

# 63-F-278

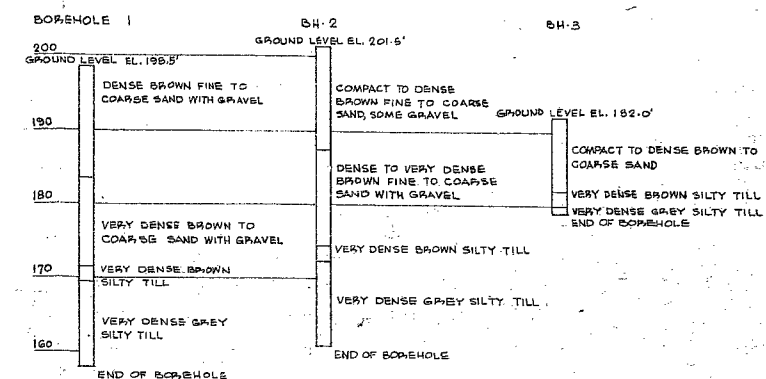
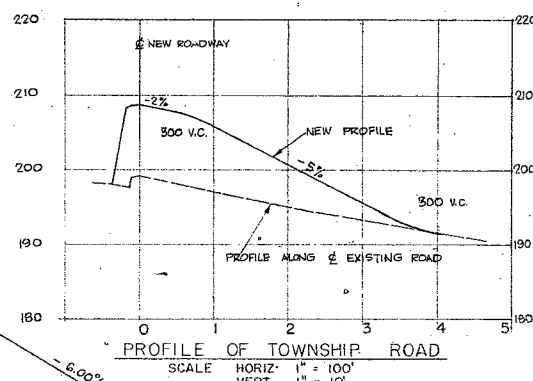
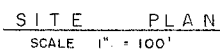
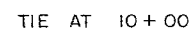
C.N.R

OVERHEAD AT

POTOWN

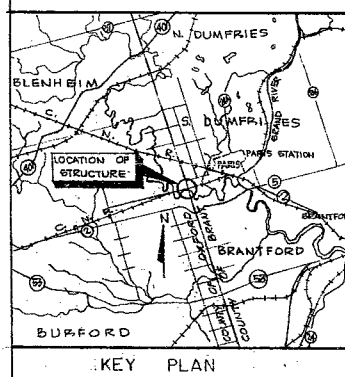
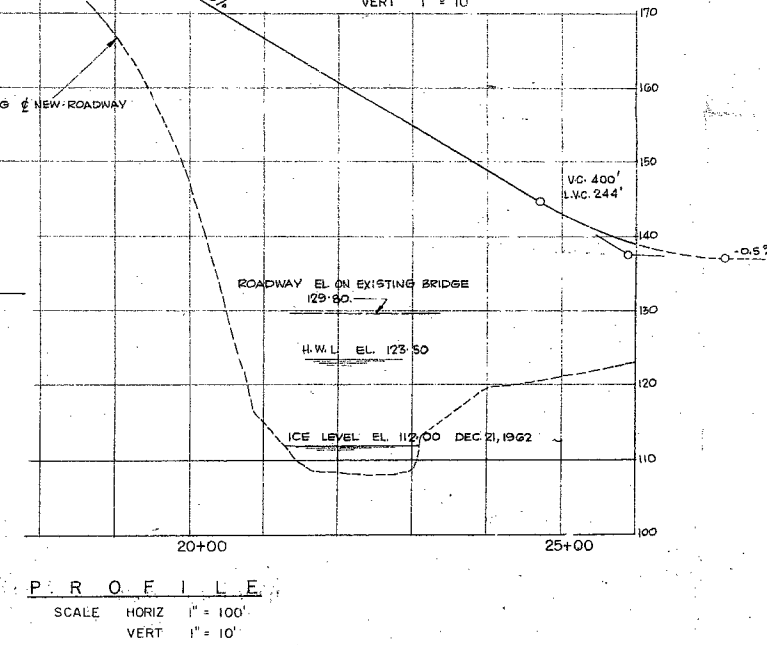
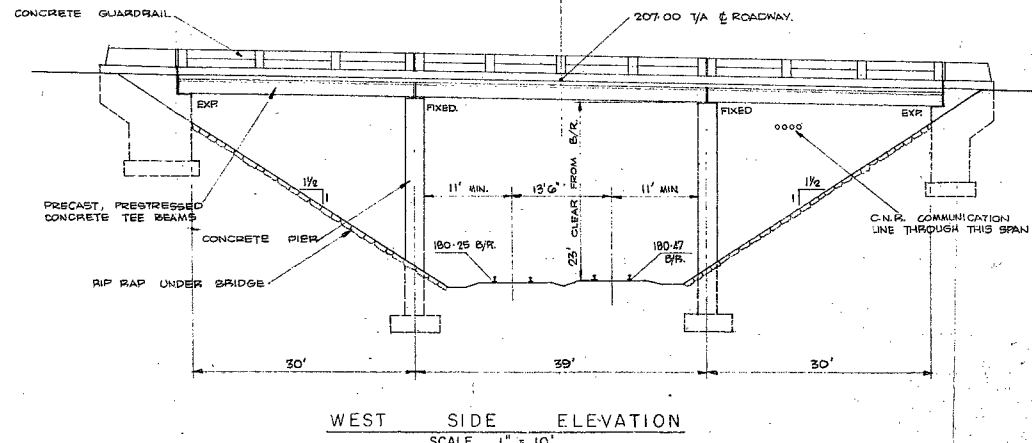
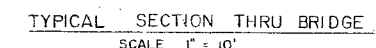
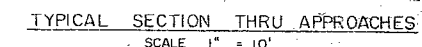
BLENHEIM

TWP.



### TEST BORING RESULTS

BORINGS ARE BY FRANKI OF CANADA, LIMITED, TORONTO, ONTARIO.  
BORINGS ARE FOR GENERAL INFORMATION ONLY AND ARE NOT GUARANTEED BY THE TOWNSHIPS.



## DESIGN CRITERIA

ESTIMATED A.D.T	CLASS 400-1000 (YEAR 1983)
DESIGN SPEED	40 MPH
MAX. GRADE	6 %
STOPPING SIGHT DISTANCE	275'
ROAD SURFACE WIDTH	22'
SHOULDER	8'
BRIDGE ROADWAY WIDTH	30'

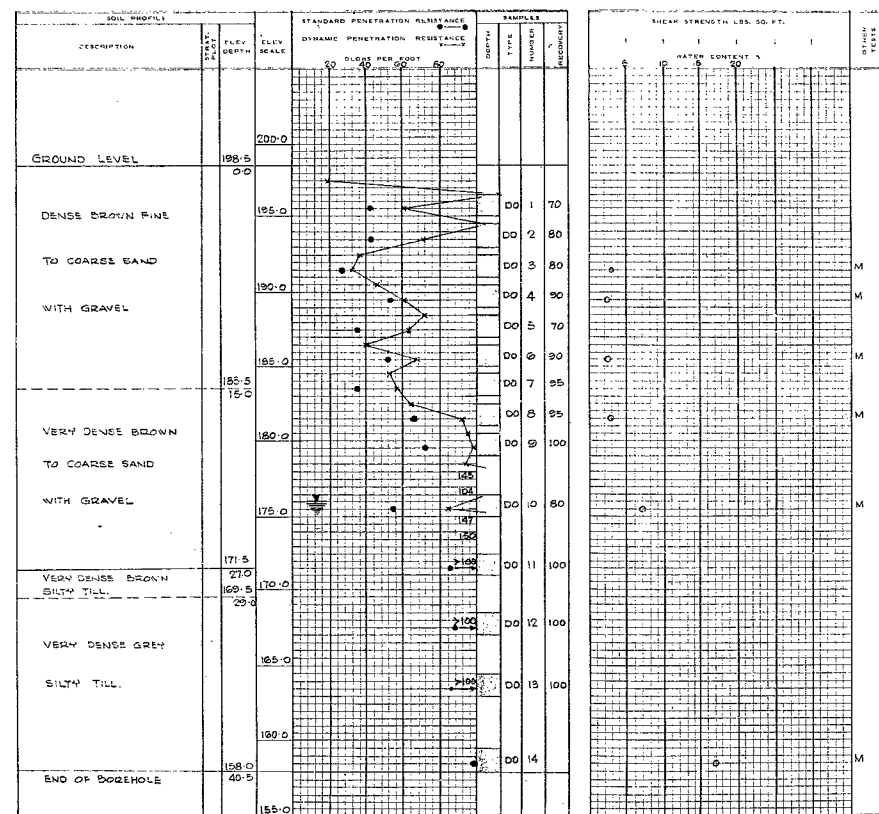
REVISIONS			CURRY & JOSE TWP'S OF BLENHEIM & SOUTH DUMFRIES	DRAWN BY L.H.
DATE	BY	REMARKS	CNR. OVERHEAD AT PUTOWN MILEAGE 33-21 DUNDAS' SUBDIVISION	DRAWN BY G.R.
			LOT 1 CON 1 TOWNSHIP OF BLENHEIM COUNTY OF OXFORD	CHECKED BY L.H.
			NAME OF DRAWG. SITE PLAN AND PROFILE	DATE 13 MAR 63
			C. C. PARKER & ASSOCIATES Ltd. CONSULTING ENGINEERS	2032 REV. NO.
			LONDON HAMILTON EDMONTON	DEPARTMENT NO.

FRANKI OF CANADA LTD.

## BORING RECORD

CONTRACT PC.1187 BORING 1 BORING DATE MARCH 26 1963  
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

## LABORATORY TESTS



## SAMPLE TYPES

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

## RC ROCK CORE

K<sub>p</sub> FIELD PERMEABILITY TEST  
G<sub>w</sub> GROUND WATER LEVEL  
AT TIME OF BORING

REMARKS

O WATER CONTENT  
X ATTENDING LIMITS  
Y UNSAT. UNIT WEIGHT  
M MECHANICAL ANALYSIS  
K<sub>p</sub> PERMEABILITY FIELD  
K<sub>u</sub> PERMEABILITY LAB.  
G<sub>w</sub> RELATIVE DENSITY  
S<sub>u</sub> SPECIFIC GRAVITY  
P COMPACTION

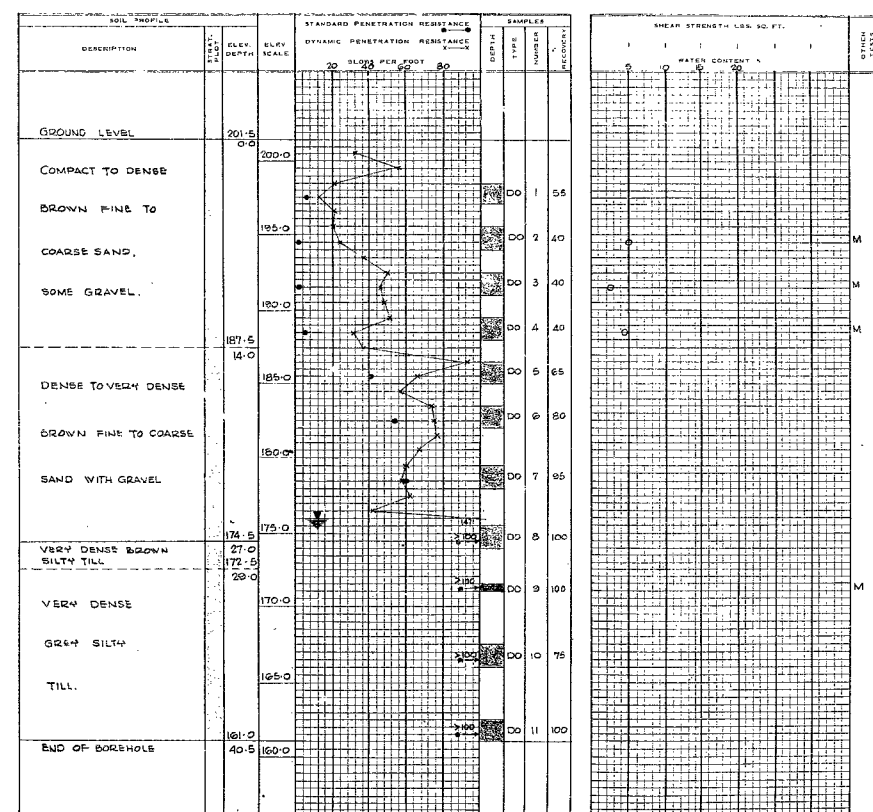
UNCONFINED  
UNDRAINED TRIAXIAL  
IN SITU VANE  
LAB VANE  
CONSOLIDATED UNDRAINED  
CUP CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS  
CONSOLIDATED DRAINED  
CONSOLIDATION

FRANKI OF CANADA LTD.

## BORING RECORD

CONTRACT PC.1187 BORING 2 BORING DATE MARCH 27 1963  
DATUM LOCAL DIAM. 4 1/2 HAMMER 140 LBS. DROP 30 IN

## LABORATORY TESTS



## SAMPLE TYPES

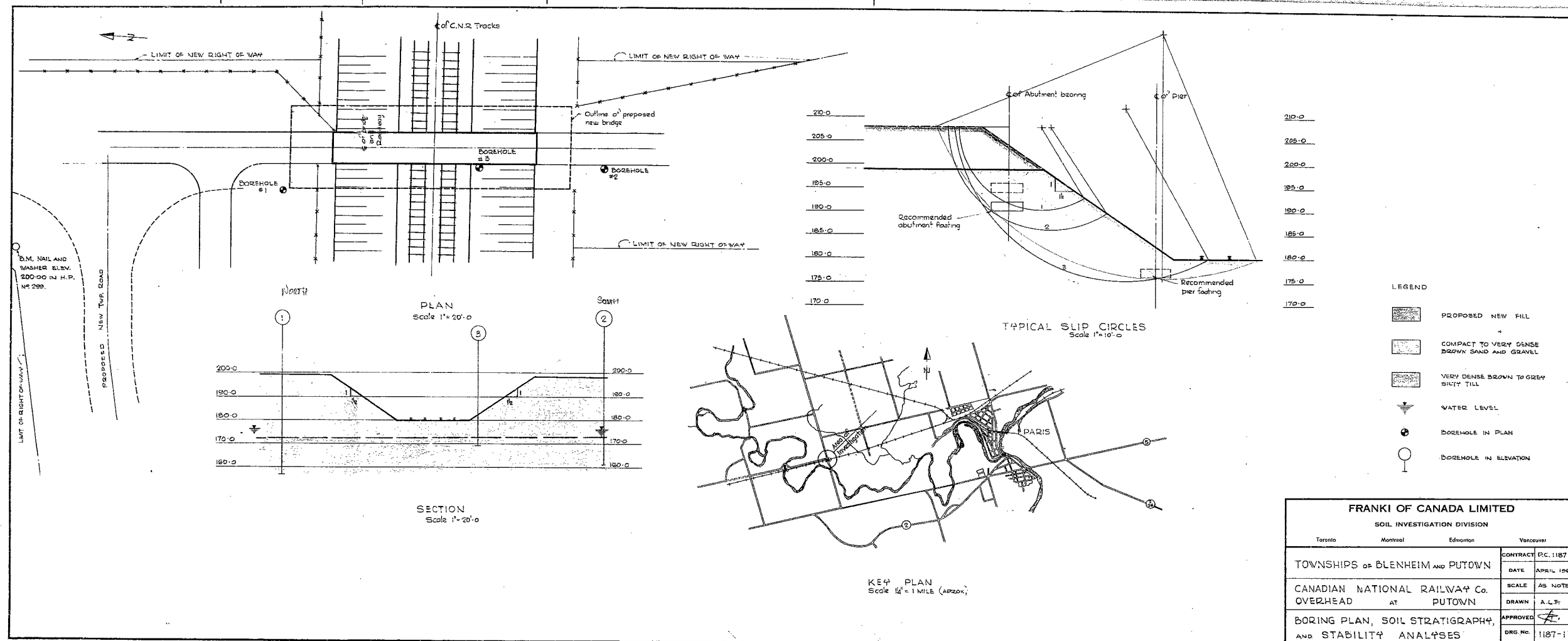
AS AUGER SAMPLE  
DO DRIVE OPEN  
DF DRIVE FOOT VALVE  
SD SLEEVE OPEN  
SF SLEEVE FOOT VALVE  
TO THIN WALLED OPEN  
TP THIN WALLED PISTON  
WS WASHED SAMPLE

RC ROCK CORE  
K<sub>p</sub> FIELD PERMEABILITY TEST  
G<sub>w</sub> GROUND WATER LEVEL  
AT TIME OF BORING

REMARKS

O WATER CONTENT  
X ATTENDING LIMITS  
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K<sub>p</sub> PERMEABILITY FIELD  
K<sub>u</sub> PERMEABILITY LAB.  
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P COMPACTION

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



FRANKI OF CANADA, LIMITED

SOIL INVESTIGATIONS

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

R E P O R T

to

TOWNSHIPS OF BLENHEIM AND SOUTH DUMFRIES

on

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED C.N.R. OVERHEAD AT PUTOWN

TOWNSHIP OF BLENHEIM, COUNTY OF OXFORD

ONTARIO

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

Our Reference  
PC 1187

April 16, 1963

## INTRODUCTION

Franki of Canada has been retained by C. C. Parker and Associates Limited, on behalf of the Townships of Blenheim and South Dumfries, to carry out a soil investigation at the site of a proposed new bridge over the Canadian National Railway tracks in Lot 1, Concession 1 in the Township of Blenheim, County of Oxford, near a community locally known as Putown.

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 fieldwork was carried out on March 26 and 27 and April 3, 1963 and consisted of three detailed boreholes, two of which with an adjacent dynamic cone penetration test. The location of the borings are shown on Drawing 1187-1, which also shows a section of the inferred soil stratigraphy. A detailed log for each boring is 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 Appendix I. Samples remaining after testing will be stored until November 1, 1963 and then discarded unless other instructions are received.

Elevations referred to in this report are related to a local datum, consisting of a nail and washer in Hydro Pole No.299 on the north-west corner of the road intersection, just north of the bridge. The elevation of this benchmark has been taken as

2.

200.00 and approximately compares with Geodetic Elevation 877.

### SOIL CONDITIONS

The main soil strata at the site are brown sand with gravel, underlain by grey glacial till.

The sand with gravel stratum extends from the ground surface to about elevation 172 and has a corresponding thickness of about 26 feet at the top of the cut and about 8 feet at the bottom of the cut. The sand is generally slightly to moderately organic in the upper 7 feet in boreholes 1 and 2. Individual sizes of the sand and gravel are subangular to angular. From the Boring Records of boreholes 1 and 2, it can be seen that although the dynamic cone penetration tests are comparable, the results of the standard penetration tests are very different. From the ground surface to about elevation 182, the material in borehole 1 has 'N' values between 30 and 50 blows per foot, indicating a dense structure. In borehole 2 however, over the same depth, the 'N' values are of the order of 2 to 5 blows per foot, suggesting a very loose density, which is not borne out by the results of the dynamic cone penetration test. Since the tests were carried out in both boreholes in the same way, and since no variation is suggested from geological evidence, grain-size analyses were carried out for further comparison. The grainsize distributions within the relevant depth are given on Figures 1 and 2 of Appendix I for boreholes 1 and 2, respectively. It can be seen that the material in borehole 1 is not only coarser than that in borehole 2, but also better graded. When overburden is removed, as in a borehole, the density obtained from the 'N' value in standard sampling in uniform materials tends to be conservative. It is therefore considered that the density suggested by the 'N' values

in borehole 2 is too low and should be closer to the results indicated by the dynamic cone penetration test.

At depth, the sand and gravel becomes more erratically graded, as may be seen from Figure 3. The natural moisture content of the stratum is low and is of the order of 3 to 5 percent.

The sand and gravel is underlain by a stratum of glacial till, extending from about elevation 170 to the depth explored. The till is composed mainly of silt, which grades into clayey silt and silty clay with depth and is mixed with some sand and gravel. Two typical grainsize distribution curves are given on Figure 3, one for the silty part of the stratum and one for the more clayey part at depth. The upper part of the till is brown in colour, as a result of oxidation, the remainder is grey. The till is very dense, as indicated by the results of the standard penetration tests, which gave 'N' values in excess of 100 blows per foot. The natural moisture content determined on one sample was 17 percent.

#### WATER CONDITIONS

The ground water level was observed throughout the period of the investigation and water level observation pipe was inserted in boreholes 1 and 2, after completion of the sampling. The boreholes remained dry during the sampling operations but ground water entered the holes soon afterwards and stabilized within 24 hours. The stabilized ground water level at the site was found to be at elevation 176, or about 4 feet below the bottom of the cut.

There is a possibility that the observed water level is a perched level. Because of the low permeability of the till, however, this perched water for practical purposes would act in



excavations in a similar way as true ground water. In addition, the excavation will probably have to be sheeted in any case, to satisfy Railway regulations. No attempt was therefore made to further define the true ground water level.

## DISCUSSION

### a. General

The railway tracks at the road intersection are in a cut about 18 feet deep. At present the road is carried over the tracks by a wooden structure. This structure will be replaced, and at the same time the roadway will be raised to accommodate a new proposed grade. The new bridge will be a simply supported three-span reinforced concrete structure, about 35 feet wide. The centre span will be about 40 feet long and the approach spans about 30 feet. The piers will be of the portal type and the abutments of the closed front type with wingwalls. From the available loading information it is known that the maximum dead plus live loadings will be of the order of 500 and 650 kips for the north and south abutments, respectively, using presently planned foundation elevations of 194 and 191.5, respectively. The maximum pier loading will be of the order of 800 kips.

The new roadway will be higher than the present one and the corresponding heights of fill required are about 9 and 7 feet at the north and south sides of the cut, respectively.

The sand and the till are very suitable materials for the adoption of spread footing type foundations. The pier and abutment footings are discussed in the following paragraphs.

b. Pier Foundations

The main consideration affecting the pier foundations is the requirement of frost protection. Although the sand is not seriously subject to frost action, as indicated by the grainsize distribution curves, it is recommended that at least 4 feet of earth cover be provided. The bottom of the cut is at about elevation 180 and in consideration of the above, the maximum foundation level is at elevation 176. Because of the required minimum distance of 11 feet between the pier and the centre line of the tracks, the earth cover requirement is partly satisfied by the slope of the cut. However, the minimum earth cover occurs beneath the bottom of the cut, which determines the foundation level at or below elevation 176. At this elevation the sand stratum is very dense as indicated by the results of boreholes 1 and 2. Borehole 3 indicates a lower density at this elevation but this is partly due to overburden effects. It is recommended that for foundation design, an allowable bearing pressure of 6000 pounds per square foot be used for square or strip footings up to about 6 feet in width. This allowable pressure allows for the proximity of the ground water level. In the present design each leg of the pier will be carried by an individual footing. Because of the proximity of the railway tracks, a rectangular footing will probably be required. Based on the available loading information, it is considered that a footing about 6 by 11 feet in size would be satisfactory. When the footing width chosen is in excess of 7 feet, the allowable bearing pressure should be reduced to 5000 pounds per square foot. Under the allowable bearing capacities, total settlement should be very small and differential settlement well within limits for the structure.

c. Pier Excavations

The recommended foundation elevation 176 coincides with the elevation of the ground water level as measured at the time of the investigation. As discussed under "water conditions", this observed level may be perched. In any case the investigation was carried out in the season in which maximum ground water movement may be expected and it is therefore considered that during construction either similar or better conditions would be encountered. In this respect, it is advisable to check the ground water conditions prior to calling for tender, so that it will be known within reasonable limits whether ground water will be encountered.

When, during construction, ground water is at or above the foundation, it must be lowered temporarily. Because of the relatively high permeability of the sand stratum, it will not be practical to lower the water level by gravity methods and/or pumping. It is therefore recommended that the excavation be sheeted. Sheet piling should be closed and should extend into the till stratum, to about elevation 170. The till is of very low permeability and for practical purposes will prevent entry of ground water.

Because of the proximity of the railway tracks, it will probably be necessary in any case to sheet the excavation to prevent loss of ground under the tracks. It will further be necessary to brace at least three sides of the excavation. The sheet piling and bracing should be designed to withstand earth pressure and the surcharge induced by trains. It is recommended that for these heavily-used tracks the horizontal coefficient of earth pressure, including surcharge, be taken as 1.0. During construction trains should further be slowed down to minimize vibration.

#### d. Abutment Foundations

In the present design, the footing of the north abutment is 9 by 34 feet in size and for the south abutment 7 by 34 feet. Using the available loading information, the uniformly distributed pressure beneath the north and south abutment footings would be 2100 pounds per square foot. Including the moment distribution due to eccentricity, the theoretical maximum and minimum pressure, using a linear pressure distribution are about 4000 and 100 pounds per square foot. In practice a linear pressure distribution is not correct, because the edge of the foundation farthest away from the point of load application no longer contributes to the bearing capacity. In other words, the active width of the foundation is reduced and the load tends to become equally distributed over this reduced width. For design, this reduced width should be taken as the full width minus twice the eccentricity. Using this approach, the uniformly distributed pressure will be of the order of 3000 pounds per square foot. For the footing width under consideration, this bearing pressure can be allowed at and below elevation 196 at the location of borehole 1 and at or below elevation 190 at the location of borehole 2. Settlements under this pressure should be very small and insignificant for the simply supported structure.

#### e. Abutment Stability

The foundation elevation is not only influenced by the bearing capacity of the soil but also by stability considerations. At the present time, the cut is about 18 feet deep and has slopes of 1 vertical to 1½ horizontal. In the proposed design the depth of the cut will be increased by fill at the top to a total of about 27 feet. The stability of the slope and the abutment with

this additional surcharge was investigated.

Both sliding wedge and circular arc type of sliding surfaces were investigated. Drawing 1187-1 shows two typical modes of failure investigated. Circles 1 and 2 are slope circles and determine the local stability of an abutment. Circle 1 shows the abutment founded at elevation 194. The angle of frictional resistance of the sand was taken as 30 degrees, allowing for the apparent looser condition at the south end of the structure. The factor of safety thus obtained, was 1.19.

The minimum desirable factor of safety is of the order of 1.2. Where a structure is placed in the slope, as in this case, the factor of safety should be raised to at least 1.25. This may be achieved by increasing the span, but probably more economically by lowering the abutment. Circle 2 on the Drawing is the typical sliding surface when the abutment is lowered to elevation 190. The factor of safety obtained for this case was 1.23. Although this is somewhat less than desirable, it is considered that this design will be adequate, since the frictional resistance of the sand has been taken conservatively, resulting in a consequently conservative factor of safety.

In addition, a deeper sliding surface was considered, including both an abutment and a pier. Circle 3 is the typical sliding circle for this case. The factor of safety obtained was in excess of 2.0 and therefore satisfactorily.

In conclusion, it is recommended that both on the north and south sides of the cut the abutment be founded at elevation 190.

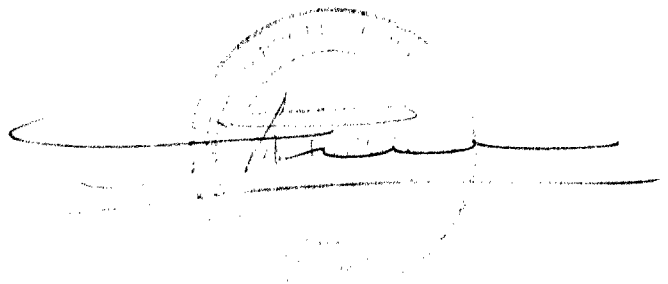
It is important at this point to emphasize that

sufficient provision should be made to drain the abutments so that no water can accumulate behind the abutment and increase the overturning forces.

#### CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by compact to very dense sand with gravel extending to elevation 170, underlain by very dense silty till.
2. The ground water level at the time of the investigation was at elevation 176, which may possibly represent a perched level.
3. Piers may be founded at elevation 176, using an allowable bearing pressure of 6000 pounds per square foot.
4. It is recommended that the abutments be founded at elevation 190, using a bearing pressure of 3000 pounds per square foot.
5. Settlements under the piers and abutments will be very small and insignificant for a simply supported structure.
6. Pier excavations should be sheeted both to minimize ground water problems and to prevent loss of ground under the railway tracks. Sheet piling should extend into the till stratum.

FRANKI OF CANADA LIMITED



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

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.

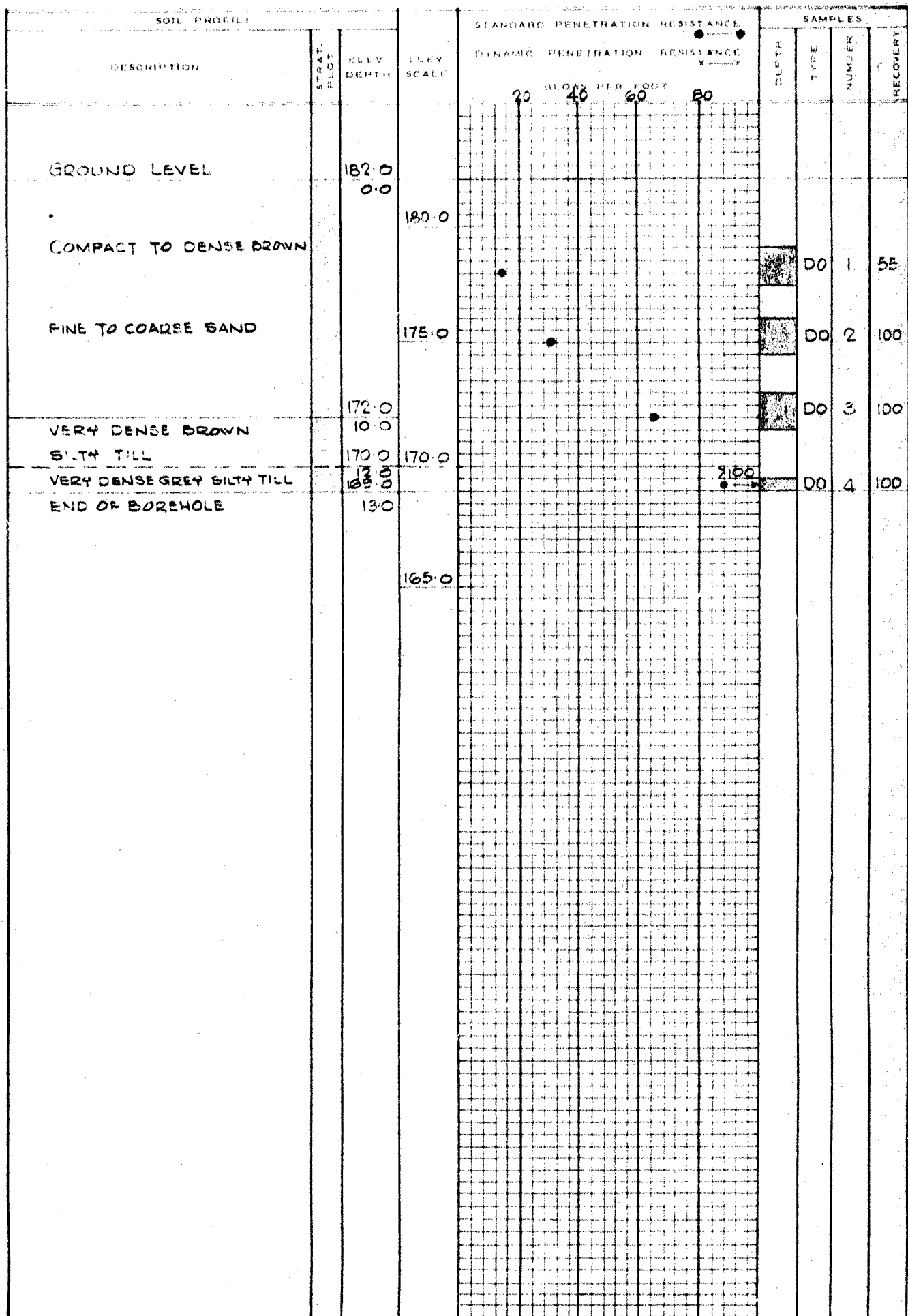
SAMPLES

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 PC1187 BORING 3 BORING DATE APRIL 2 1963  
 DATUM LOCAL DIAM. 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<sub>F</sub> FIELD PERMEABILITY TEST  
 ▼ GROUND WATER LEVEL  
 AT TIME OF BORING

REMARKS

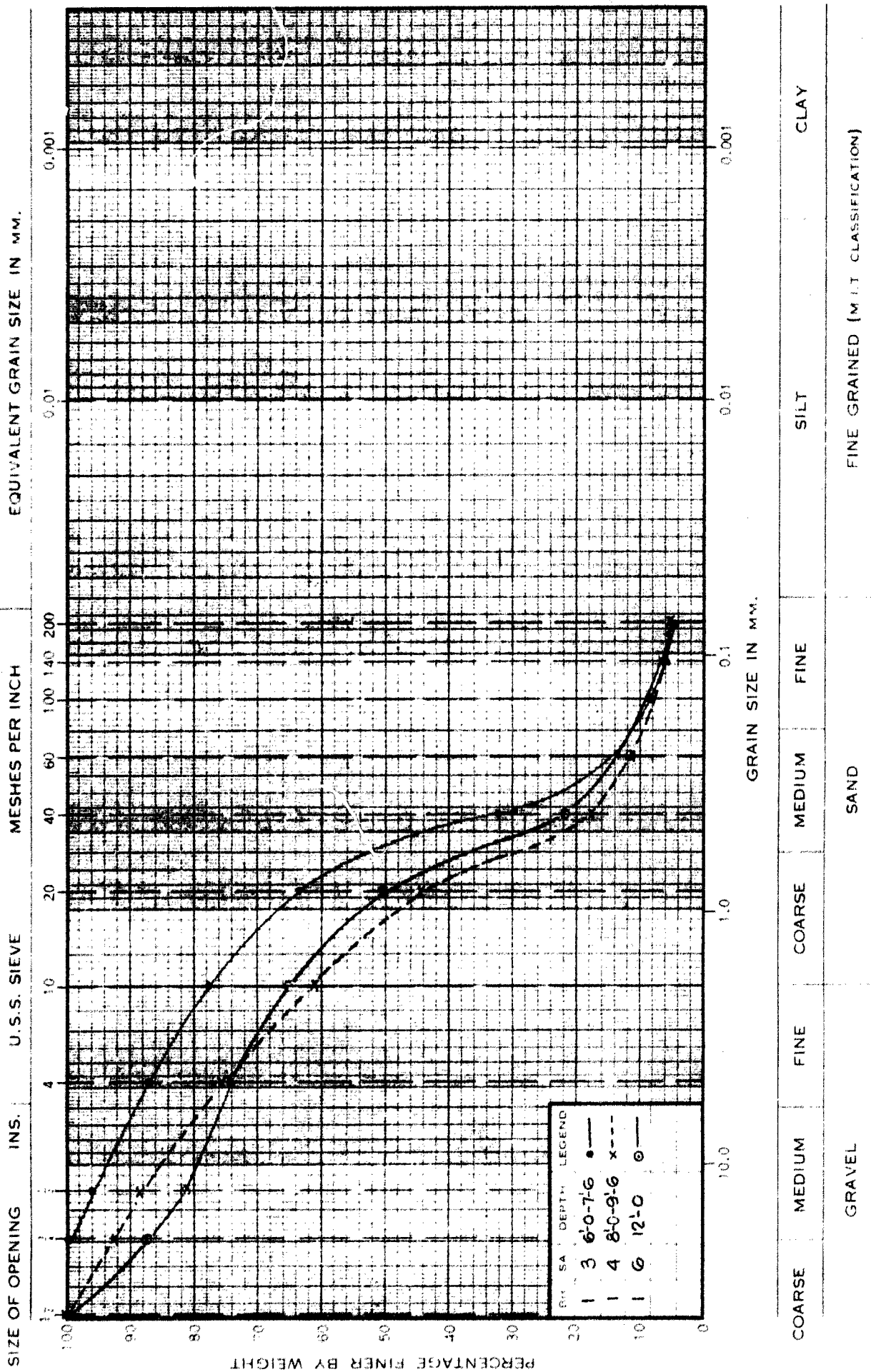


FRANKI OF CANADA LIMITED  
GRAIN SIZE DISTRIBUTION

APPENDIX

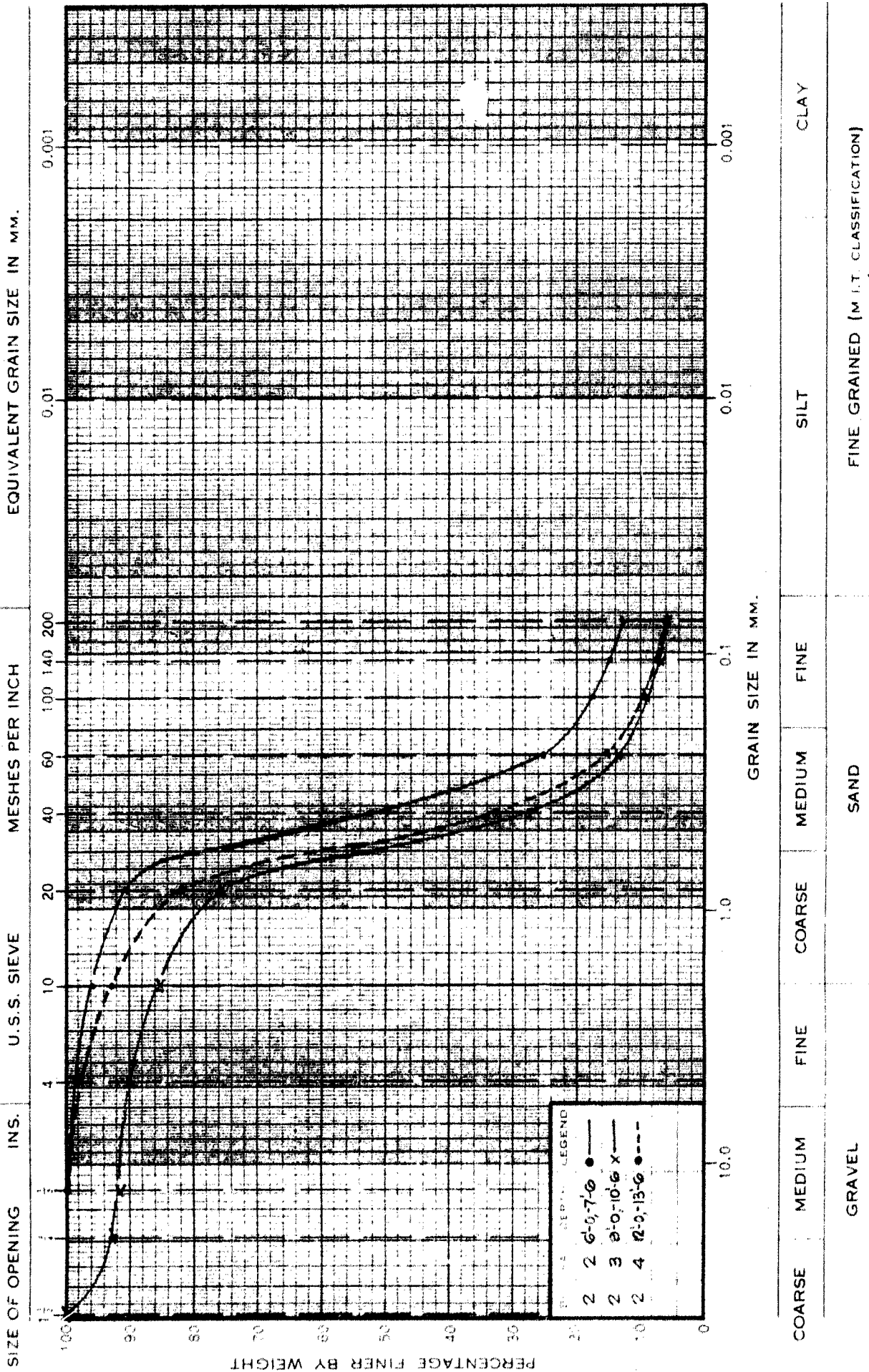
FIGURE

CONTRACT PC 1187



FRANKI OF CANADA LIMITED  
GRAIN SIZE DISTRIBUTION

APPENDIX 1  
FIGURE 2  
CONTRACT PC 1187



FRANKI OF CANADA LIMITED  
GRAIN SIZE DISTRIBUTION

APPENDIX 1  
FIGURE 3  
CONTRACT PC 1187

