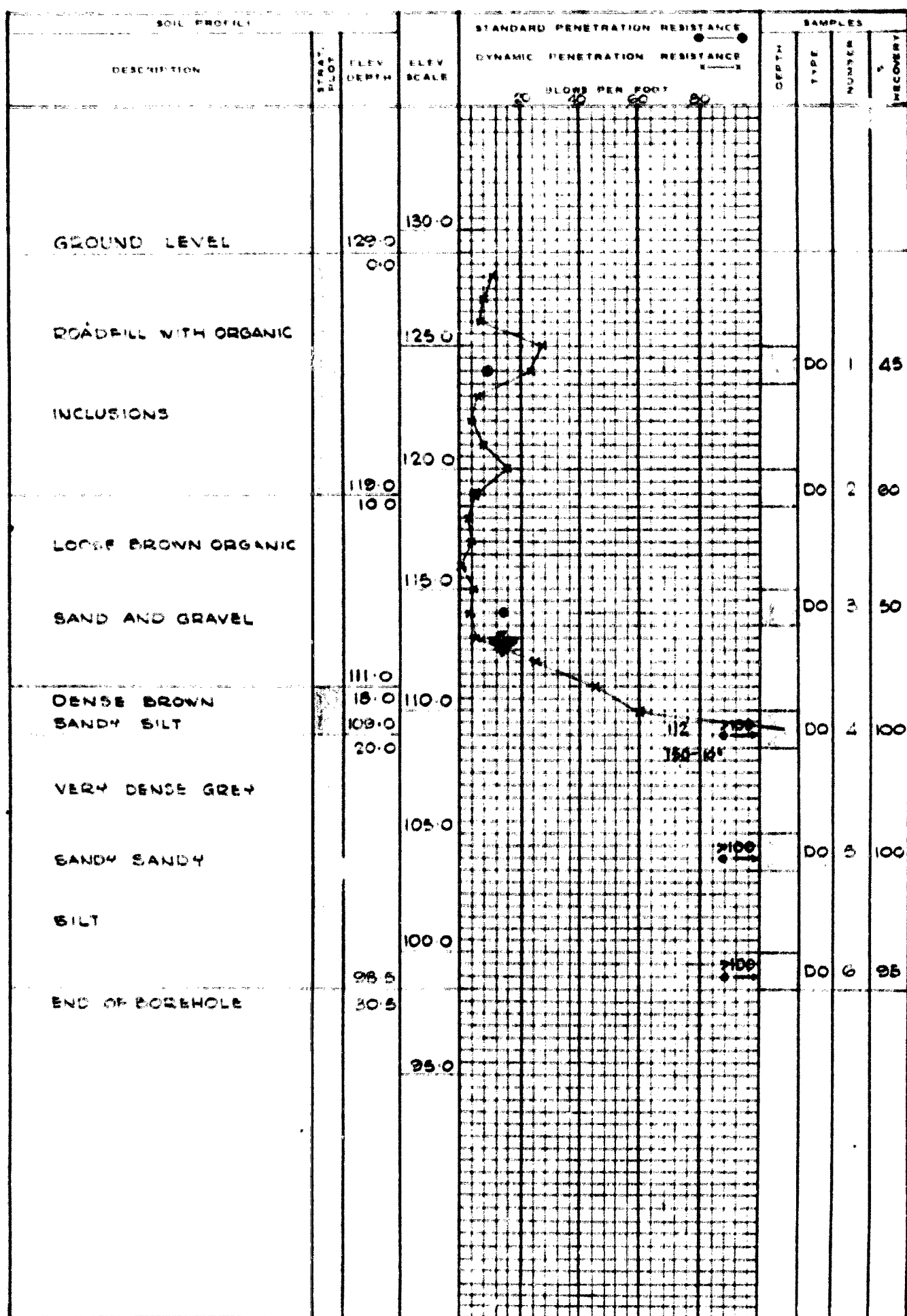
SAMPLE TYPES

AS AUGER SAMPLE
 DO DRIVE OPEN
 OF 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

CONTRACT PC 1185 BORING 10 BORING DATE MARCH 27 1963
 DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN



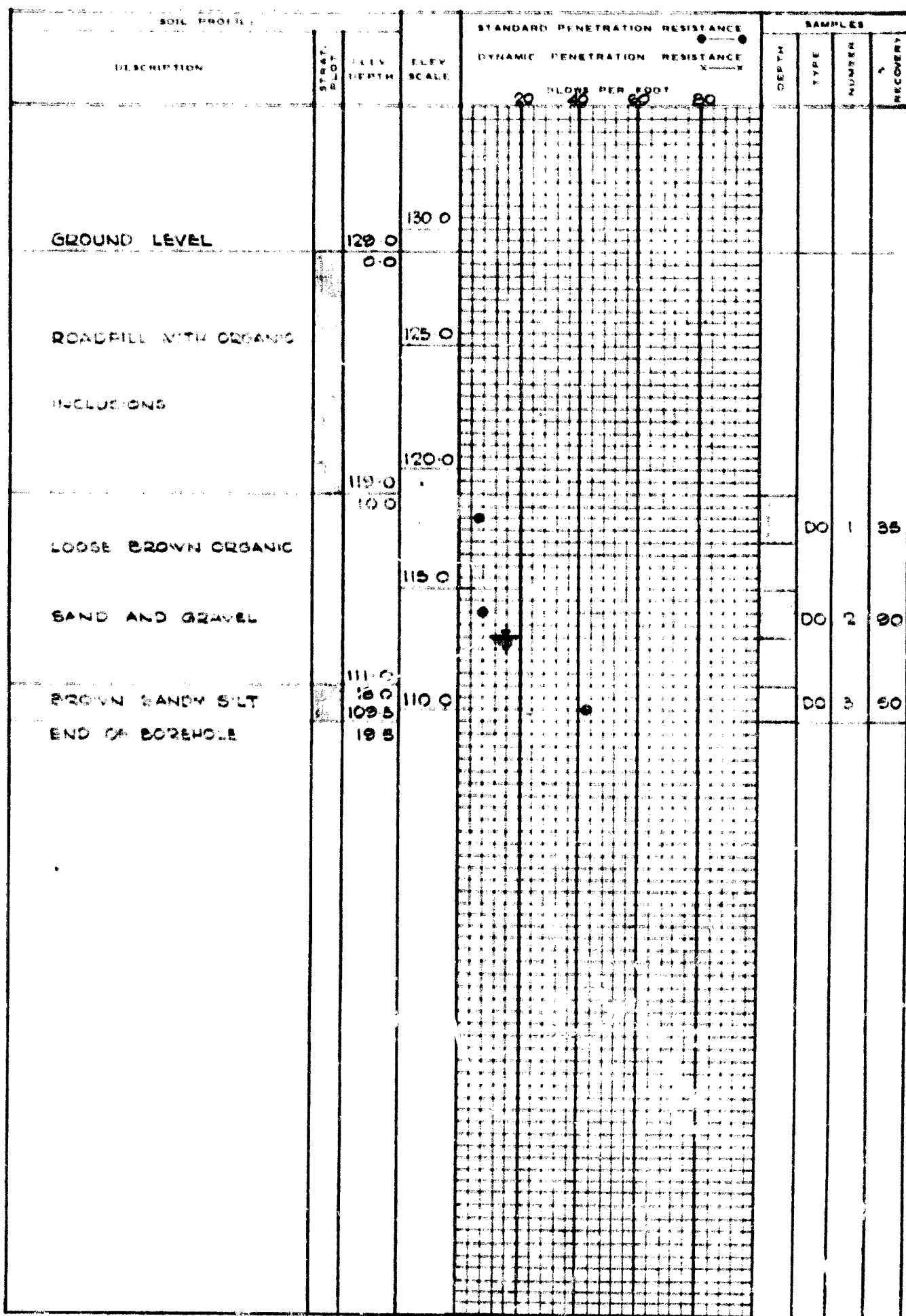
SAMPLE TYPES

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

CONTRACT PC 188 BORING 10A BORING DATE APRIL 5 1963
 DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN



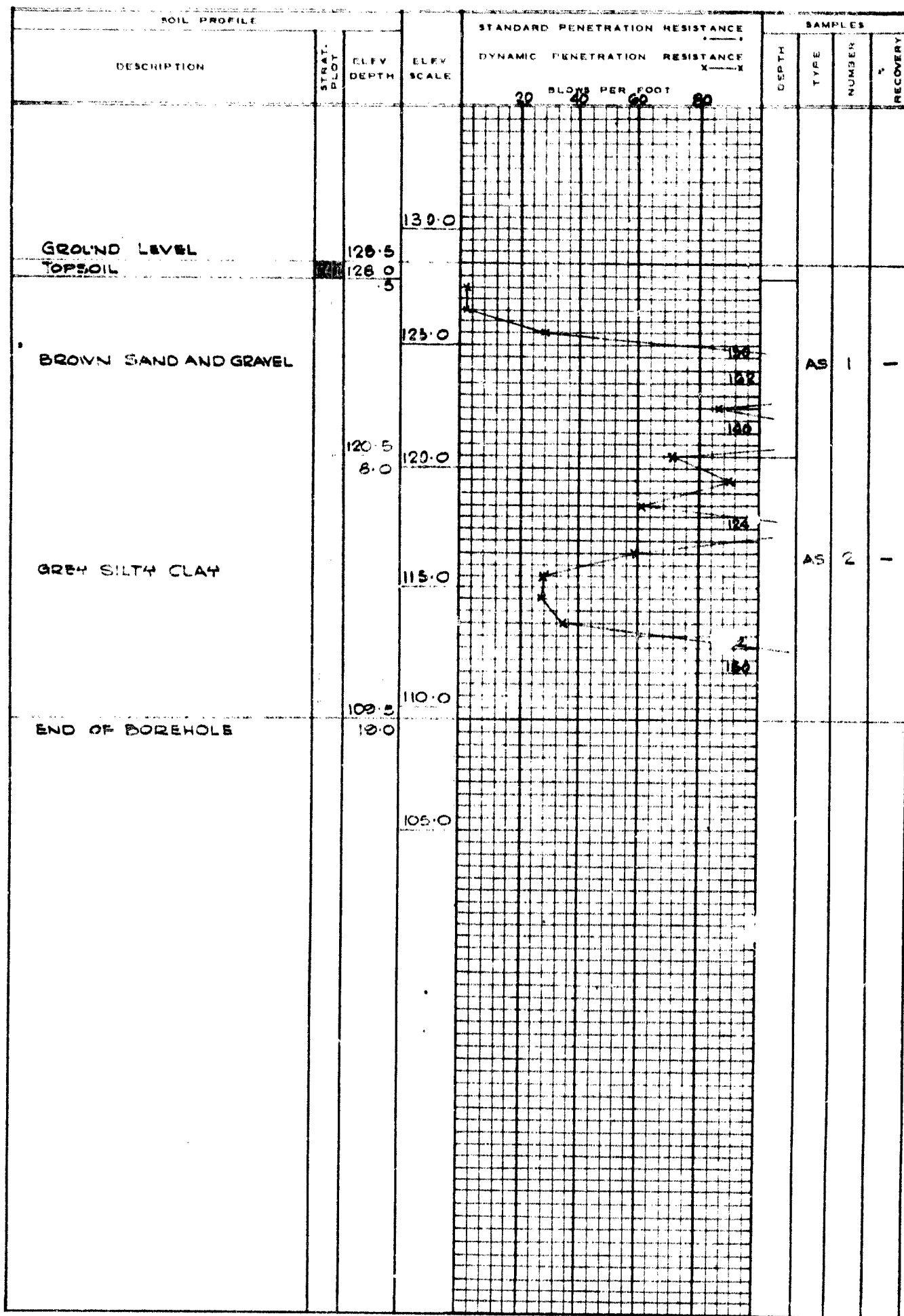
SAMPLE TYPES

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

CONTRACT PC.1186 BORING 11 BORING DATE APRIL 8 1963
 DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN



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

CONTRACT PC1188 BORING 12 BORING DATE APRIL 8 1963
 DATUM LOCAL DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES			
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE X—X	DEPTH	TYPE	NUMBER	RECOVERY
				20 40 60 80 BLOWS PER FOOT				
			130.0					
GROUND FLOOR		128.5						
TOPSOIL		127.5						
		127.0						
VERY DENSE BROWN		125.0						
SAND AND GRAVEL			120.0			DO	1	85
		117.5						
		11.0				AS	2	-
GREY CLAYEY SILT								
		114.5	115.0			AS	3	-
END OF BOREHOLE		14.0						
			110.0					

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

CONTRACT PC 188 BORING 12A BORING DATE APRIL 8 1963
DATUM LOCAL DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES			
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE X—X	DEPTH	TYPE	NUMBER	RECOVERY
				BLOWS PER FOOT 20 40 60 80				
			120.0					
GROUND LEVEL		118.5						
TOPSOIL		118.0						
		.5						
			115.0					
VERY DENSE BROWN SAND AND GRAVEL						DO	1	80
		111.5						
		7.0	110.0					
VERY STIFF TO HARD						DO	2	
GREY SILTY CLAY			105.0					
		103.0				DO	3	100
END OF BOREHOLE		15.5						
			100.0					

SAMPLE TYPES

AS AUGER SAMPLE
DO DRIVE OPEN
OF 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

CONTRACT PC 1188 BORING 13 BORING DATE APRIL 8 1962
 DATUM LOCAL DIAM. 4 1/2" HAMMER 140 LBS. DROP 80 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES			
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS PER FOOT	DEPTH	TYPE	NUMBER	RECOVERY
				20 40 60 80				
GROUND LEVEL		124.0	125.0					
TOPSOIL		123.8						
COMPACT BROWN VERY SILTY FINE SAND			120.0					
		117.0				DO	1	80
		7.0						
STIFF TO HARD BROWN			115.0					
						DO	2	100
			110.0					
TO GREY SILTY CLAY						DO	3	85
			105.0					
		103.5				DO	4	45
END OF BOREHOLE		20.5						
			100.0					

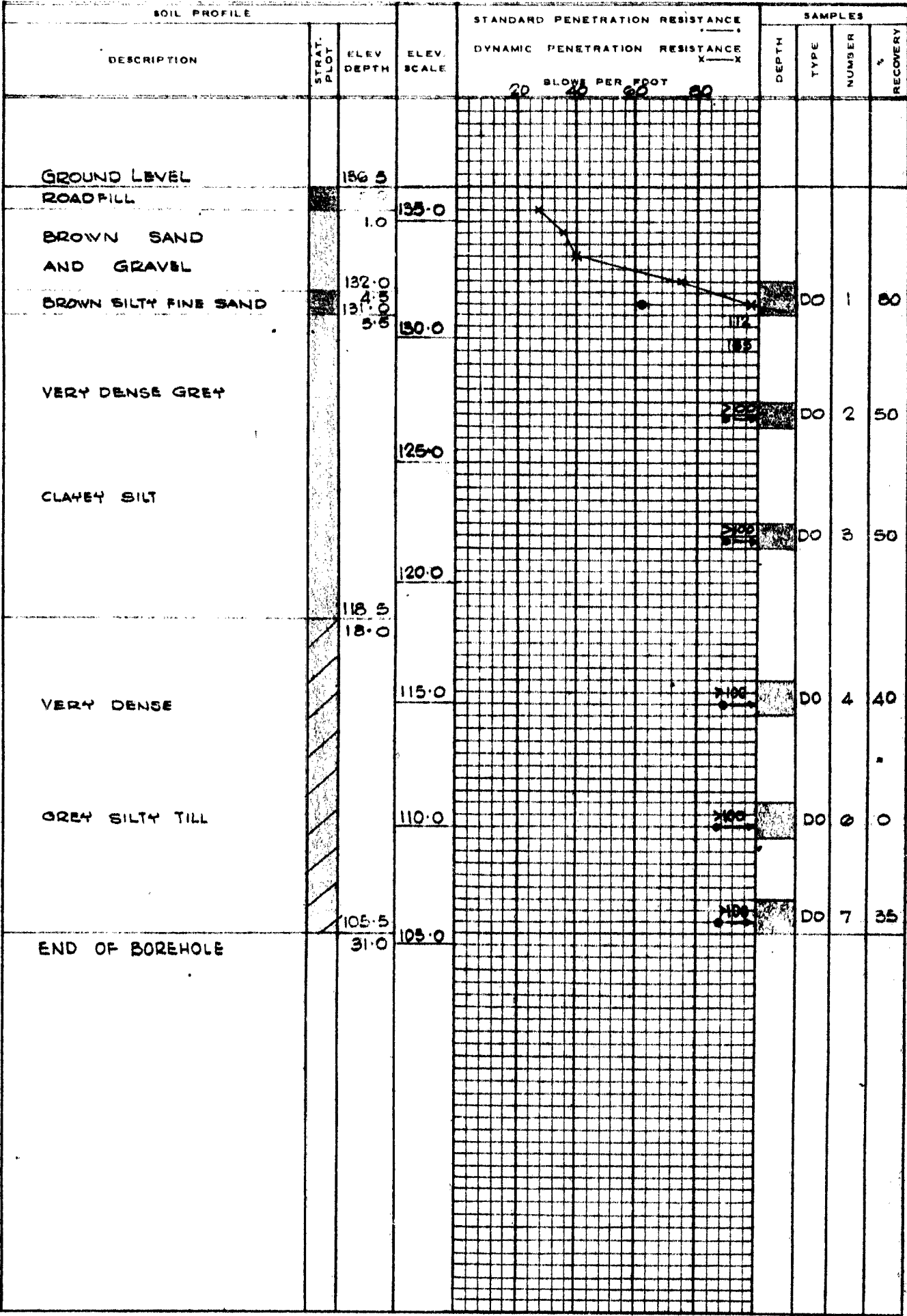
SAMPLE TYPES

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

CONTRACT PC1188 BORING 14 BORING DATE APRIL 8, 17 1963
DATUM LOCAL DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN



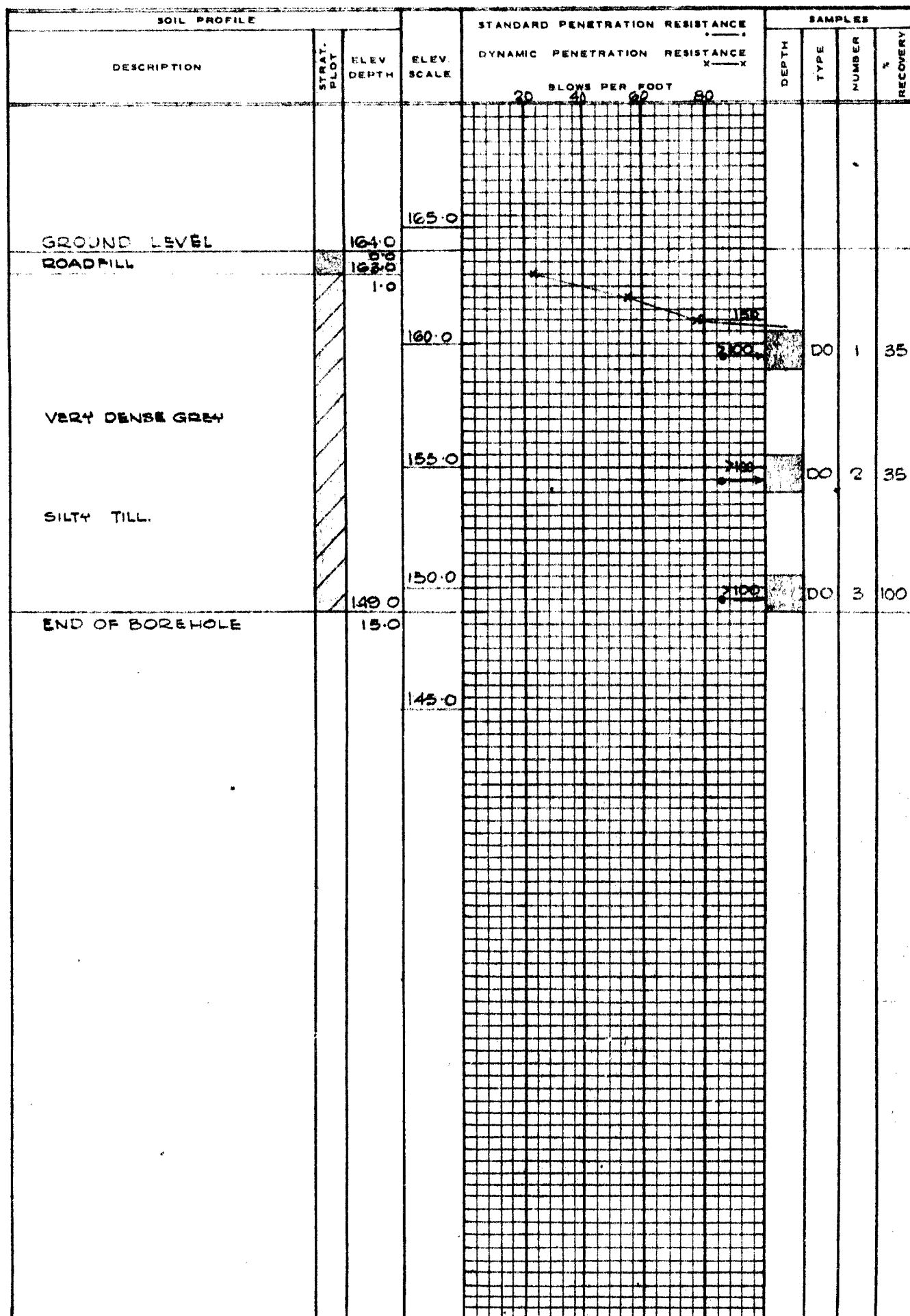
SAMPLE TYPES

- 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 _____

CONTRACT P.C.1188 BORING 15 BORING DATE APRIL 9 1969
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN



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

CONTRACT PC 1188 BORING 16 BORING DATE APRIL 10 1963
 DATUM LOCAL DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES			
DESCRIPTION	STRAT. PLT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS PER FOOT	DEPTH	TYPE	NUMBER	RECOVERY
GROUND LEVEL		123.0	125.0					
DP SOIL		122.5						
COMPACT BROWN SILTY		0.5	120.0					
FINE SAND WITH GRAVEL						DO	1	50
			115.0					
VERY DENSE BROWN CLAYEY		113.5				DO	2	100
SAND AND GRAVEL		9.5						
		110.0	110.0					
VERY DENSE GREY CLAYEY		13.0				DO	3	100
SAND AND GRAVEL			105.0					
		102.5				DO	4	70
END OF BOREHOLE		20.5	100.0					

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

CONTRACT PC1188 BORING 16A BORING DATE APRIL 10 1963
 DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE				SAMPLES				
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				DEPTH	TYPE	NUMBER	RECOVERY
				X → X BLOWS PER FOOT							
GROUND LEVEL TOPSOIL BROWN SILTY FINE SAND WITH GRAVEL			125.0						AS	1	-
		123.0									
		122.5									
		0.5									
GREY CLAYEY SILT END OF BOREHOLE			120.0						AS	2	-
		115.0	115.0								
		.8.0									
			110.0								
		108.0							AS	4	-
	15.0										
			105.0								

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

CONTRACT PC1188 BORING 105 BORING DATE APRIL 16 1963
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE				STANDARD PENETRATION RESISTANCE DYNAMIC PENETRATION RESISTANCE BLOWS PER FOOT		SAMPLES			
DESCRIPTION	STAT. PLD.	ELEV. DEPTH	ELEV. SCALE			DEPTH	TYPE	NUMBER	RECOVERY
			125.0						
GROUND LEVEL		123.0							
TOP SOIL		122.5					AS	1	-
BROWN SILTY FINE SAND		0.5							
			120.0						
WITH GRAVEL							AS	2	-
		115.0	115.0						
		8.0					AS	3	-
GREY CLAYEY SILT			110.0						
		108.0					AS	4	-
END OF BOREHOLE		15.0							
			105.0						

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

CONTRACT PC 1188 BORING 10C BORING DATE APRIL 16 1963
DATUM LOCAL DIAM. 42" HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE				SAMPLES				
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				DEPTH	TYPE	NUMBER	RECOVERY
				BLOWS PER FOOT							
			125.0								
GROUND LEVEL		123.0									
TOPSOIL		122.5									
BROWN SILTY FINE SAND		0.5							AS	1	-
			120.0								
WITH GRAVEL											
		117.0									
		6.0							AS	2	-
			115.0								
GREY CLAYEY SILT											
			110.0								
		108.0							AS	4	-
END OF BOREHOLE		15.0									
			105.0								

SAMPLE TYPES

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

RC ROCK CORE
KF FIELD PERMEABILITY TEST
▽ GROUND WATER LEVEL
AT TIME OF BORING

REMARKS

CONTRACT PC 1188 BORING 16D BORING DATE APRIL 10 1968
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES				
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		DEPTH	TYPE	NUMBER	RECOVERY
				X—X					
				GLOWS PER FOOT					
			125.0						
GROUND LEVEL		123.0							
TOPSOIL		122.5					AS	1	-
BROWN SILTY FINE			120.0						
SAND WITH GRAVEL									
		117.0					AS	2	-
		6.0							
			115.0						
GREY CLAYEY SILT							AS	3	-
			110.0						
		108.0					AS	4	-
END OF BOREHOLE		15.0							
			105.0						

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

CONTRACT PC 1188 BORING 16E BORING DATE APRIL 16 1963
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES				
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE		DEPTH	TYPE	NUMBER	RECOVERY
				X — X					
				BLOWS PER FOOT					
GROUND LEVEL TOPSOIL BROWN SILTY FINE SAND WITH GRAVEL		123.0	125.0				AS	1	-
		122.5							
		0.5							
GREY CLAYEY SILT		120.0					AS	2	-
		117.0							
		6.0							
		115.0							
		110.0							
END OF BOREHOLE		108.0					AS	4	-
		15.0							
			105.0						

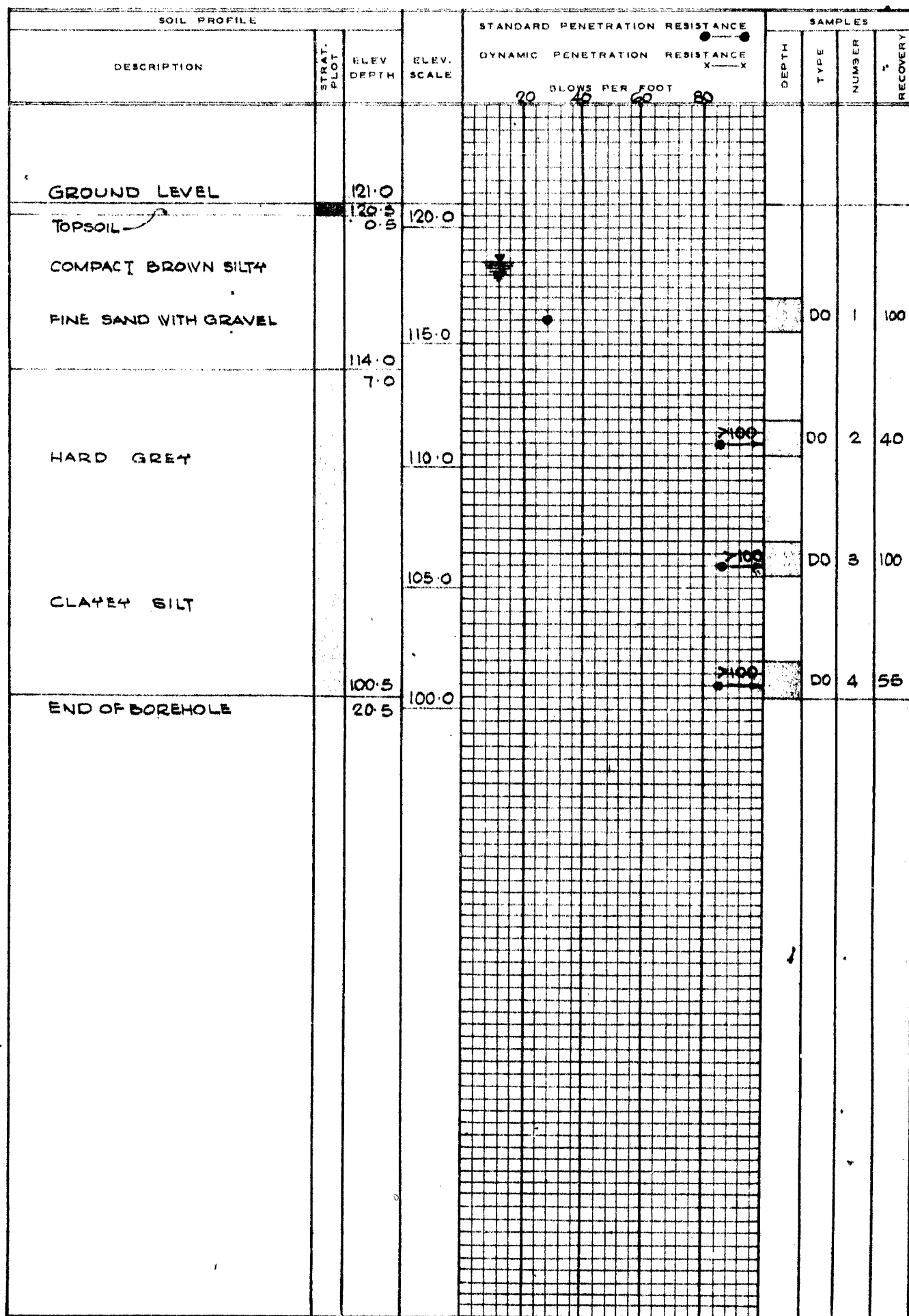
SAMPLE TYPES

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

CONTRACT PC 1188 BORING 17 BORING DATE APRIL 16 1968
 DATUM LOCAL DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN



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

CONTRACT PC 1188 BORING 18 BORING DATE APRIL 16 1963
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		DYNAMIC PENETRATION RESISTANCE		SAMPLES			
DESCRIPTION	STRAT. PLOT	ELEV. DEPTH	ELEV. SCALE	BLOWS PER FOOT			DEPTH	TYPE	NUMBER	RECOVERY
				20	40	60	80			
			125.0							
GROUND LEVEL		122.5								
TOPSOIL		122.0								
		0.5								
			120.0							
COMPACT BROWN SILTY								DO	1	100
FINE SAND WITH GRAVEL										
			115.0							
		113.5								
		9.0						DO	2	100
HARD GREY										
			110.0							
								DO	3	100
CLAYEY SILT										
			105.0							
								DO	4	100
		102.0								
END OF BOREHOLE		20.5								
			100.0							

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

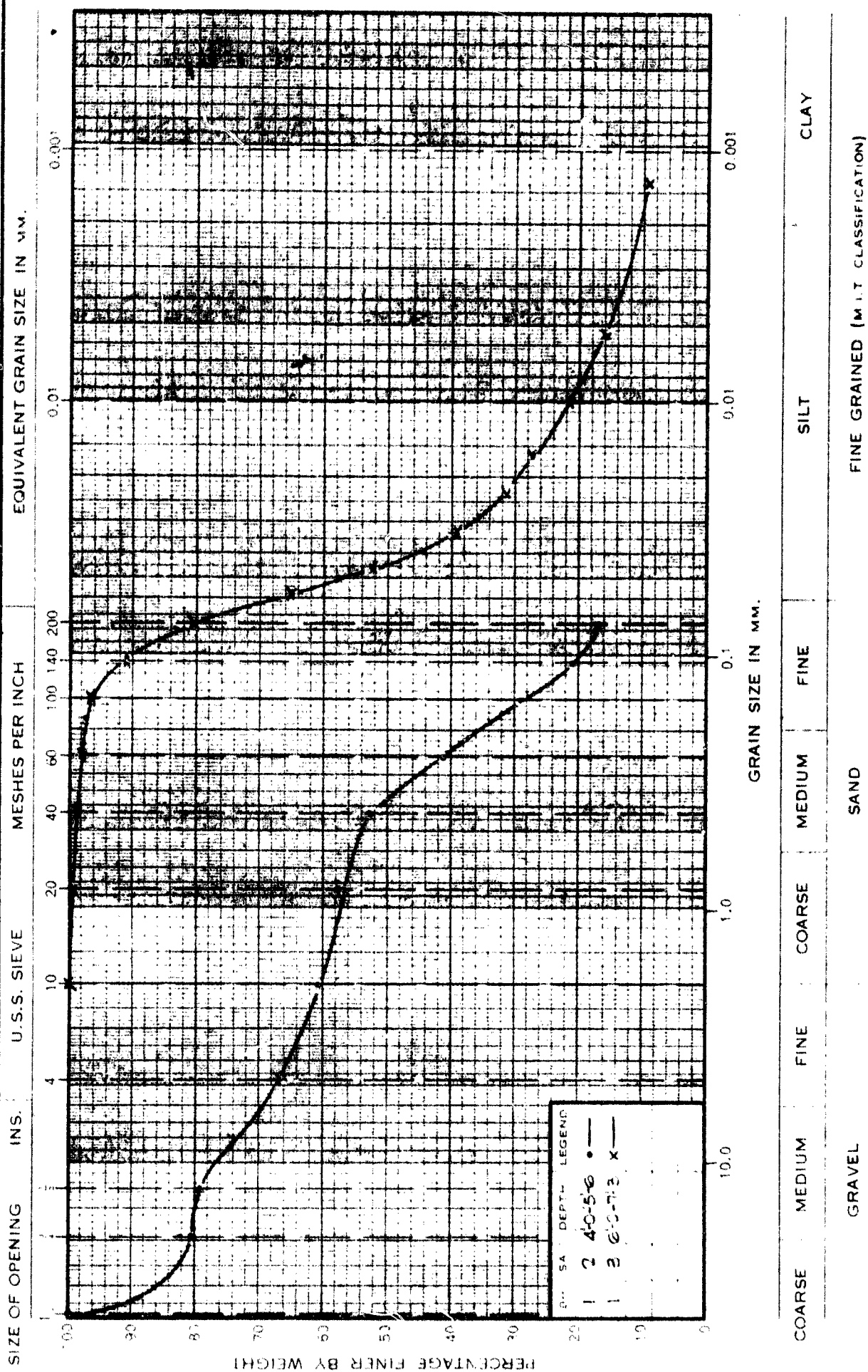
RC ROCK CORE
K_F FIELD PERMEABILITY TEST
▽ GROUND WATER LEVEL
AT TIME OF BORING

REMARKS

APPENDIX II

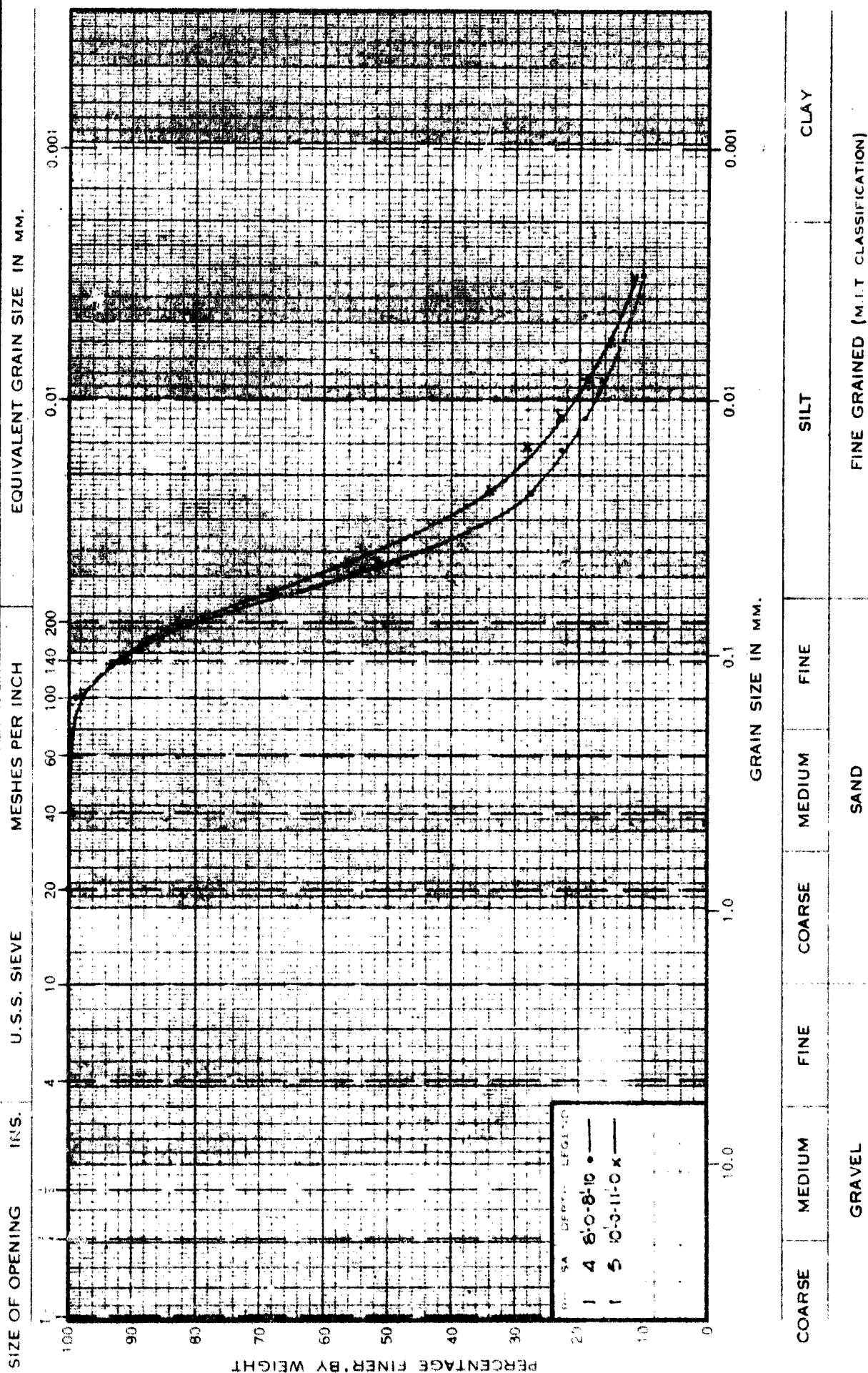
Figures -- Laboratory Testing

APPENDIX II
FIGURE I
CONTRACT PC 1192



FRANKI OF CANADA LIMITED
GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 2
CONTRACT P.C. 1188



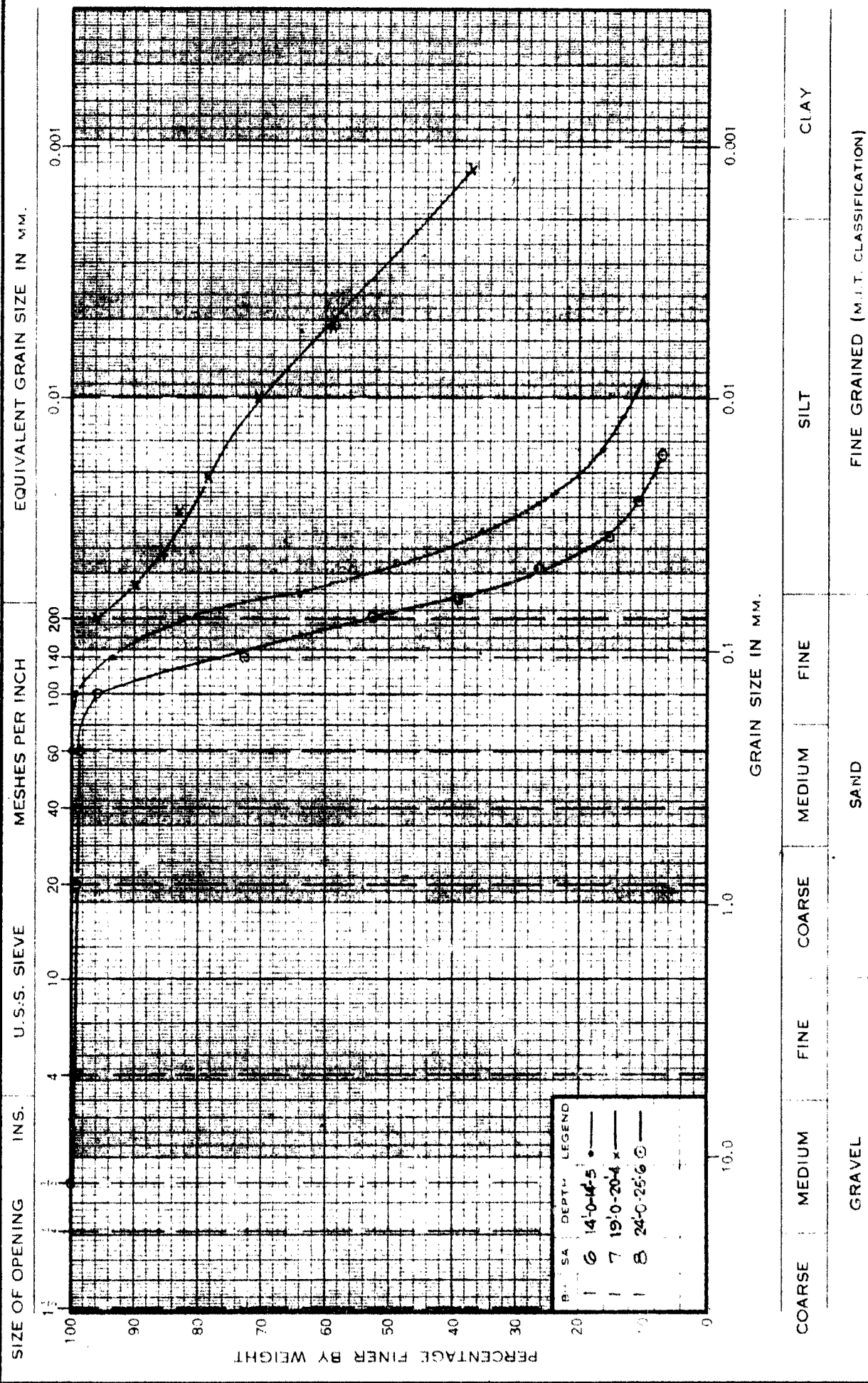
FRANKI OF CANADA LIMITED

GRAIN SIZE DISTRIBUTION

APPENDIX II

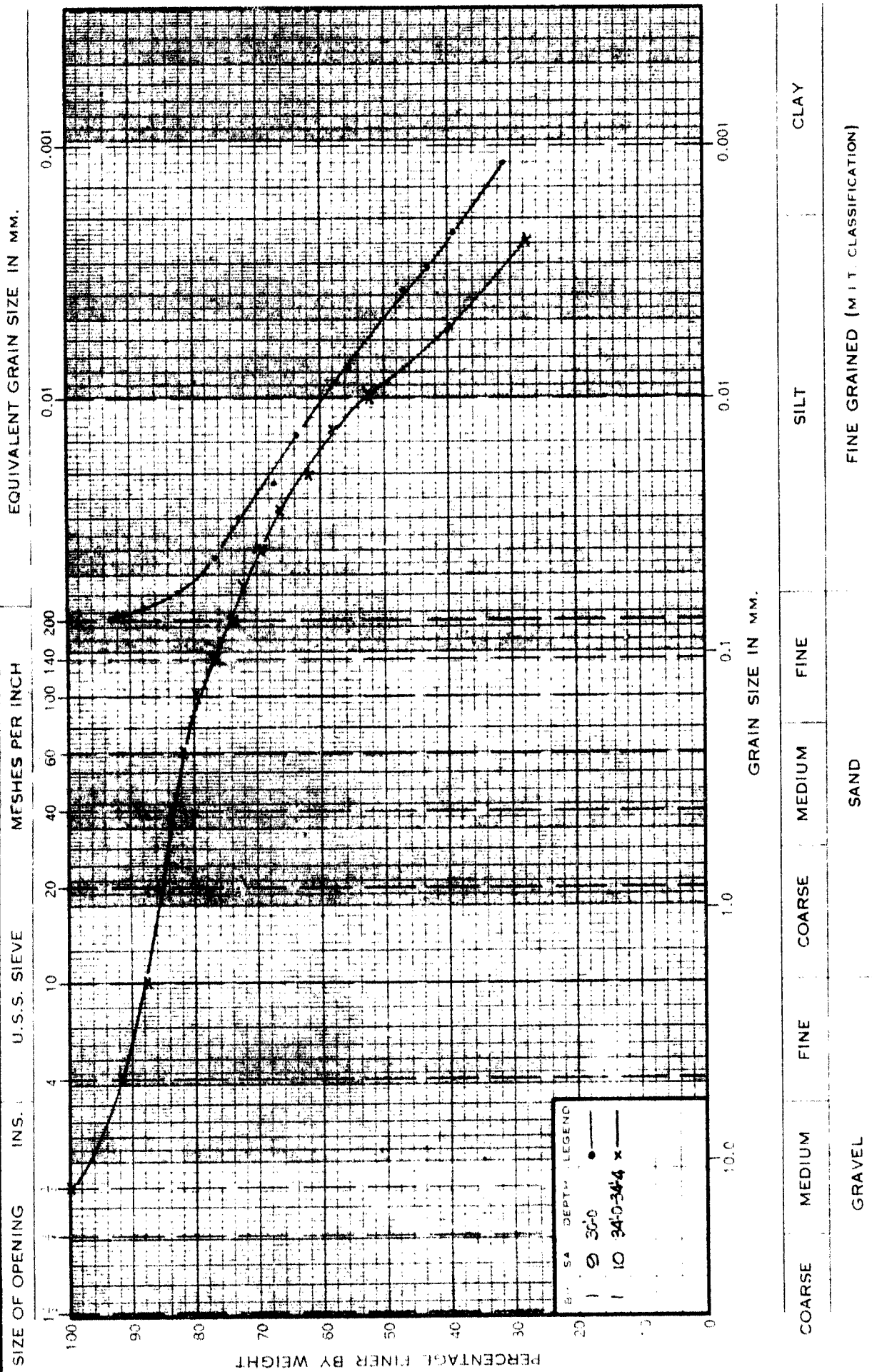
FIGURE 3

CONTRACT PC1188

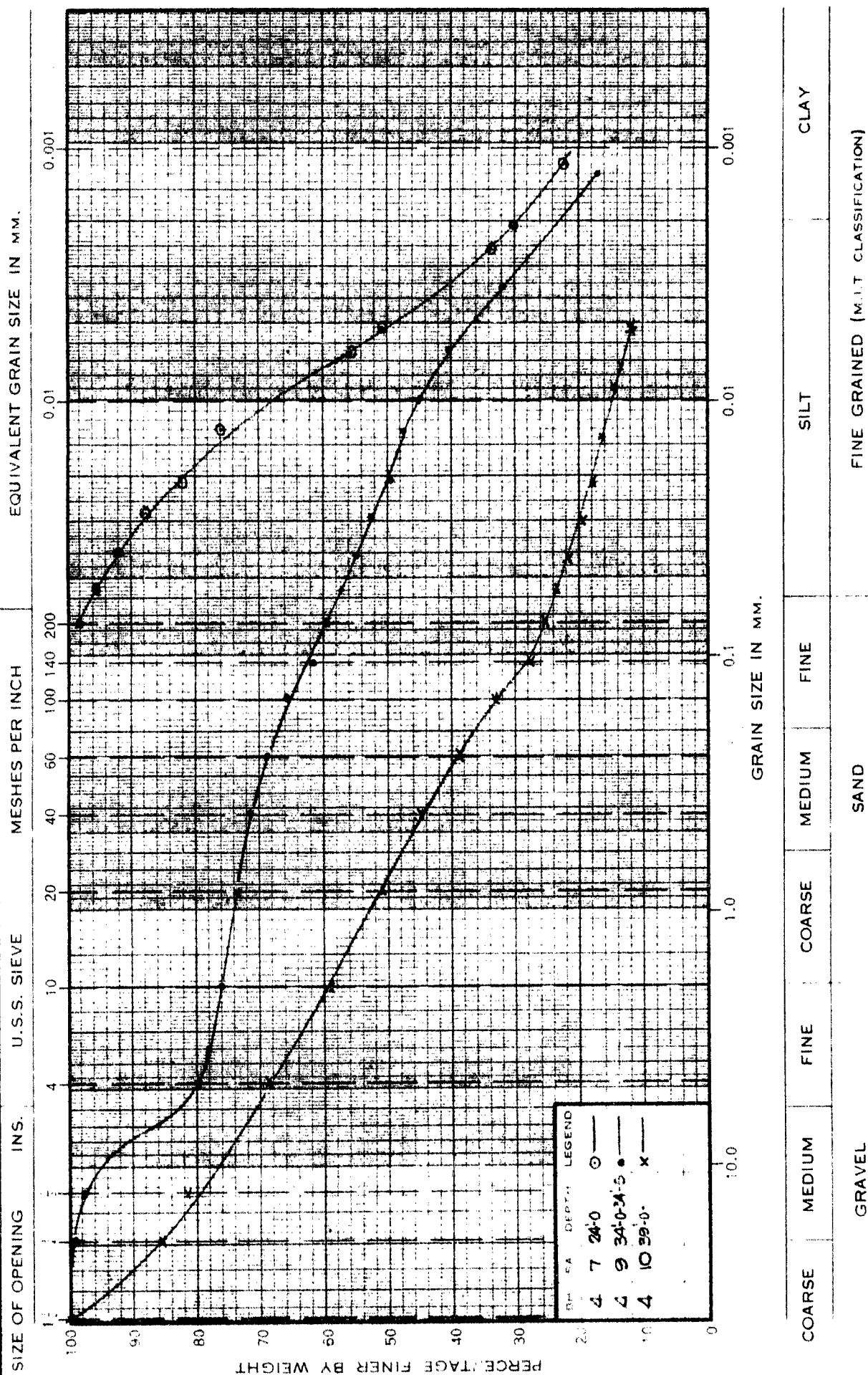


FRANKI OF CANADA LIMITED
GRAIN SIZE DISTRIBUTION

APPENDIX II
FIGURE 4
CONTRACT PC 1188



APPENDIX II
FIGURE 5
CONTRACT PC1188

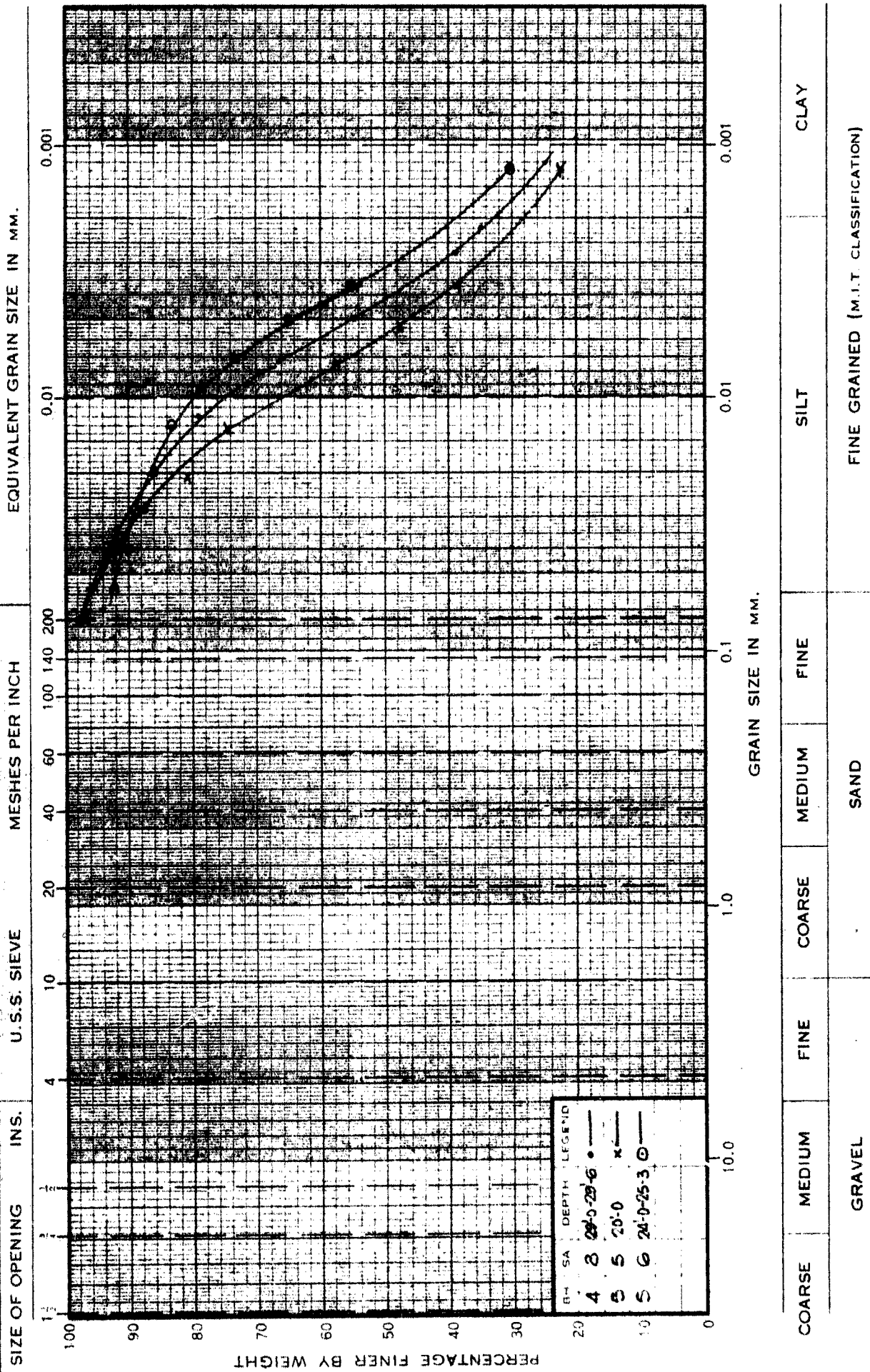


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GRAIN SIZE DISTRIBUTION

APPENDIX II

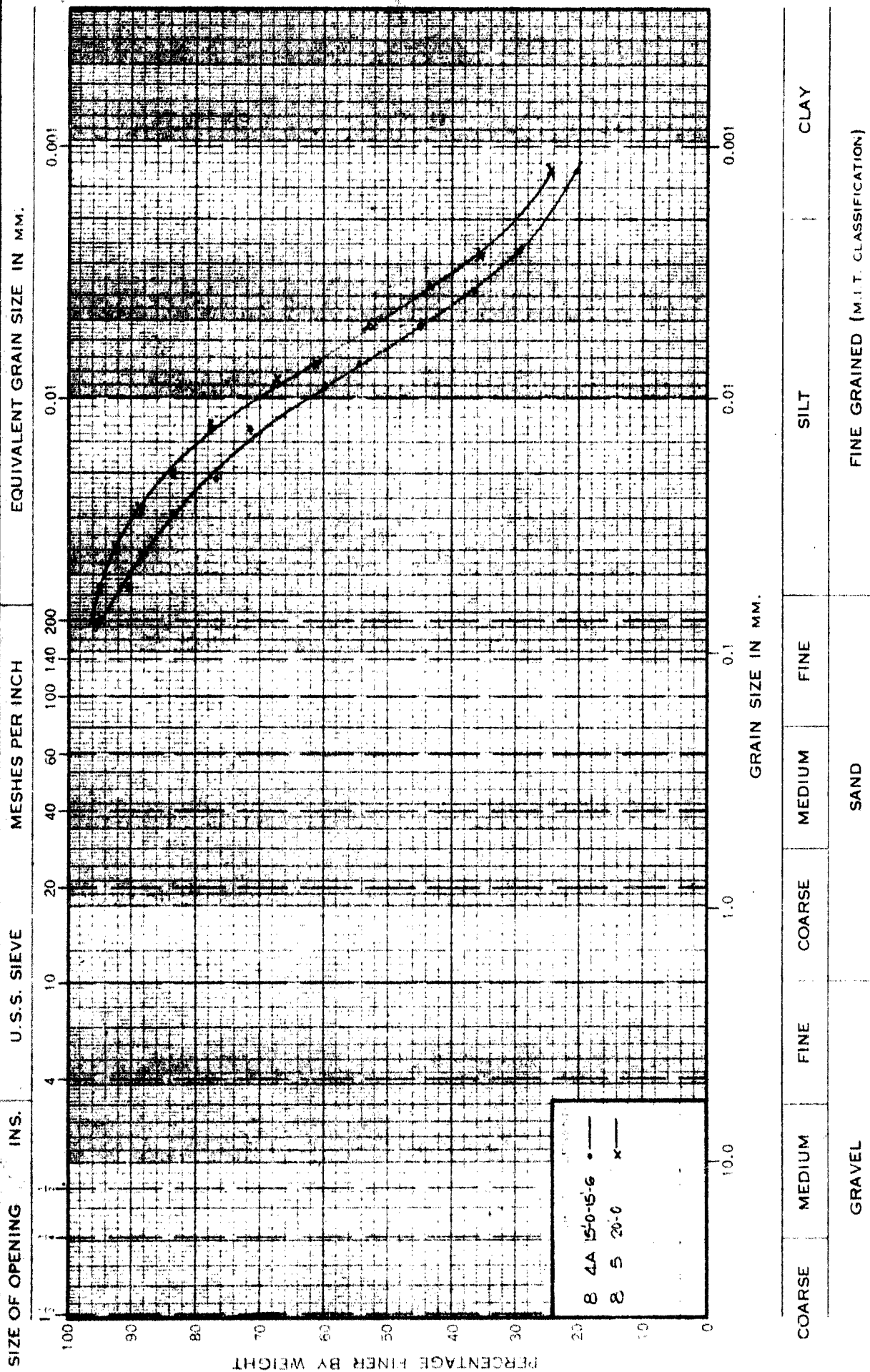
FIGURE 6

CONTRACT PC 1183

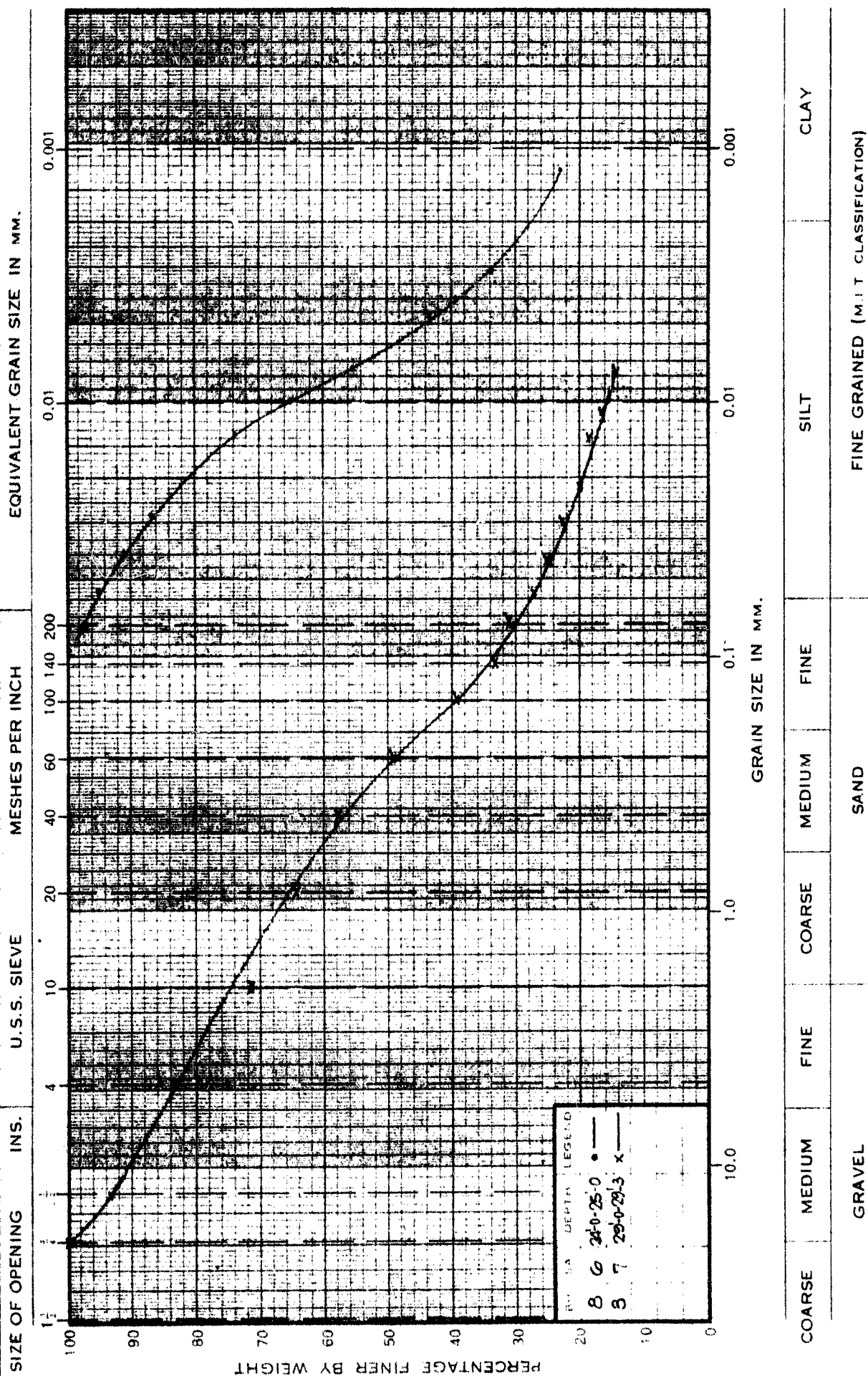


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GRAIN SIZE DISTRIBUTION

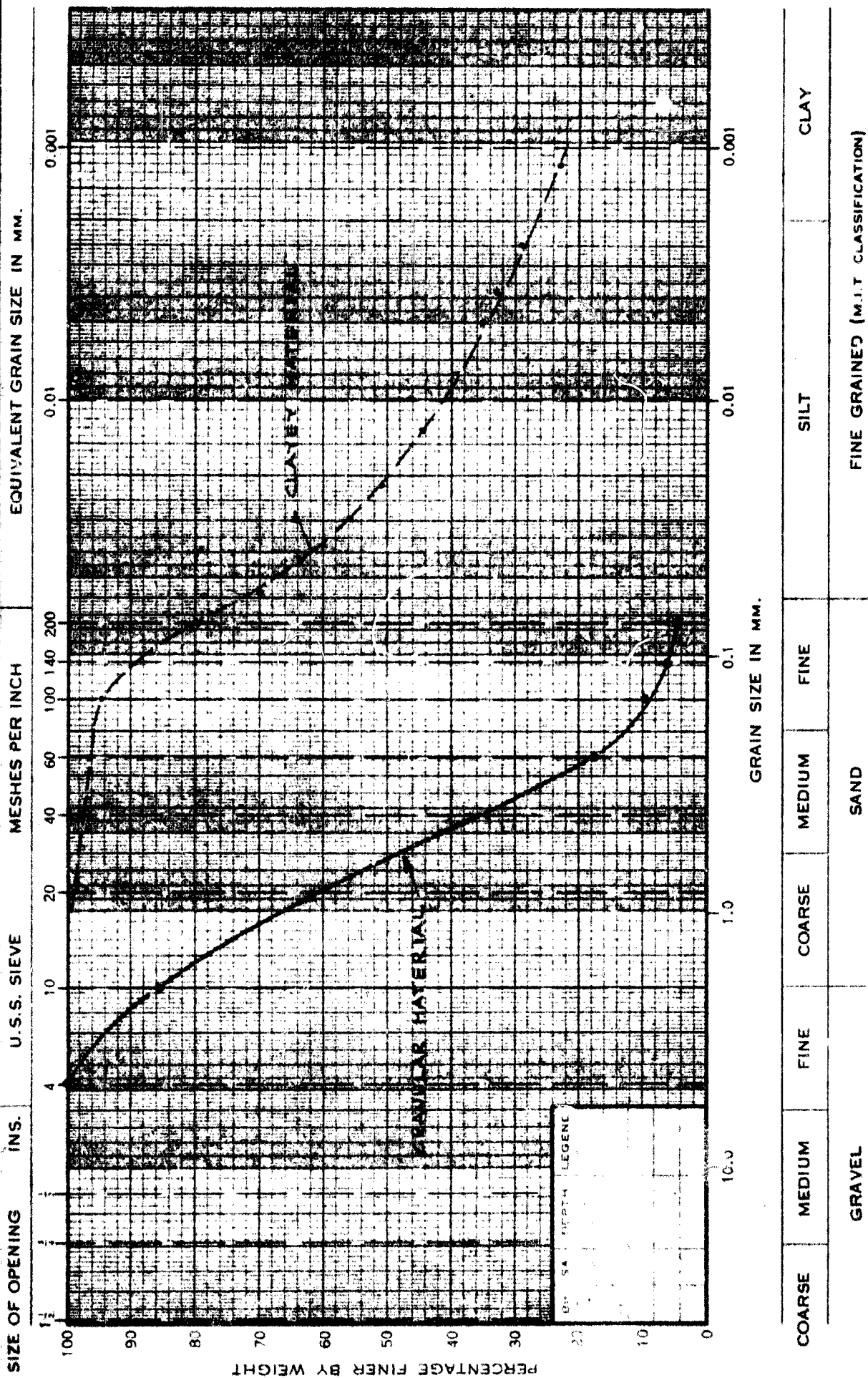
APPENDIX II
FIGURE 7
CONTRACT PC 1188



CONTRACT PC 1188

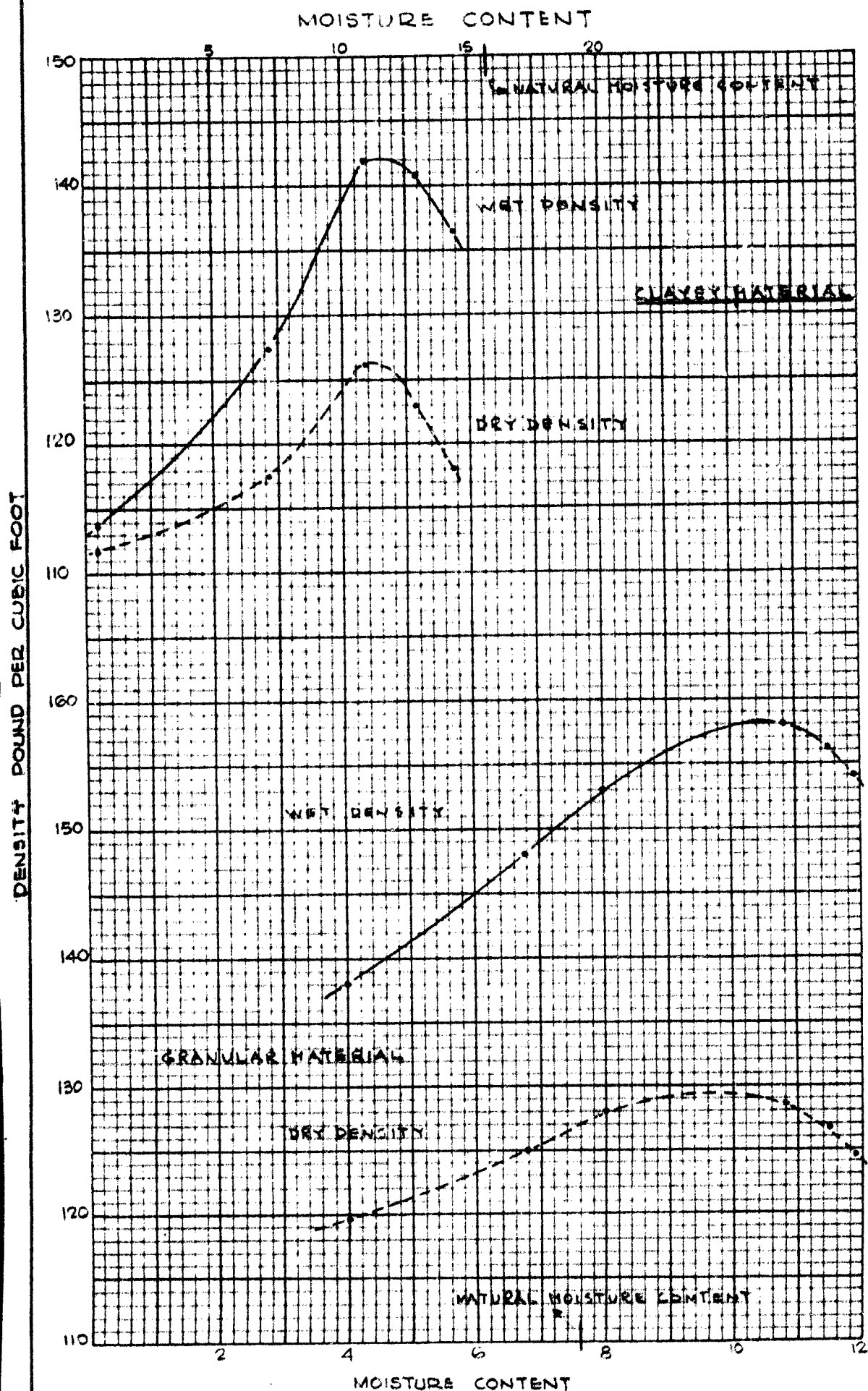


APPENDIX II
FIGURE 9
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STANDARD PROCTOR
COMPACTION TESTS

APPENDIX II
FIGURE 10
CONTRACT PC1188



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MOISTURE CONTENTS AND
ATTERBERG LIMITS
VERSUS ELEVATION

APPENDIX II

FIGURE II

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