

Mr. C. Grebski,
Bridge Design Engr.,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attention: Mr. A. Radkowski

December 15, 1965

Hamilton Drive Underpass,
Hwy. #403, W.P. 184-60,
District #4 (Hamilton).

We have reviewed the bridge design drawings D 5742-1 & 2, showing the design loads for spread footings and piled foundations for the above-mentioned structure, and submit the following comments:

The differential settlements between the footings on piles and spread footings will be within tolerable limits for a continuous type of structure. In our opinion, the maximum differential settlement will be in the order of one inch.

AGS/MdeP

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office ✓
Gen. Files

P.S. -- Drawings returned herewith.

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario.

DATE: September 13, 1965.

OUR FILE REF.

IN REPLY TO

SUBJECT:

Hamilton Drive Underpass,
Hwy. 403, W.P. 184-60,
District No. 4.

Enclosed please find a print of our Preliminary Bridge Plan D 5742-P1 for the above structure.

Would you please inform us if you have any comments or let us have your approval if the plan is satisfactory.

WSM/ag

W. S. Melinyshyn
W. S. Melinyshyn,
Regional Bridge Location Engineer.

A meeting was held in the Bridge Office between Foundation Section and Bridge Design Section regarding the above mentioned job on Sept 21/65. We have advised about the possibility of spread footings for Abutment and Pier #1 locations. The Bridge Section will study this alternative and finalize the scheme.

Dr. Devata
Sept 21/65

OVER

FOOTING ELEVATION — 794

BH 1

$N = 100$

Footing width Soil Pressure

5

6 + t.s.f.

NORTH PIER

ELEV. 794

BH 1 $\frac{1}{2}$ 3

Same as N. abutment

centre PIER

BH 3, ELEV. 782 / $N = 20 ?$

Soil pressure = 1.5 t.s.f. due to 5' width

+ = 1.5 t.s.f.

3.0 t.s.f.

SOUTH ABUTMENT

ELEV. 780, BHS 3 & 5

N = 20

DITTO for SOUTH PIER

DETERMINE PENETRATION FOR DESIGN LOAD
OF 40 tons FOR CONDITIONS AS FOLLOWS

HAMMER - D-12

DRIVING ENERGY = 22,600 FT-LBS

PILE 12 BP53, 50' LENGTH

WT. OF STEEL ANVIL \approx 700 LBS.

S.F. = 3

ULTIMATE LOAD = 3(40) = 120 tons

DETERMINE PENETRATION (S) FOR ANY VALUE
OF MEASURED REBOUND.

HILEY FORMULA — DD-1213

$$P = \frac{50(53)}{2000} + \frac{700}{2000}$$
$$= 1.675$$

$$W = 1.38$$

$$\frac{P}{W} = \frac{1.675}{1.38} = 1.21$$

C	S		
INCHES	INCH/BLOW	BLOWS/INCH	BLOWS/FOOT
0.1	0.45		
0.2	0.40	2.22	27
0.3	0.35	2.5	30
0.4	0.30	2.86	34 ← MOST PROBABLE
0.5	0.25	3.33	40
		4	48

DETERMINE LOAD CARRYING CAPACITY (Q_f)
 OF 12BP53

BY MEYERHOF $Q_f = 4NA_f + \frac{\bar{N}A_s}{100}$

$$Q_f = \frac{\bar{N}A_s}{100} = \frac{10(1)(6)(10)}{100} = 6.0$$

$$Q_f = \frac{20(1)(6)(12)}{100} = 14.4$$

$$Q_f = \frac{30(1)(6)(20)}{100} = 36.0$$

$$Q_f = 4NA_f = 4(100)\left(\frac{15.6}{144}\right) = 0.4$$

$$\leq Q_f$$

$$= \underline{\underline{56.8 \text{ tons}}}$$

792

$\bar{N} = 10$

782

$\bar{N} = 20$

770

$\bar{N} = 30$

750

AT 750

$N = 100$

12BP53

72
10
1440

180
20
3600

52.4
14.4

P.C. 963 Pile Capacity

Assume $F_s = A_s K \bar{\sigma} \tan \phi$ $F_p = \pi r^2 (\gamma D_f N_q + 0.6 \gamma + N_f)$

Assume round pile batt 8" top 6" $\bar{\sigma} = 7"$

$A_s = \pi \frac{7}{12} = 1.83 \text{ ft}^2 / \text{ft length}$ $A_p = \pi r^2 = \pi \left(\frac{3}{12}\right)^2 = 0.70 \text{ ft}^2$

Assume $\gamma = 130$ $\gamma' = 70 \text{ pcf}$

$\phi \approx 35 \rightarrow N_q = 40$ $N_f = 40 \text{ ultimate}$

$\therefore Q_p = \pi \left(\frac{3}{12}\right)^2 (70 \times D_f \times 40 + 0.6 \times 70 \times \frac{3}{12} \times 40) = [550 D_f + 80] \text{ pounds}$

Assume $K = 1.0$ after driving $\phi = 35 \rightarrow \tan \phi = .70$

$\therefore Q_s = 1.83 \times 1.0 \times .7 \times \gamma h \times \frac{1}{2} l = 1.83 \times 1.0 \times .7 \times 70 \times \frac{1}{2} l^2 = 44.8 l^2$

$D_f = l = 20' \rightarrow Q_u = 550 \times 20 + 44.8 \times 400 + 80 = 29080 \# = 14.5 T$

$D_f = l = 30' \rightarrow Q_u = 550 \times 30 + 44.8 \times 900 + 80 = 56960 \# = 28.5 T$

$D_f = l = 40' \rightarrow Q_u = 550 \times 40 + 44.8 \times 1600 + 80 = 93880 \# = 46.9 T$

12 inch cylindrical pile $\rightarrow A_s = \pi \times 1 = 3.14 \text{ ft}^2$ $A_p = \frac{1}{4} \pi \times 1 = 0.78 \text{ ft}^2$

$Q_p = 0.78 (70 \times D_f \times 40 + 0.6 \times 70 \times \frac{1}{2} \times 40) = [2200 D_f + 660] \text{ pounds}$

$Q_s = 3.14 \times 1.0 \times 0.7 \times 70 \times \frac{1}{2} l^2 = 77 l^2$

$D_f = l = 20' \rightarrow Q_u = 2200 \times 20 + 77 \times 400 + 660 = 75460 \# = 37.7 T$

$D_f = l = 30' \rightarrow Q_u = 2200 \times 30 + 77 \times 900 + 660 = 135960 \# = 68 T$

$D_f = l = 40' \rightarrow Q_u = 2200 \times 40 + 77 \times 1600 + 660 = 211660 \# = 106 T$

Example: First assume excavated base then compare drive base to the Meyerhof.

Say base $\phi = 2'$ and shaft $\phi = 20' = 1.7'$

$$A_s = \pi \times 1.7 = 5.3 \text{ ft}^2 \quad A_p = \frac{1}{4} \pi \times 4 = 3.14 \text{ ft}^2$$

$$Q_p = 3.14 (70 \times D_p \times 40 + 0.6 \times 70 \times 185 \times 40) = 8800 D_p + 4500$$

$$Q_s = 5.3 \times 1.0 \times 0.7 \times 70 \times \frac{1}{2} L^2 = 130 L^2$$

$$D_p = L = 10' \rightarrow Q_u = 8800 \times 10 + 130 \times 100 + 4500 = 105500 \# = 53 T$$

$$D_p = L = 20' \rightarrow Q_u = 8800 \times 20 + 130 \times 400 + 4500 = 232500 \# = 116 T$$

$$D_p = L = 30' \rightarrow Q_u = 8800 \times 30 + 130 \times 900 + 4500 = 365500 \# = 193 T$$

Drive based on pile Meyerhof: base = $2' \phi$ including $F_s = 3$

$$D_p = 10' \rightarrow \text{safe load} = 25 T$$

$$D_p = 20' \rightarrow \text{safe load} = 75 T$$

$$D_p = 30' \rightarrow \text{safe load} = 160 T$$

Now assume $\phi = 37.5^\circ \rightarrow M_x = 60$, $M_y = 70$ $\phi = 77$ $K = 1.0$

Minor Axis

$$Q_s = 1.83 \times 1.0 \times .77 \times \frac{1}{2} l^2 = \frac{1}{2} l^2 = 1.83 \times 1.0 \times .77 \times 70 = \frac{1}{2} l^2 = 49.4 l^2$$

$$Q_f = \pi \left(\frac{3}{12} \right)^2 \left(70 \times D_f \times 60 + 0.6 \times 70 \times \frac{3}{12} \times 70 \right) = [820 D_f + 150] \text{ pounds}$$

$$D_f = l = 20' \rightarrow Q_u = 820 \times 20 + 49.4 \times 400 + 150 = 36340 \# = 48 T$$

$$D_f = l = 30' \rightarrow Q_u = 820 \times 30 + 49.4 \times 900 + 150 = 69240 \# = 35 T$$

$$D_f = l = 40' \rightarrow Q_u = 820 \times 40 + 49.4 \times 1600 + 150 = 111950 \# = 56 T$$

12" concrete gph.

$$Q_f = 0.78 \left(70 \times D_f \times 60 + 0.6 \times 70 \times \frac{1}{2} \times 70 \right) = [3300 D_f + 1150] \text{ pounds}$$

$$Q_s = 3.14 \times 1.0 \times 0.77 \times 60 \times \frac{1}{2} l^2 = 84.6 l^2$$

$$D_f = l = 20' \rightarrow Q_u = 3300 \times 20 + 84.6 \times 400 + 1150 = 100990 \# = 50 T$$

$$D_f = l = 30' \rightarrow Q_u = 3300 \times 30 + 84.6 \times 900 + 1150 = 176290 \# = 88 T$$

$$D_f = l = 40' \rightarrow Q_u = 3300 \times 40 + 84.6 \times 1600 + 1150 = 268150 \# = 134 T$$

2' beam casing (uncracked)

$$Q_f = 3.14 \left(70 \times D_f \times 60 + 0.6 \times 70 \times 0.85 \times 70 \right) = 13200 D_f + 7800$$

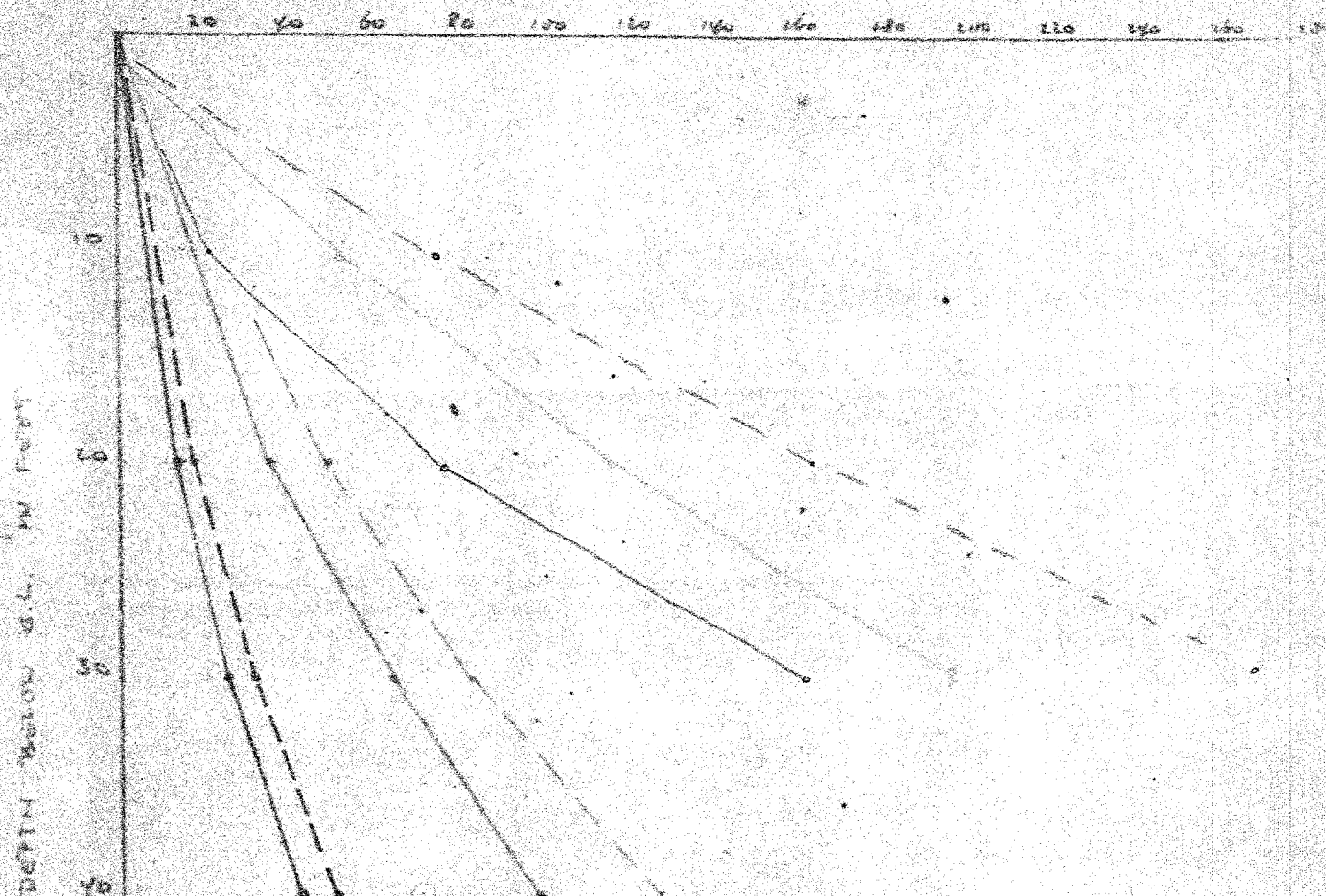
$$Q_s = 5.3 \times 1.0 \times 0.77 \times 70 \times \frac{1}{2} l^2 = 143 l^2$$

$$D_f = l = 10' \rightarrow Q_u = 13200 \times 10 + 143 \times 100 + 7800 = 154100 \# = 77 T$$

$$D_f = l = 20' \rightarrow Q_u = 13200 \times 20 + 143 \times 400 + 7800 = 329000 \# = 164 T$$

$$D_f = l = 30' \rightarrow Q_u = 13200 \times 30 + 143 \times 900 + 7800 = 532500 \# = 266 T$$

CAPACITY IN TONS



LEGEND:

- $\phi = 35^\circ$
- wood pile 8" butt 6" tip — ultimate capacity — static formula
 - concrete cyl. pile 12" — ultimate capacity — static formula
 - driven caisson 2' dia base using Meyerhoff $F_0 = 4$
 - installed caisson 2' dia base — ultimate capacity — static formula

--- } DITTO, BUT TAKING
 --- } $\phi = 37\frac{1}{2}^\circ$

PILE CAPACITY

Class B Round Timber Piles

Min. Tip Dia. = 8"

Min. Butt. Dia = 12"

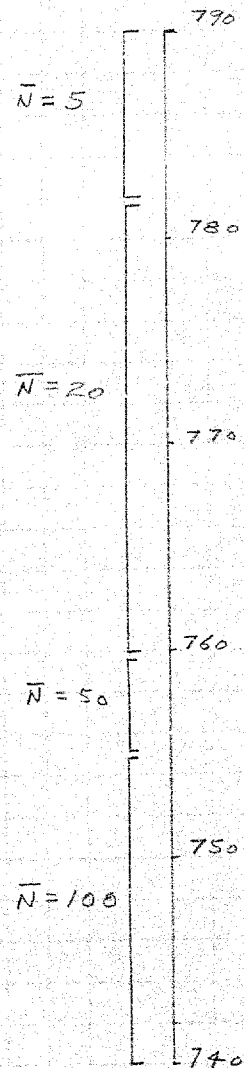
Length = 40 ft.

Meyerhof

$$Q_f = 4NA_p + \frac{\bar{N}A_s}{50}$$

$$A_p = \frac{\pi (9.25)^2}{4 \cdot 144} \quad (\text{at lower } \frac{2}{3} \text{ point})$$

$$= 0.467 \text{ ft}^2$$



$$Q_f = 4(100)(0.467) + 5(\pi) \frac{(12)(8)}{(12)(50)} + 20(\pi) \frac{(10)(22)}{(12)(50)} + 50(\pi) \frac{(8)(5)}{(12)(50)}$$

$$= 187 + 2.5 + 23.0 + 10.4$$

Upper limit of skin friction = 1 ton per sq ft.

$$Q_f = 187 + 2.5 + 23.0 + 10.4$$

$$= 223 \text{ tons}$$

$$Q_A = \frac{224}{3} \div \underline{74} \text{ tons}$$

Wt. of Hammer (striking part) = 3500 LBS

Hammer with energy $\approx 12000 \text{ ft-Lbs} = Wh$

Allowable load on pile = 25 tons = R

coefficient of restitution $e = 0.35$ (assumed)

length of pile being driven = 40 ft.

Weight of driving anvil = 700 lbs.

Density of timber pile = 30 p.c.f.

Determine penetration per blow (s) for any assumed value of (c).

Determine efficiency of blow (n) from

$$n = \frac{W + e^2 P}{W + P}$$

P = wt of pile + wt of anvil

$$= \frac{\pi (10)^2 (40)(30)}{4 \cdot 144} + 700$$

$$= 655 + 700$$

$$= 1355 \text{ LBS}$$

$$n = \frac{3500 + (0.35)^2 (1355)}{3500 + 1355} = 0.75$$

$$s + \frac{c}{2} = \frac{4 n W h}{R}$$

$$= \frac{4 (0.75) (12000)}{25 (2000)}$$

$$= .72$$

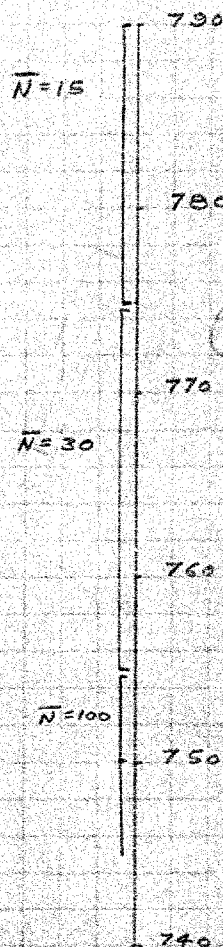
$$s = 0.72 - \frac{c}{2}$$

for $c = 0.5$ $s = 0.47''$ per blow.

say $\frac{1}{2}''$ per blow for conditions as above.

Data from BH 3.

Determine allowable load
per pile Q_A by
Meyerhof formula.



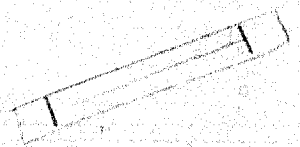
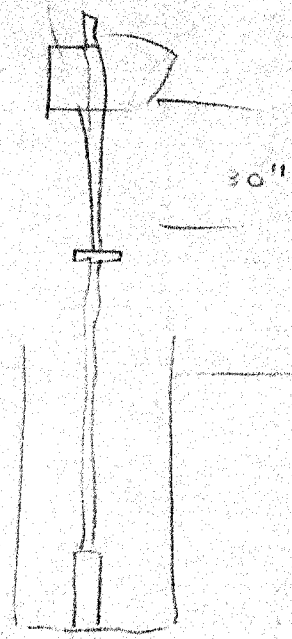
$$Q_f = 4(100)(0.467) + 15(\pi) \frac{(12)(15)}{(12)(50)} + 30(\pi) \frac{(10)(20)}{(12)(50)}$$

$$= 187 + 141 + 314$$

$$= 232.5$$

$$Q_A = \frac{232.5}{3} = \underline{\underline{77 \text{ tons}}}$$

140 LB
↓ 30"



Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

February 8, 1961.

REVIEW OF PRELIMINARY PLAN
by: Foundations Office.

Attention: Mr. F. Gormek.

Re: Ancaster Township Underpass,
W.P. 18th-60, Hwy. #403,
District #4.

The Preliminary Plan D 4821-P for the above structure, has been received and reviewed. We are submitting the following comments for your consideration:-

The plan shows the North abutment and Pier 'A' supported by spread footings. Using an allowable bearing pressure of 3 t.s.f., the safety of the foundation with respect to complete failure, can be assured. However, with the information available, it is not possible to estimate the probable settlement. Because differential settlement must be avoided with this type of structure, it is recommended that all piers and abutments be supported on friction piles. With 12 BP 53 as shown on the preliminary plan, an allowable load of 40 tons per pile can be used in the design of the structure.

The information given in the log of Borehole 1 of the foundation report, indicates that the subsoil between elevations 780 and 790 may be difficult to penetrate with piles. It is thought, however, that with the use of small displacement piles such as the recommended 'H' piles, and a driving energy of 22,600 ft.-lbs. (Delmag D-12), this layer can be penetrated. The piles should be driven to at least elevation 750. The driving can then be stopped when the penetration is about 36 blows per foot using a driving energy of 22,600 ft.-lbs.

cont'd. /2 ...

If there are any further questions regarding the foundation considerations for this structure, please contact our Office.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.

Per:

R. J. Salvas
for (R. J. Salvas,
PROJECT FOUNDATION ENGR.)

RJS/MaeF

cc: Foundations Office
Gen. Files.

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

February 8, 1961.

REVIEW OF PRELIMINARY PLAN
by: Foundations Office.

Attention: Mr. F. Gormek.

Re: Ancaster Township Underpass,
W.P. 184-60, Hwy. #403,
District #4.

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cont'd. /2 ...

If there are any further questions regarding the foundation considerations for this structure, please contact our Office.

L. G. Sodeman,
PRINCIPAL FOUNDATION ENGR.

Per:

for M. de Salas
(R. J. Salvas,
PROJECT FOUNDATION ENGR.)

RJS/MdeF

cc: Foundations Office
Gen. Files.

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

December 20, 1960.

FOUNDATION INVESTIGATION REPORT

by: Franki of Canada, Limited.

Attention: Mr. S. McCombie.

Re: Proposed Underpass - Hamilton Drive &
Line 'B' of Hwy. #403, Ancaster, Ont.
District #4.

WP 184-60

Attached to this memo, we are sending you the above mentioned report submitted by Franki of Canada, Ltd. We have reviewed the report and have found the factual field and laboratory data well presented. We have also found the calculations to be correct.

The Consultant has considered both spread footings and footings on piles. As far as footings on piles are concerned, we would recommend that piles be driven down into the very dense layer of silt with gravel where they will meet practical refusal, or 80 blows/ft. Such piles can be loaded with a safe load of 40 tons per pile. No pile loading tests would be required. We would also recommend that serious consideration be given to spread footings first.

Should any other queries arise in connection with this project that you would like to discuss, please do not hesitate to contact our Office.

AOB/ndef

Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Hensay
I. C. Campbell
R. E. Richardson
T. J. Kovich
A. Watt
Foundations Office
Gen. Files.

L. G. Soderman,
PRINCIPAL FOUNDATION ENGR.
Per:

Astermacy
(A. G. Stermac,
FOUNDATION OFFICE ENGR.)

FRANKI OF CANADA, LIMITED

SOIL INVESTIGATIONS

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

R E P O R T

to

DEPARTMENT OF HIGHWAYS, ONTARIO

on

SOIL INVESTIGATION

PROPOSED UNDERPASS

ANCASTER - ONTARIO

Distribution :

10 copies : Department of Highways, Ontario
2 copies : Franki of Canada Limited

Our Reference
OP 21460
PC 953

25th November 1960

I N D E XSOILS REPORT

INTRODUCTION	Page 1
SITE	" 1
PROCEDURE	" 1
SUMMARISED SOIL CONDITIONS	" 2
DISCUSSION	" 2
CONCLUSIONS AND RECOMMENDATIONS	" 6
PERSONNEL	" 6

APPENDIX I

SOIL CONDITIONS	Page I.
WATER CONDITIONS	" IV.
BORING RECORDS	

APPENDIX II

FIGURES - LABORATORY TESTING

APPENDIX III

FIGURE - PILE CAPACITY

APPENDIX IV

PHOTOGRAPHS

DRAWING PC 963-1

BORING PLAN AND SOIL STRATIGRAPHY

INTRODUCTION

Franki of Canada Limited has been retained by the Department of Highways, Ontario, by letter dated November 15th, 1960, to carry out a soil investigation at the site of a proposed underpass at the intersection of Hamilton Drive and the proposed line B of Highway 403, near Ancaster, Ontario.

The object of the investigation was to determine the soil conditions at the site as they affect the foundation design of the proposed underpass and the stability of the approach embankments.

SITE

The site is part of Lot 39, Concession III in the Township of Ancaster, County of Wentworth, and is located about 2 miles south west of Ancaster, Ontario.

Hamilton Drive at the location of the proposed underpass crosses a shallow valley about 10 feet deep. Some photographs of the site are shown in Appendix IV.

PROCEDURE

The field work was carried out between October 19th and 31st 1960 and between November 16th and 18th 1960 and consisted of 6 detailed boreholes with adjacent dynamic penetration tests, together with five additional dynamic penetration tests and one additional shallow augerhole. The latter was put down adjacent to borehole 1, to check the possibility of excess hydrostatic pressures existing at depth. Dynamic penetration tests 1, 3 and 6 were carried out in 2 parts at depth.

The locations of the borings are shown on Drawing PG 963-1 which also shows a section of the inferred soil stratigraphy. A detailed account of the soil conditions encountered together with a detailed log for each boring may be found on the Boring Records in Appendix I.

The results of laboratory testing on soil samples obtained are given on the Boring Records and on the Figures of

2.

Appendix II. Samples remaining after testing will be stored until June 1st, 1961, and then destroyed unless other instructions are received.

Elevations referred to in this report are related to Geodetic Datum. The elevations of the ground surface at the boring locations were obtained from a Department of Highways Benchmark consisting of a nail and washer in the south east root of an elm tree located 70 feet left of station 29+40 of the proposed Highway 403. The benchmark elevation was 814.19.

SUMMARISED SOIL CONDITIONS

The site, outside the area occupied by Hamilton Drive, is generally covered by about 12 inches of topsoil. The topsoil and the roadbed near the centre of the site are underlain by about 5 to 11 feet of very loose to compact organic sandy silt. Beneath the organic sandy silt stratum and beneath the topsoil outside the low lying area is a stratum of compact to very dense silty fine sand, 17 to 36 feet in thickness. The silty sand stratum is underlain by about 11 feet of dense to very dense silt, containing clay pockets and layers. The silt stratum overlies about 3 to 5 feet of very stiff silty clay which, in turn, overlies a stratum of very dense silt with gravel to the depth explored.

DISCUSSION

General

It is understood that the proposed underpass will be a 4 span skewed structure, about 40 feet wide in the direction of the skew. The centre spans will be about 65 feet long and the outside spans about 40 feet long. At the present time, no structural details are available, but it is assumed that the proposed underpass will be a reinforced concrete structure with simply supported spans. It is further estimated that the maximum pier load will not exceed 500 tons. Under these assumptions, the induced pressure per square foot of pier would be about 3 to 4 tons.

3.

The approach embankments will be of the order of 200 feet long and about 30 and 20 feet high at the south and north abutments respectively and presumably constructed of locally obtained material.

Foundations

The organic sandy silt stratum at the site covers most of the area of the proposed underpass. Due to its generally very loose relative density and the considerable organic content the silt stratum is not suitable for foundation purposes.

The silty sand stratum beneath the organic silt is generally dense to very dense at shallow depth beneath its surface. Although the 'N' values in this stratum tend to decrease with depth, it is believed, as discussed in Appendix I, that these lower values are conservative.

It is considered that the silty sand stratum is suitable for the use of spread footing foundations. For adequate frost protection, all footings should be provided with at least 4 feet of earth cover. The recommended foundation elevations and allowable bearing pressures are contained in the following table :

<u>Location</u>	<u>Foundation Elevation</u>	<u>Allowable Bearing Pressure</u>
North abutment	794	3 tons/sq.ft.
North pier	794	3 tons/sq.ft.
Centre pier	782	2 tons/sq.ft.
South pier	780	2 tons/sq.ft.
South abutment	778	2 tons/sq.ft.

The above recommended allowable bearing values are based on the standard penetration resistances obtained and it has been assumed that these values show the effect of saturation below the foundation level and further corrections for saturation were therefore not applied.

The allowable bearing pressures are applicable for footings not exceeding 5 feet in width, which is considered to be

4.

well within design limits.

Under the above conditions, settlements should be minor and any differential settlement that might occur could easily be accommodated by simply supported spans.

Excavations

The above recommended foundation elevations are at considerable depth, up to 14 feet, below present ground level. However, the organic silt near the centre of the site must be excavated in any case for reasons of both obtaining adequate bearing for the proposed Highway 403 and to ascertain adequate stability of the approach embankments.

It is therefore recommended that the existing creek be detoured and the organic silt stratum be removed. When this is done, the additional excavation to footing level will be minor.

However, it is recommended that the creek be detoured in such a way that the ground water level at the construction site be lowered to a minimum possible elevation. If it were possible to lower the ground water level by natural means to below foundation level, excavation to the foundation level could proceed normally. When this cannot be achieved, however, the bottom of the excavation will be affected by excess hydrostatic pressures and it is therefore recommended that in this case the excavation be dewatered. This could be done by a system of well points or by benching and pumping from longitudinal bench trenches. Such bench trenches should be provided with graded filters.

Piled Foundations

Alternatively, piled foundations may be used under the proposed piers and abutments.

Pile capacity computations have been carried out for 2 pile types, a wooden pile with 6 inch tip and 8 inch butt diameter and a 12 inch cylindrical concrete pile.

The results of the computations are shown on Figure 1 of Appendix III, where the computed ultimate pile capacity is

5.

plotted as a function of pile length. The computations were carried out under the following assumptions :

1. Grade of Highway 403 at elevation 799.
2. Bottom of pile cap at elevation 795.
3. Skin friction effective below elevation 795.
4. Initial angle of shearing resistance ϕ equal to 35 degrees.
5. Increase in angle of shearing resistance due to driving from 35 degrees to 37.5 degrees.
6. Coefficient of neutral earth pressure K_0 equal to 1.0 after driving.
7. Angle of skin friction equal to ϕ .
8. Point resistance based on Terzaghi's bearing capacity formula.
9. Submerged unit weight of soil equal to 70 pounds per cubic foot.

It is considered that considerable driving energy is required to obtain the required pile tip elevation.

It is further recommended that at least 2 pile loading tests be carried out to determine the design load.

Approach Embankments

With the grade of Highway 403 at elevation 799 as shown on the Department of Highways profile, the maximum height of the embankment will be about 20-feet.

Before the embankment is constructed, all topsoil and organic sandy silt should be completely removed.

The stability of the embankment at its maximum height was examined and it was found that the factor of safety against sliding is in excess of 1.7 assuming a side slope of 1 vertical to $1\frac{1}{2}$ horizontal, an embankment unit weight of 130 pounds per cubic foot, a K_0 value of 0.5 and an angle of shearing resistance of the subsoil of 30 degrees. Since these values are conservative it is considered that the stability of the embankment is assured.

From the preliminary layout it is understood that the abutments will be of the spill through type and the factor of safety of the end slope should be adequate.

6.

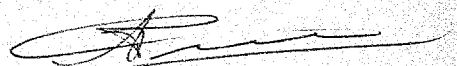
CONCLUSIONS AND RECOMMENDATIONS

1. The site is part of a generally dense to very dense stratified sand and silt stratum, covered in the construction area by up to 11 feet of very loose organic sandy silt.
2. The ground water level at the site is at about elevation 790 corresponding to creek level.
3. It is considered that spread footing foundations could be adopted under allowable bearing pressures of 2 to 3 tons per square foot.
4. Measures to facilitate excavation to footing level are discussed in the report.
5. Alternatively, piled foundations may be used. Ultimate pile capacities have been computed for 2 types of pile, using average conditions across the site, and the results are shown in the report.
6. If changing the regime of the natural drainage at the site would pose a problem, it would probably be most economical to found the north abutment and south pier on spread footings and the remaining piers and abutment on piles.
7. It is recommended that when piled foundations are adopted at least 2 pile load tests be carried out to determine the working load.
8. A pile load test is especially in order, if piles meet refusal at shallow depth, since this refusal may be a temporary phenomenon due to the set up of pore pressures.
9. The organic silt beneath proposed embankments should be completely removed. Under this condition, the stability of the approach embankments was examined and found to be adequate.

PERSONNEL

The field work was carried out under the supervision of Mr. E. Kuhnert. The report was written by Mr. A. Prior.

Yours very truly,
FRANKI OF CANADA LIMITED



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

A P P E N D I X I

SOIL CONDITIONS ... PAGE I.

WATER CONDITIONS ... PAGE IV.

BORING RECORDS ...

APPENDIX I.

PAGE I.

SOIL CONDITIONS

The main soil strata encountered by the borings are as follows :

Topsoil and Road Fill

The site is generally covered by 1 to 6-feet of silty topsoil or sand and gravel road fill depending upon the proximity to Hamilton Drive.

Very Loose to Compact Brown to Grey-brown Organic Sandy Silt

Beneath the topsoil or road-fill in boreholes 3, 5 and 13 and inferred in dynamic penetration tests 4, 8 and 10 is a stratum of organic silt, ranging in thickness from about 5 feet in borehole 13 to 11 feet in borehole 3. The organic material occurs in the form of roots and of dark brown, peat-like pockets and layers up to about $1\frac{1}{2}$ inch in thickness. Although predominantly silty, the stratum contains about 20 to 30 percent fine sand sizes.

Standard penetration or 'N' values ranged from 2 to 13 blows per foot with depth. The average value obtained was about 6 blows per foot. Based on the 'N' values obtained and on the results of the dynamic penetration tests, it is considered that the relative density of the material ranges from very loose to just compact with depth.

The wet unit weight obtained ranged from 81 to 127 pounds per cubic foot at natural moisture contents ranging from 84 to 22 percent.

Compact to Very Dense Brown to Reddish Brown Silty Fine Sand

Beneath the organic silt in boreholes 3, 5 and 13 and beneath the topsoil or road fill in boreholes 1, 11 and 12 is a stratum of predominantly silty fine sand. The presence of the stratum is also indicated in the dynamic penetration tests 4, 6, 7, 8 and 10 by the penetration values and occasional auger samples taken at the location of some dynamic penetration tests.

The stratum ranges in thickness from about 17 feet in borehole 5 to about 36 feet in borehole 11. The stratum is

APPENDIX I.
PAGE II.

generally cross bedded or stratified and contains seams and layers of almost pure silt, up to several inches in thickness. Typical grainsize curves for the stratum are shown on Figures 1 to 8 of Appendix II.

Selected samples from the stratum were examined under a microscope and it was found that the individual sizes are generally of rounded to sub-rounded shape.

'N' values obtained on samples from the stratum ranged from 4 to well over 100 blows per foot, with a median value of about 40 blows per foot. The low 'N' values obtained at depth in the stratum are considered to be conservative due to the relief of overburden pressure and the possibility of complete or partial loss of shearing resistance resulting from temporary excess hydrostatic conditions. It is therefore believed that the relative density at shallow depth below the surface of the stratum is at least dense.

The average wet unit weight obtained was about 134 pounds per cubic foot at an average natural moisture content of about 21 percent.

Dense to Very Dense Reddish Brown Silt with Clay Pockets

Underlying the silty fine sand stratum in boreholes 1, 3, 5 and 11 is a stratum of predominantly silt. Cross bedding and stratification is occasionally in evidence. Seams of silty fine sand up to a few inches in thickness were also encountered. Dispersed throughout the stratum are pockets of grey silty clay about 1/16 to 1/8 inch in diameter. Infrequently, the stratum contains thin seams of reddish brown silty clay or clayey silt. The thickness of the silt stratum where encountered is about 11 feet. Typical grainsize distribution curves are given on Figures 1 to 8 of Appendix II.

Selected samples from the stratum were examined under a microscope and it was found that the shape of the individual grainsize ranges from angular to needle-like.

'N' values obtained ranged from 28 to well over 100

APPENDIX I.

PAGE III.

blows per foot with a median value of 41 blows per foot. Again it is believed that the 'N' values obtained are conservative and it is considered that the relative density of the silt stratum is at least dense throughout. This is confirmed by the results of the dynamic penetration tests.

The average wet unit weight obtained was about 145 pounds per cubic foot at an average natural moisture content of about 22 percent.

Very Stiff Grey Silty Clay

Beneath the silt stratum in boreholes 1, 3 and 5 is a stratum of grey silty clay about 3 to 5 feet in thickness. In borehole 11 at the same elevation as the grey silty clay in boreholes 1, 3 and 5 a layer of reddish brown very clayey silt was encountered about 3 feet in thickness, and this layer has been included in the silty clay stratum. The stratum exhibits no structure, but contains fine gravel dispersed throughout.

An undrained triaxial test carried out on a sample of the stratum at the computed overburden pressure gave an undrained shear strength of about 3000 pounds per square foot at a failure strain of 7 percent. The stress-strain curve obtained is shown on Figure 9 of Appendix II. An Atterberg Limit Determination carried out on this sample gave a liquid limit of 33 percent and a plasticity index of 14 percent. The corresponding wet unit weight and natural moisture content were 131 pounds per cubic foot and 21 percent respectively.

Very Dense Reddish Brown Silt with Gravel

The grey silty clay overlies a stratum of reddish brown silt with gravel, encountered in boreholes 1, 3, 5 and 11. These boreholes were stopped in this stratum. This material exhibits no structure and although the sand and gravel sizes are minor percentage-wise, the stratum could possibly be classified as till. Grainsize distribution curves for the stratum are shown on Figures 1 to 8 of Appendix II.

APPENDIX I.

PAGE IV.

'N' values obtained were generally well in excess of 50 blows per foot, indicating that the stratum is very dense.

The average wet unit weight obtained was about 150 pounds per cubic foot at a corresponding natural moisture content of about 12 percent.

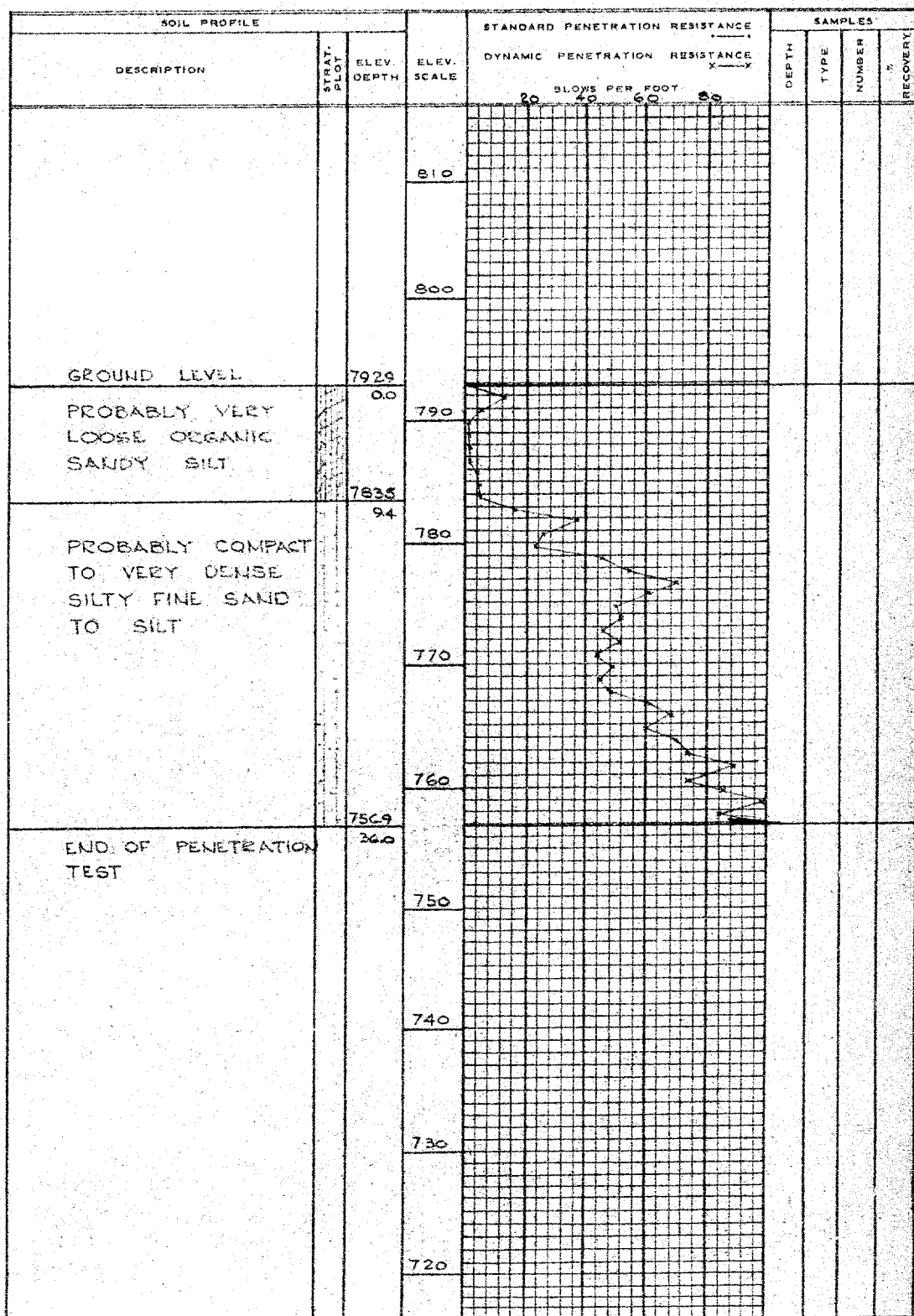
An undrained triaxial test gave an undrained shear strength of about 10,000 pounds per square foot at a failure strain of 7 percent. The failure plane had an inclination of about 60 degrees with the horizontal.

WATER CONDITIONS

Water level observation pipe was inserted in each borehole to facilitate the determination of the ground water level at the time of the investigation. It was found that the ground water level on November 18th 1960 ranged from elevation 790 at boreholes 3 and 5 to elevation 793 in borehole 12. These levels all correspond closely with the water level in the creek which at this time was at about elevation 790.

After completion of borehole 1, the water level in borehole 1 seemed somewhat high which could have possibly been due to surface conditions. To check the possibility of the occurrence of excess hydrostatic pressures at depth, a shallow hole was put down about 10 feet away from borehole 1, to a depth of 15 feet. In a short time, however, the water level in both holes settled at about elevation 793, indicating normal conditions.

CONTRACT PC 963 BORING 4 BORING DATE OCT. 27, 1960
 DATUM GEODETTIC DIAM. HAMMER 140 LBS. DROP 30 IN



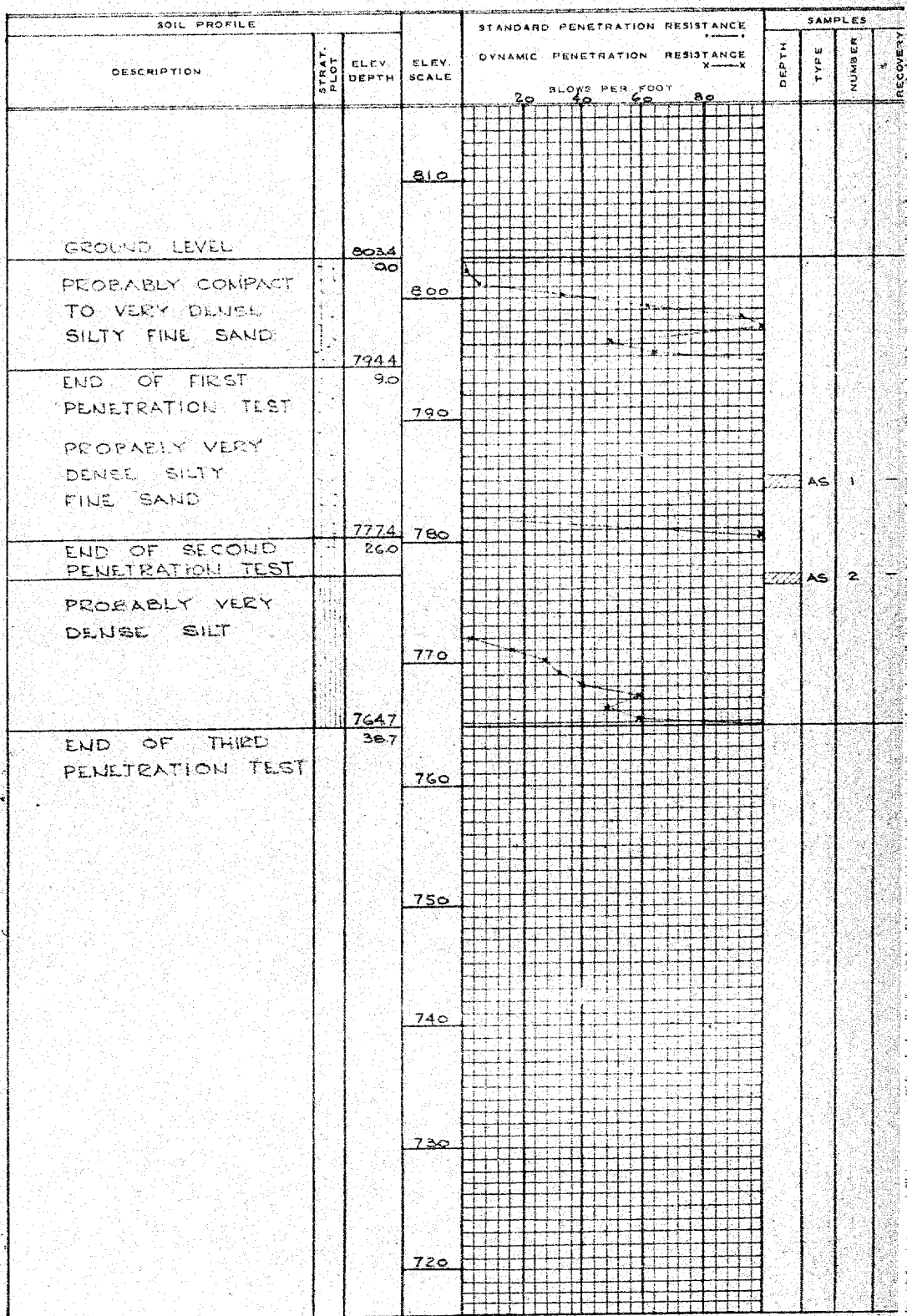
SAMPLE TYPES

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 963 BORING G BORING DATE OCT 24, 1960
 DATUM GEODETTIC DIAM. 4" AUGER HAMMER 140 LBS. DROP 30



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
 T GROUND WATER LEVEL
 AT TIME OF BORING

REMARKS

CONTRACT PC 963 BORING 7 BORING DATE OCT 24, 1960
 DATUM GEODETIC DIAM. 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				
			810					
GROUND LEVEL		801.4						
PROBABLY COMPACT TO VERY DENSE SILTY FINE SAND		00 800						
END OF PENETRATION TEST		794.8						
		66						
		790						
		780						
		770						
		760						
		750						
		740						
		730						
		720						

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 963 BORING 5 BORING DATE OCT 31, 1960
 DATUM GEODETIC DIAM. HAMMER 140 LBS. DROP 30

SOIL PROFILE			STANDARD PENETRATION RESISTANCE		SAMPLES			
DESCRIPTION	SOIL TYPE	ELEV. DEPTH	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE X—X	DEPT	TYPE	NUMBER	RECOVERED
				20 40 60 80 BLOWS PER FOOT				
			810					
			800					
GROUND LEVEL								
PROBABLY VERY LOOSE ORGANIC SILT			790					
PROBABLY VERY DENSE SILTY FINE SAND								
END OF PENETRATION TEST			780					
			770					
			760					
			750					
			740					
			730					
			720					

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 962 BORING 10 BORING DATE OCT 24, 1960
 DATUM GEODETIC DIAM. --- 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							
				20	40	60	80				
			810								
			800								
GROUND LEVEL		791.9									
PROBABLY VERY LOOSE ORGANIC SANDY SILT		00	790								
		782.5									
		94	780								
PROBABLY COMPACT TO VERY DENSE SILTY FINE SAND TO SILT			770								
			760								
		754.9									
		37.0	750								
END OF PENETRATION TEST			740								
			730								
			720								

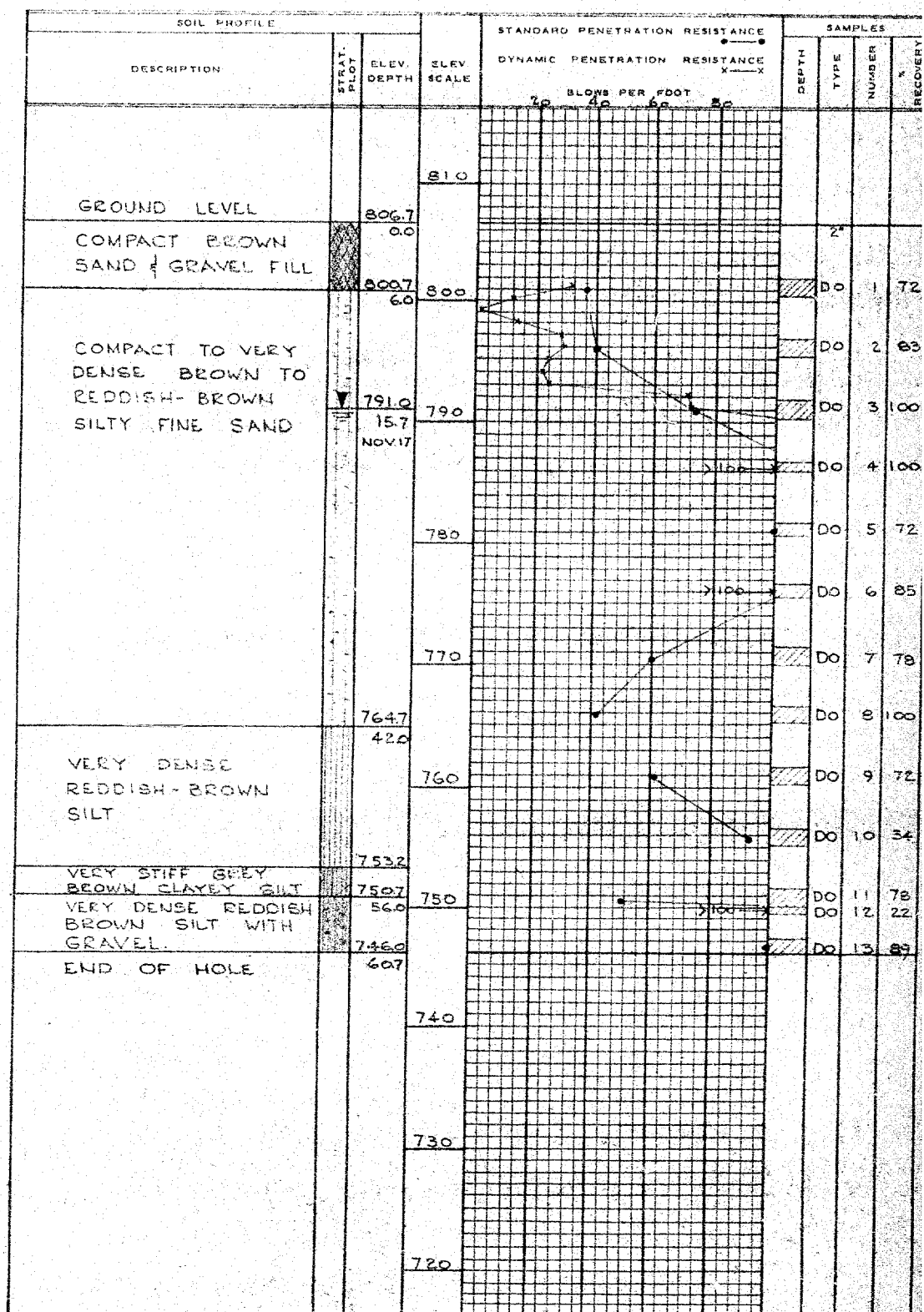
SAMPLE TYPES

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 963 BORING II BORING DATE NOV 16 & 17, 1966
 DATUM GLDPTIC DIAM. 4" AUGER & BX HAMMER 140 LBS. DROP 30 II



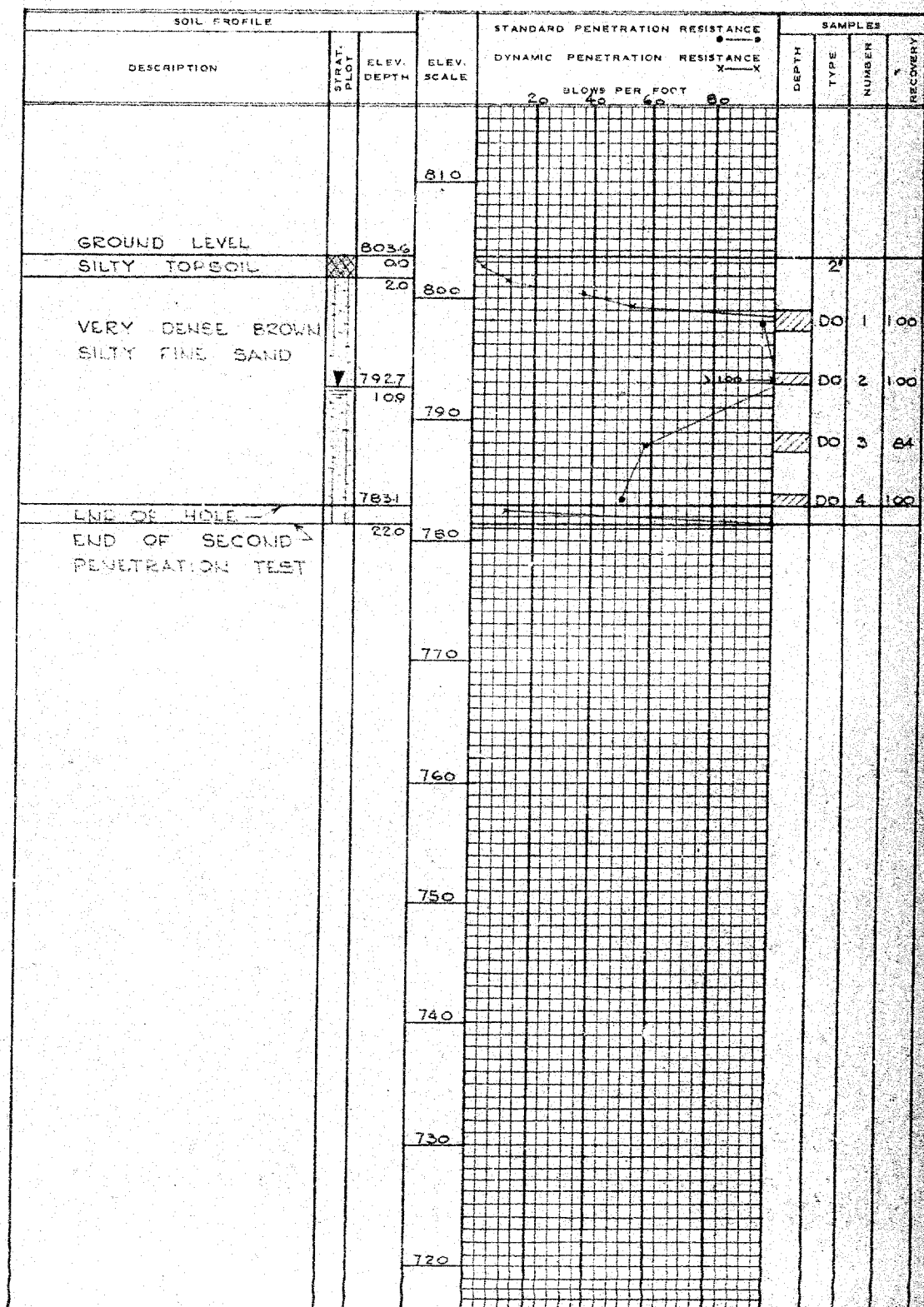
SAMPLE TYPES

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
 T GROUND WATER LEVEL
 AT TIME OF BORING

REMARKS

CONTRACT PC 963 BORING 12 BORING DATE NOV 18, 1960
 DATUM GEODETIC DIAM. 4" AUGER HAMMER 140 LBS. DROP 30 IN



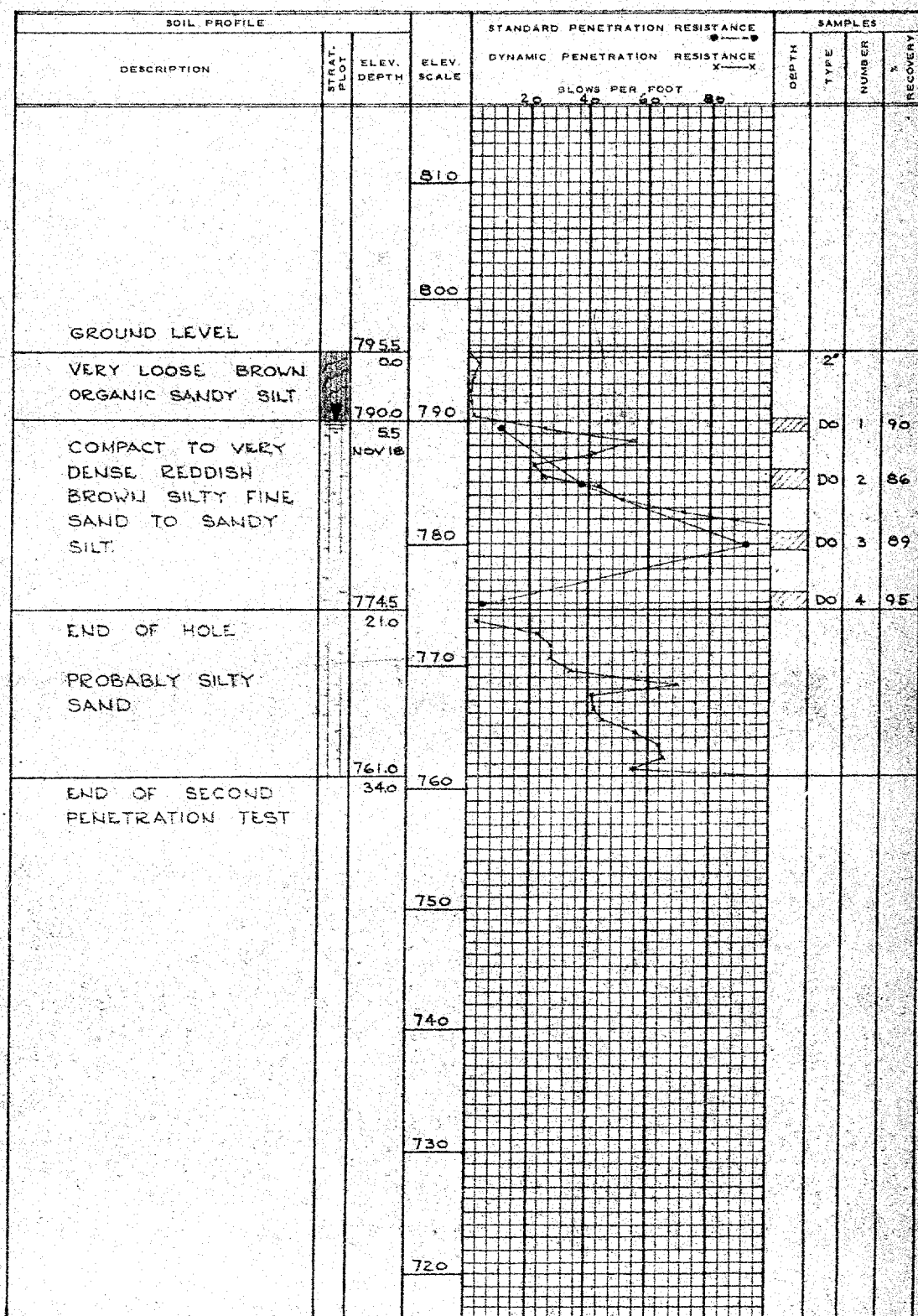
SAMPLE TYPES

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 963 BORING 13 BORING DATE NOV 18, 1960
 DATUM GEODETIC DIAM. 4" AUGER 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

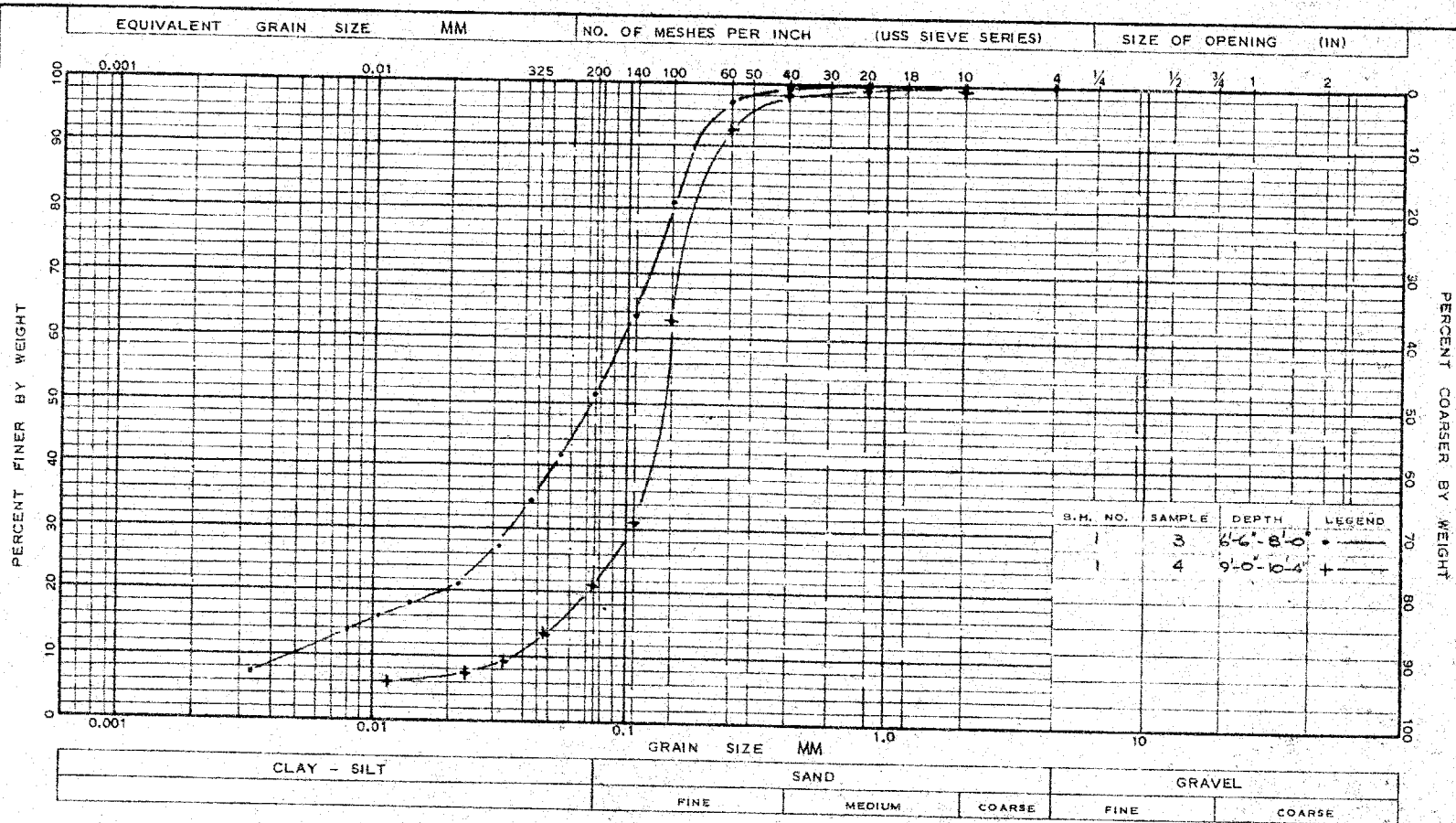
REMARKS

A P P E N D I X I I

FIGURES - - LABORATORY TESTING

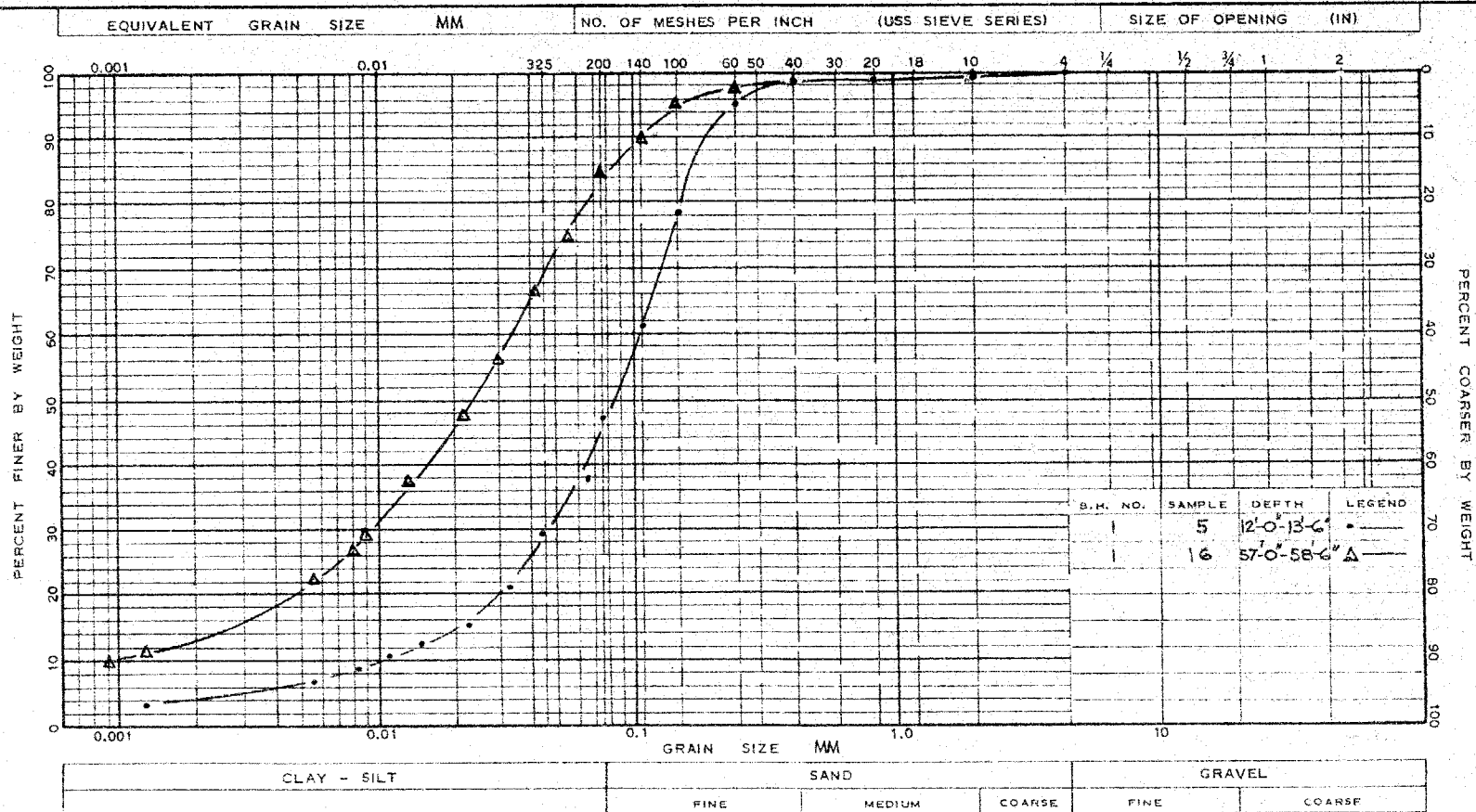
FRANKI OF CANADA
SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

P.C. NO. 963 LAB. NO. T-1
PROJECT _____
BOREHOLE NO. _____ SAMPLE NO. _____
TESTED BY _____ DATE NOV 7, 1960
CHECKED BY E.K. DATE NOV 23, 1960



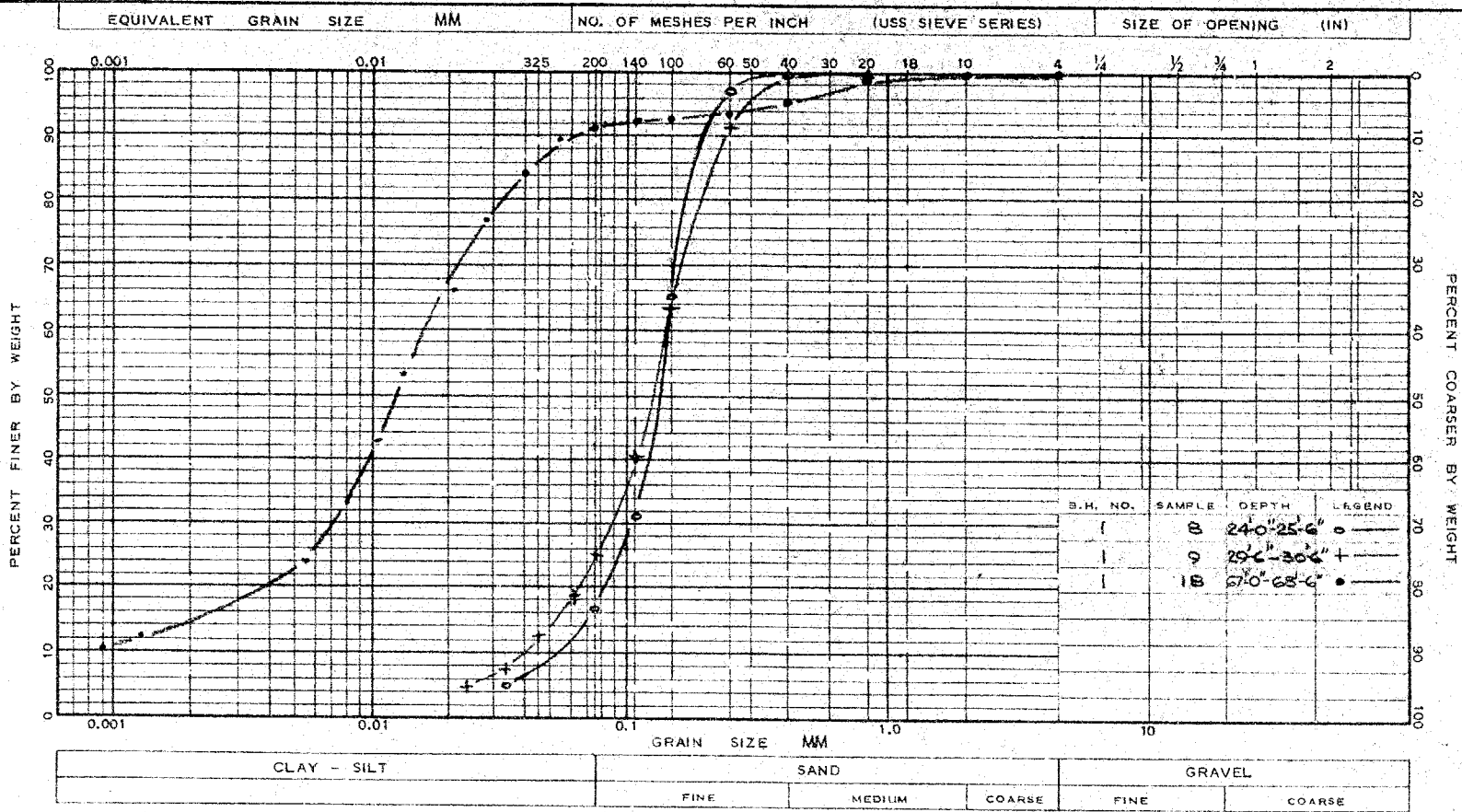
FRANKI OF CANADA
SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

F.C. NO. 963 LAB. NO. II-2
PROJECT _____
BOREHOLE NO. _____ SAMPLE NO. _____
TESTED BY _____ DATE NOV 10, 1960
CHECKED BY E.K. DATE NOV 23, 1960



FRANKI OF CANADA
SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

P.C. NO. 963 LAB. NO. II-3
PROJECT _____
BOREHOLE NO. _____ SAMPLE NO. _____
TESTED BY C.S. DATE NOV 16, 1968
CHECKED BY E.K. DATE MAY 23, 1969



FRANKI OF CANADA

SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

P.C. NO. 263 LAB. NO. II-4

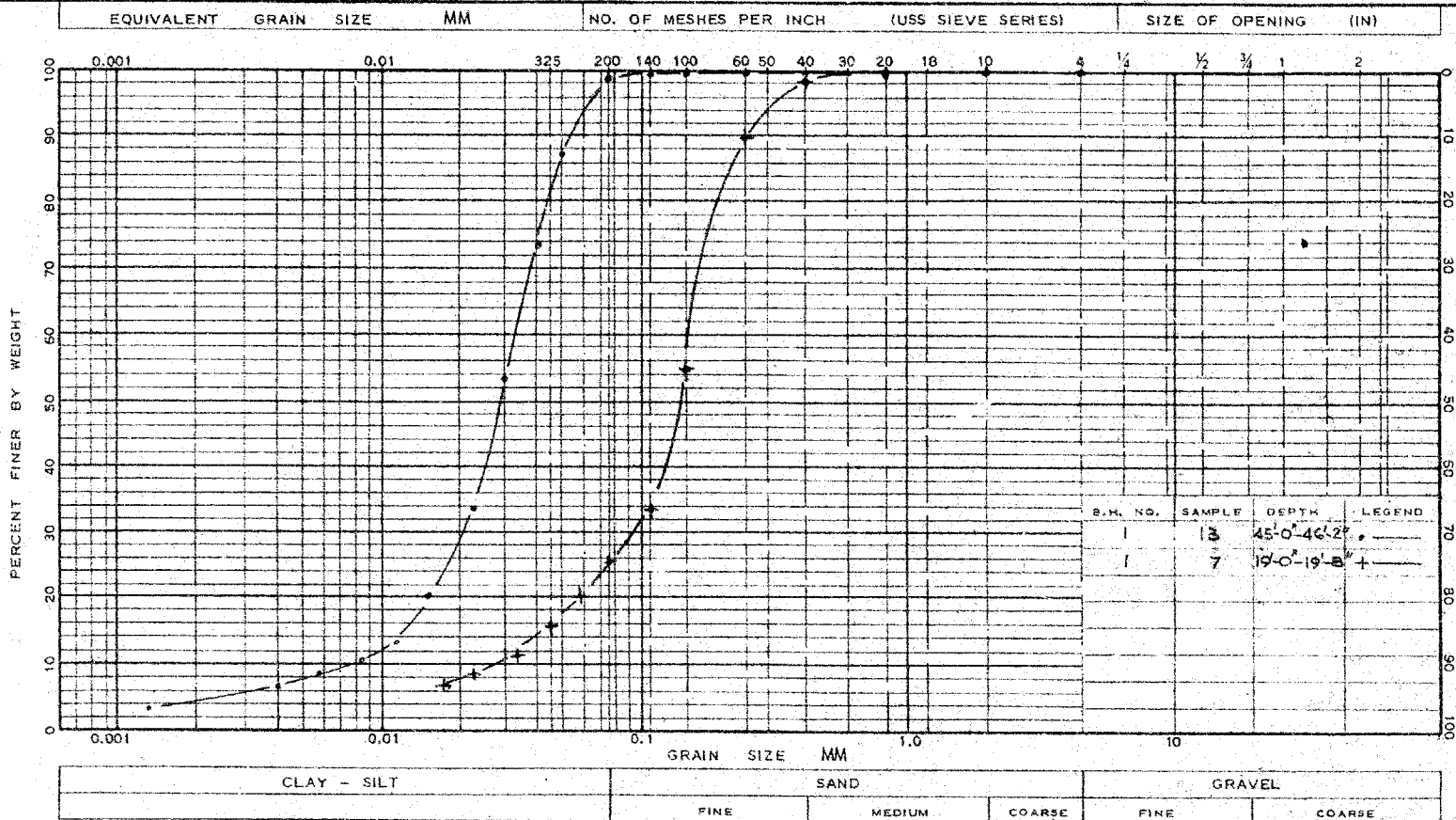
PROJECT _____

BOREHOLE NO. _____ SAMPLE NO. _____

TESTED BY _____ DATE NOV 10, 1960

CHECKED BY S.K. DATE MAY 29, 1963

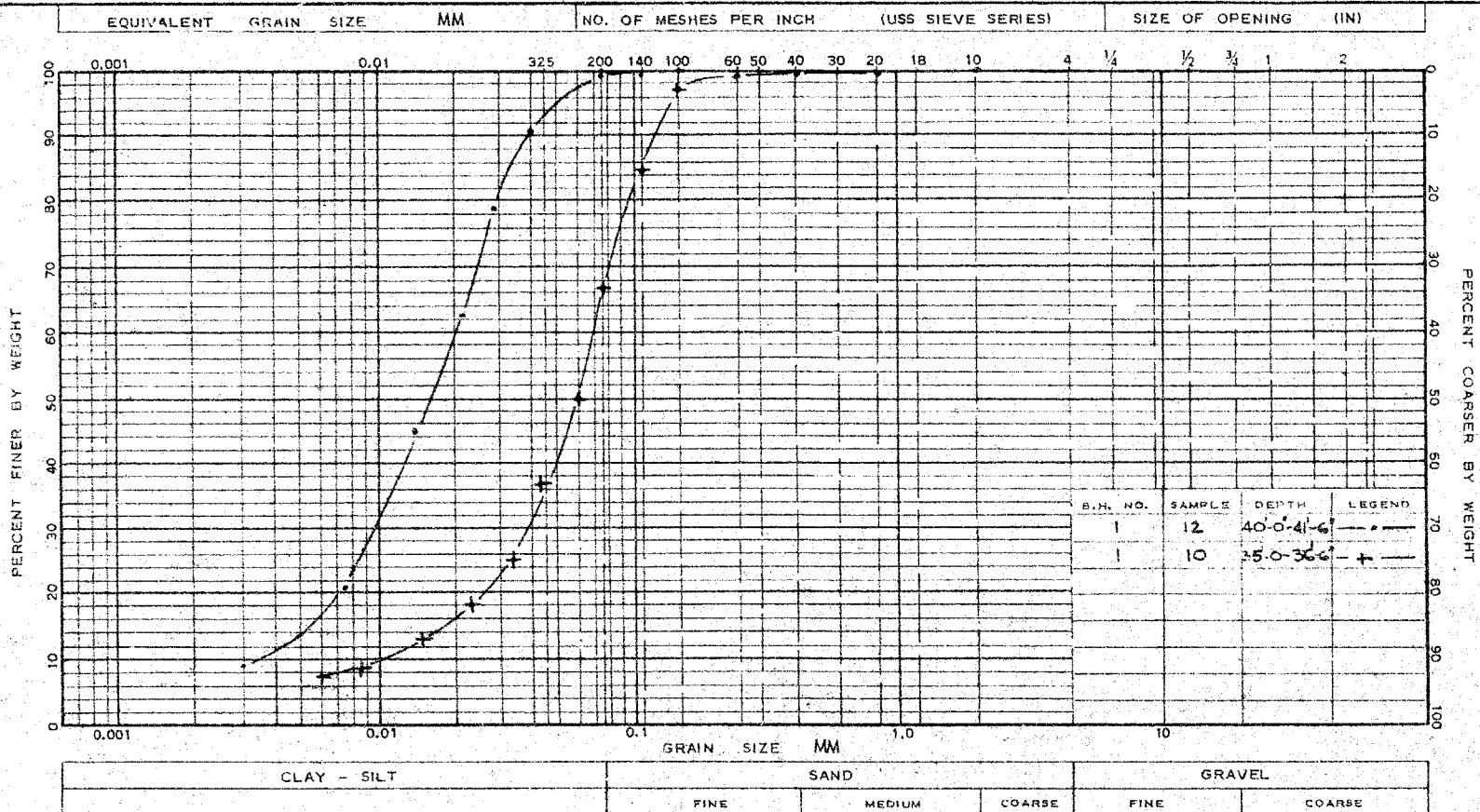
PERCENT COARSER BY WEIGHT



SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

SAMPLE NO

DATE NOV



FRANKI OF CANADA

SOIL INVESTIGATIONS MECHANICAL ANALYSIS

P.C. NO. 963 LAB. NO. II-6

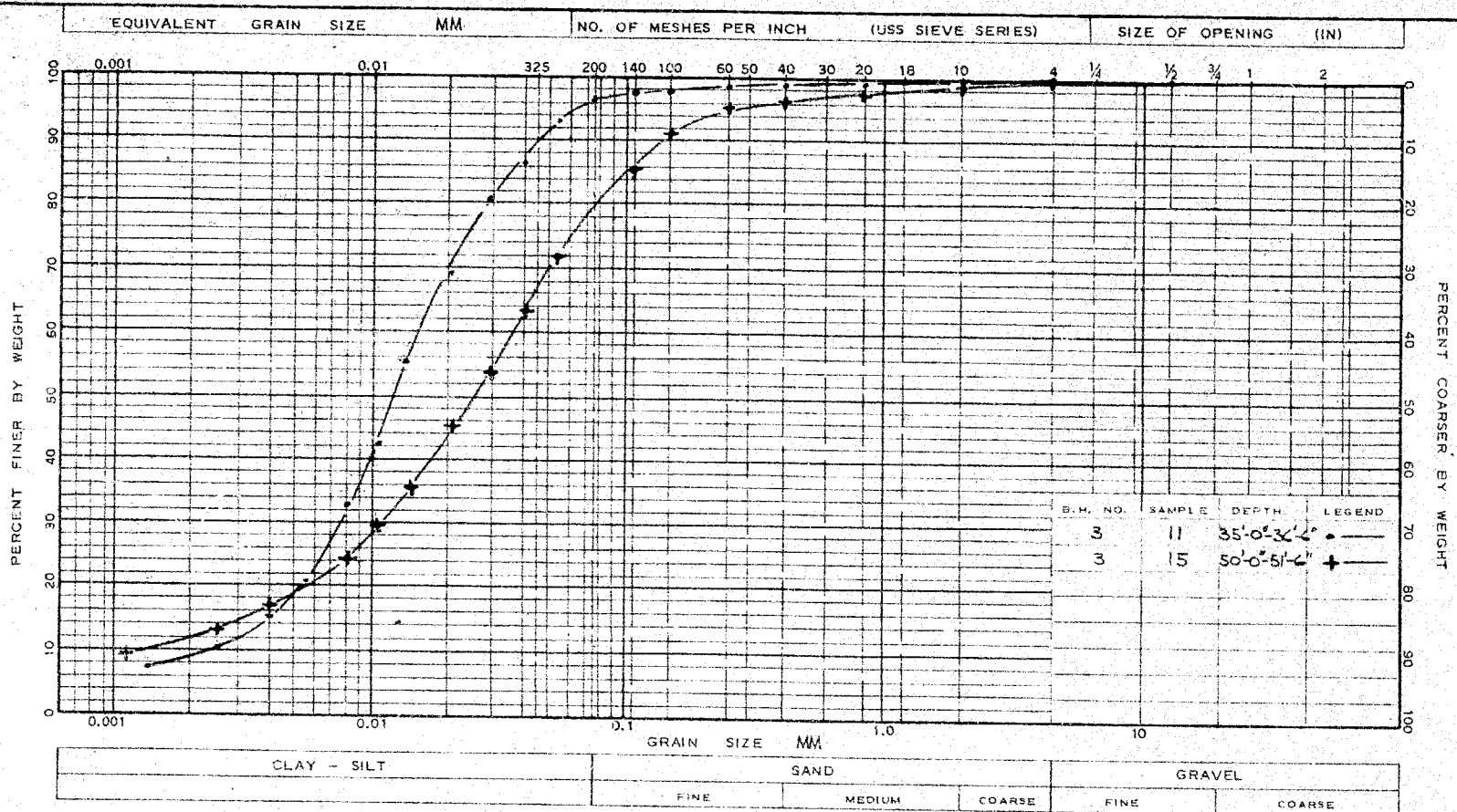
PROJECT

BOREHOLE NO.

SAMPLE NO.

TESTED BY W. C. J. DATE Nov 15, 1962

CHECKED BY E. K. DATE Nov 29, 1962



FRANKI OF CANADA

SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

P.C. NO. 963 LAB. NO. II-7

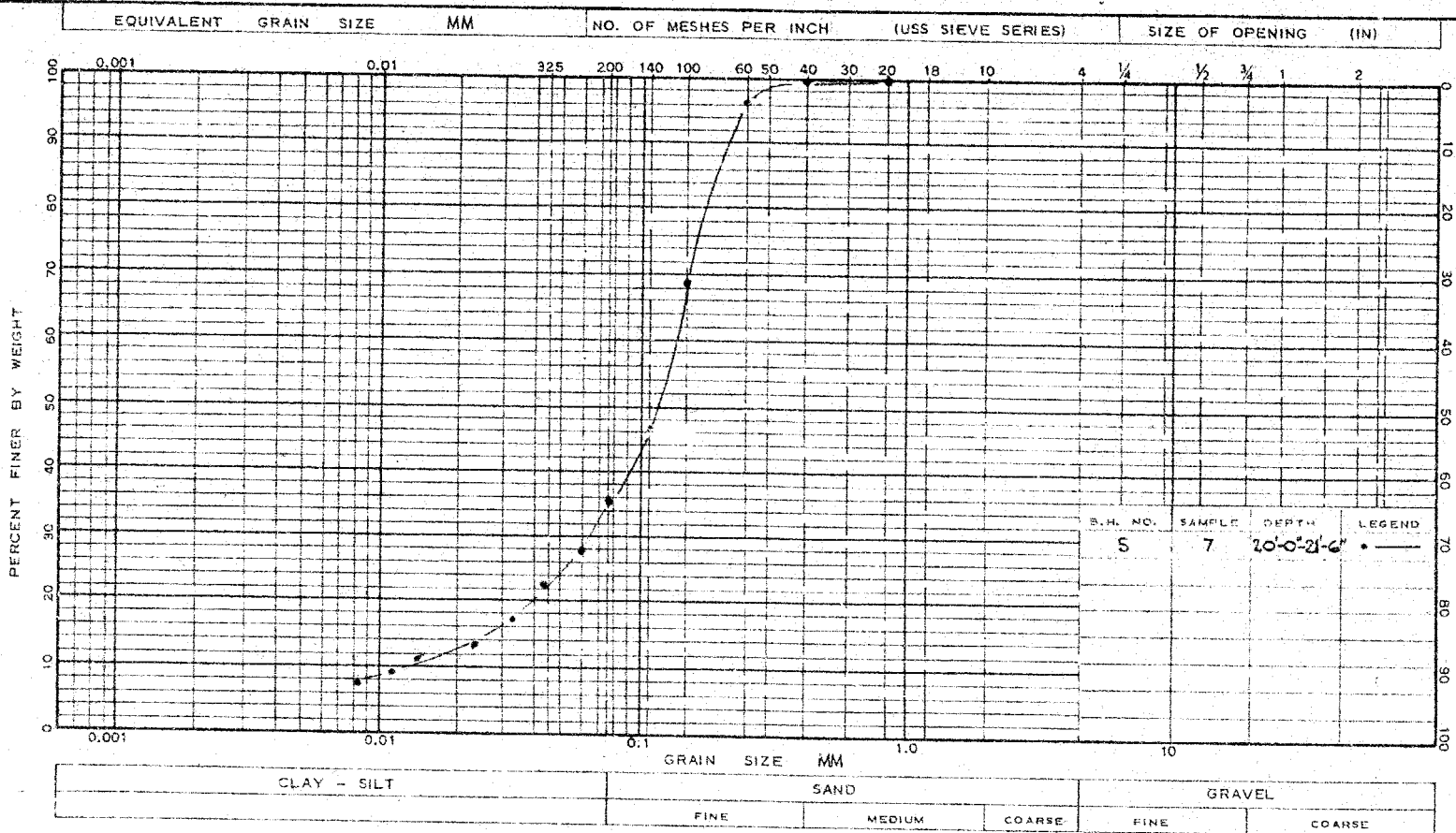
PROJECT _____

BOREHOLE NO. _____ SAMPLE NO. _____

TESTED BY C.S. DATE NOV 15, 1966

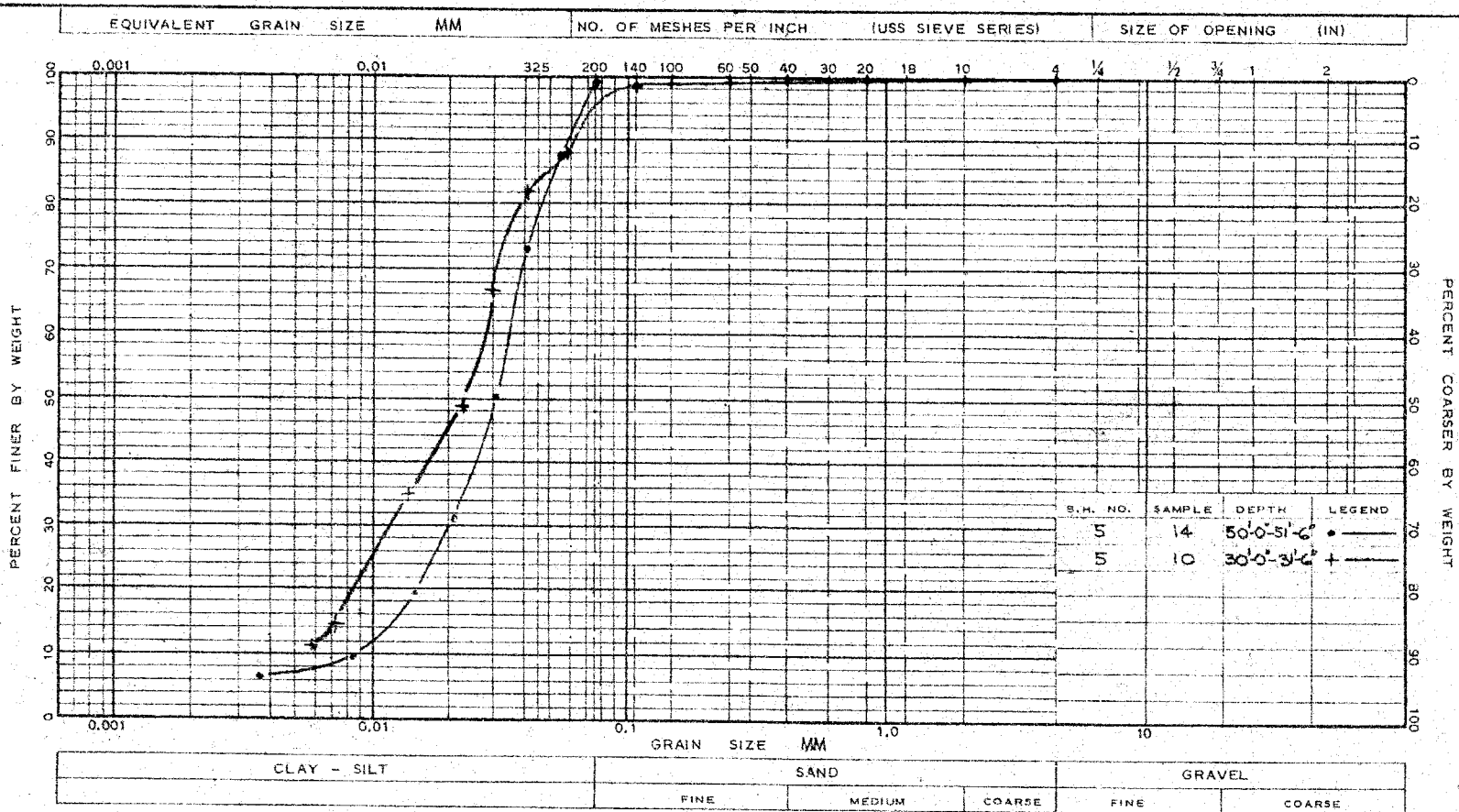
CHECKED BY E.K. DATE NOV 23, 1966

PERCENT COARSER BY WEIGHT



FRANKI OF CANADA
SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

P.C. NO. 963 LAB. NO. II-8
PROJECT _____
BOREHOLE NO. _____ SAMPLE NO. _____
TESTED BY C.S. DATE Nov 16, 1960
CHECKED BY E.K. DATE Nov 23, 1960



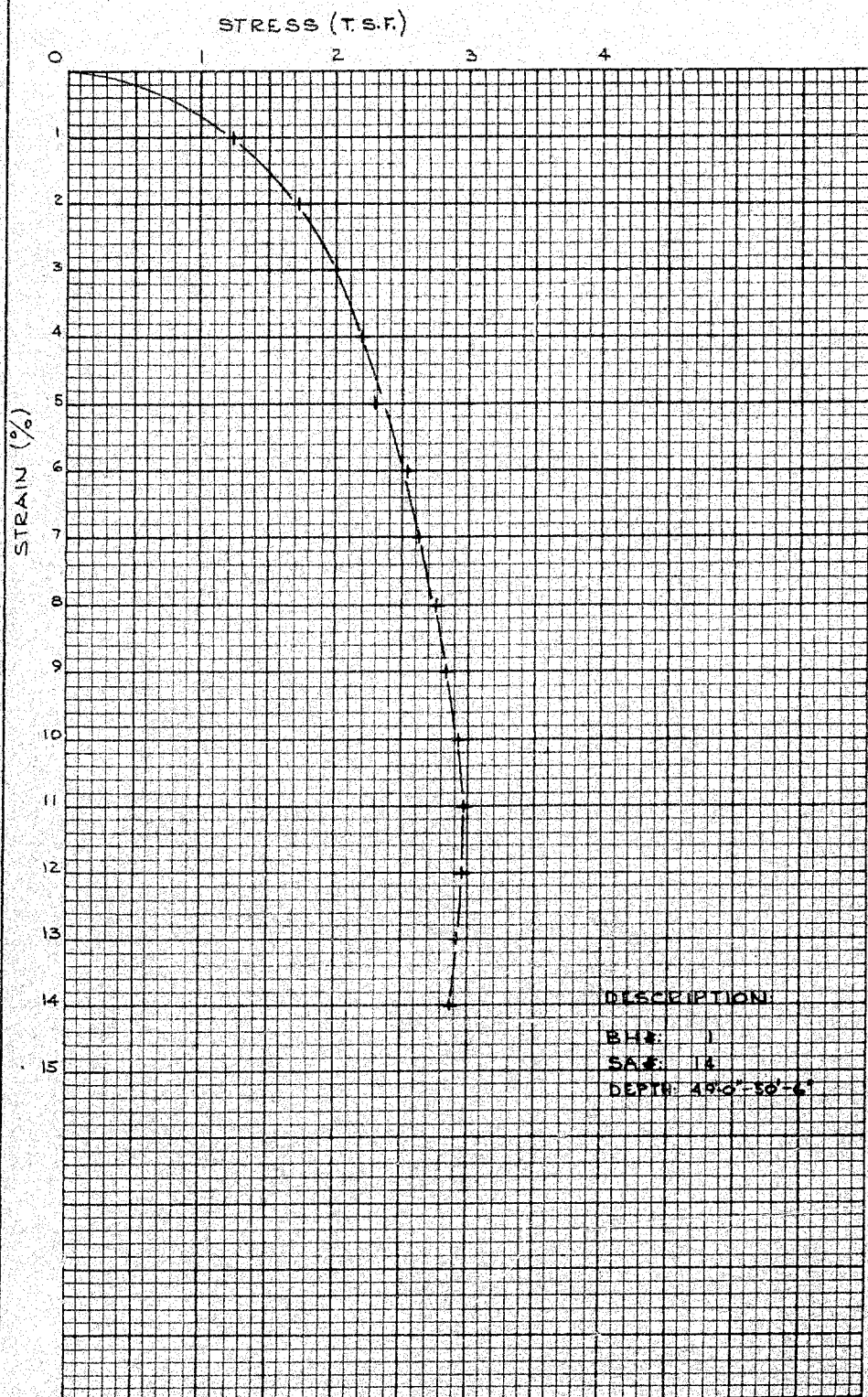
FRANKI OF CANADA LIMITED

UNDRAINED TRIAXIAL
COMPRESSION TEST

APPENDIX II

FIGURE 9

CONTRACT PC 963



A P P E N D I X III

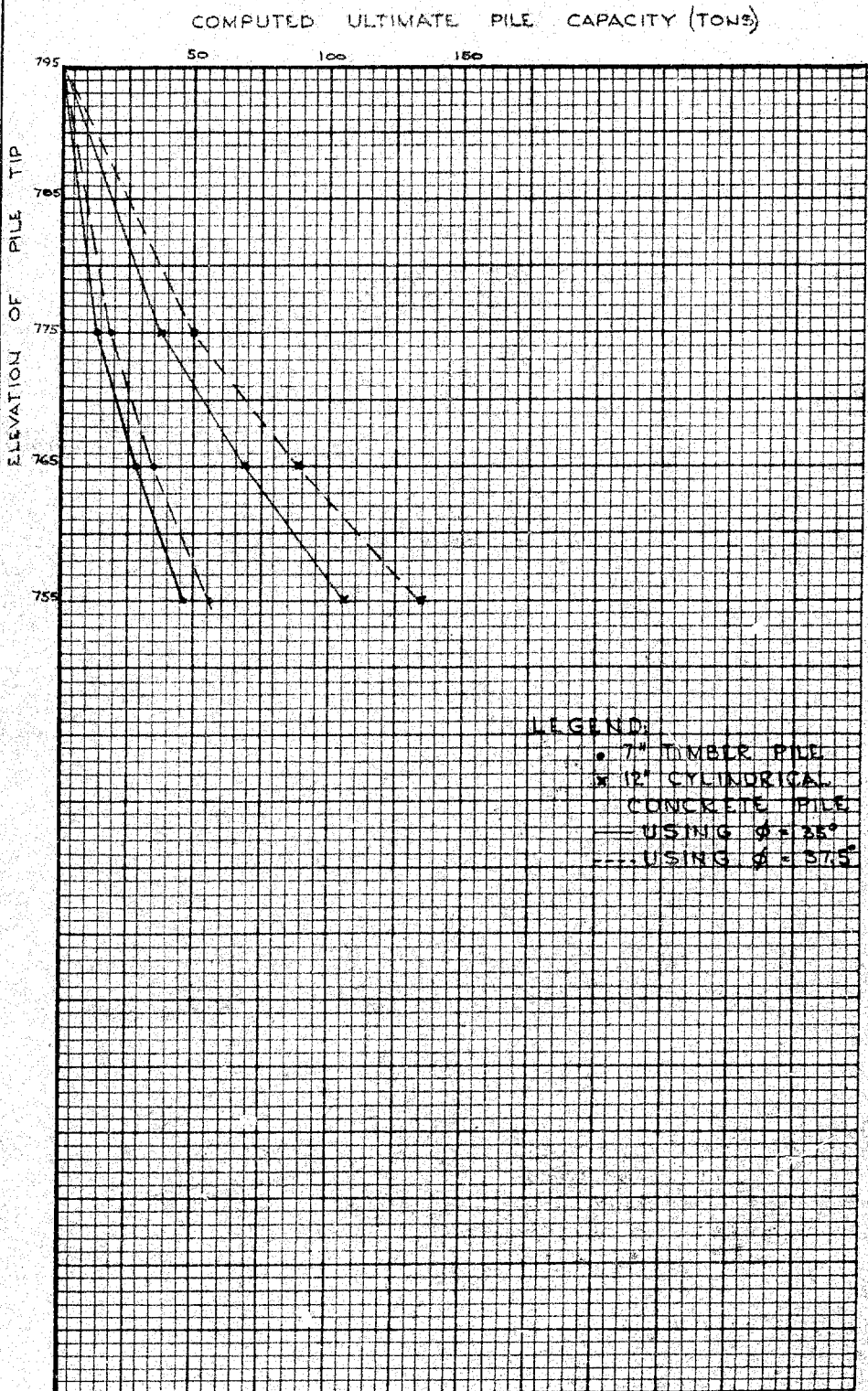
FIGURE - - PILE CAPACITY

FRANKI OF CANADA LIMITED
 ULTIMATE PILE CAPACITY
 VERSUS
 PILE TIP ELEVATION

APPENDIX II

FIGURE 1

CONTRACT PC 963



A P P E N D I X I V

PHOTOGRAPHS

APPENDIX IV

Looking North from Borehole 11. Person standing in centre-line of Highway 403. Warning signs indicate position of piers and abutments.



Looking East along centre-line of Highway 403.



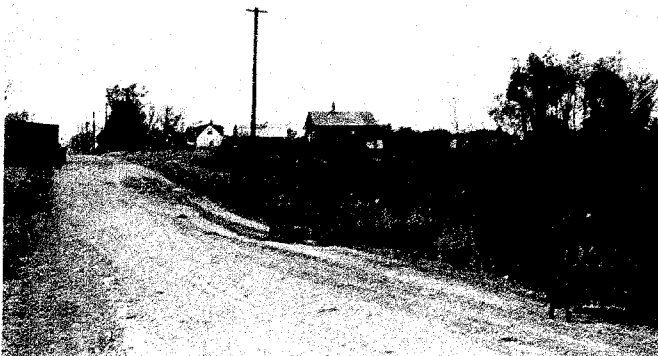
Looking North from borehole 5. Warning signs indicate position of piers and abutments.

APPENDIX IV

Looking North from Borehole 11. Person standing in centre-line of Highway 403. Warning signs indicate position of piers and abutments.



Looking East along centre-line of Highway 403.



Looking North from borehole 5. Warning signs indicate position of piers and abutments.

1880 *2.

Ground level: 705.

705 to 703: org. calc. fr.

main hole $\frac{1}{2}$ natural
depth.

Soil Profile			STANDARD PENETRATION RESISTANCE						N VALUE
DESCRIPTION	STAT. PLT.	ELEV. SURF.	ELEV. SCALE	BLOW/FEET					
				0	4	8	12	16	20
			810						
G.I.		810							
ROLL FILL			800						10
									18
									20
BASE OF FOOTING			790						2100
SILTY FINE SAND									2100
									2100
									3
			770						4
SILT WITH GREY									4
SILTY CLAY									21
PROCKETS									
			750						
SILTY CLAY									32
			740						
									60
SILT WITH GRAVEL			720						200

SOIL PROFILE				STANDARD PENETRATION RESISTANCE						N VALUE
DESCRIPTION	STAT. PLAT.	ELEV. DEPT.	ELEV. SCALE	20	40	60	80	100	120	B/C ₁
			810							
			800							
G.L.		781.5								
GRAVEL		781.5								
BASE OF PENETRATION		781.5	780							
SANDY SILT		781.5								
		781.5								
		781.5	760							
DENSE SILTY										
FINE SAND			770							
TO SILT										
			760							
		755.3	750							
END OF PENETRATION		755.3								
TEST			750							
			740							

END OF CORE PENETRATION

SOIL PROFILE			STANDARD PENETRATION RESISTANCE							N VALUE
DESCRIPTION	STAT. POINT	ELEV. DEPT.	ELEV. SCALE	20	40	60	80	100	120	S/ft
			810							
			800							
G. L.	7712									
BASE OF TESTING ORGANIC	7710		790							4
SANDY SILT										4
	7814									11
	7810		780							32
SILTY										22
FINE SAND										20
	7710		770							20
	7710									10
SILT			760							31
WITH CLAY										50
POCKETS	7532									
SILT - CLAY	7530									
	7530		750							>100
SILT WITH GRAVEL										>100
			740							76
END OF HOLE										

Soil Profile			N-Value					
Designation	Soil Profile	Soil Type	20	40	60	80	100	120
Gravel	Gravel	Gravel						
Sandy Gravel	Sandy Gravel	Sandy Gravel						
Silty Sand	Silty Sand	Silty Sand						
Base of Filling	Base of Filling	Base of Filling						
Penetration Test	Penetration Test	Penetration Test						

Soil Profile			STANDARD PENETRATION RESISTANCE						N Value
DESCRIPTION	STATION	DEPTH (ft)	20	40	60	80	100	120	B/S
		510							
		520							
G.L.									
ORGANIC SANDY SILT	755.0	750							10
Base of Footing	755.0								
Silty Fine Sand to									32
SANDY SILT	760	760							92
	766								6
End of Hole									
BITTY SAND (CLAYEY)		770							
END OF END PENETRATION TEST		760							
		750							
		740							

S. ALUMINUM.

EXCAVATE THE ORG. MAT. &
REPLACE IT WITH GRANULAR MATERIAL

AT 792: 90 Blows / ft.

SPREAD FOOTINGS

P.ED #3

from B.W. #5.

GROUND LEVEL 791.2
TO 791.0

Loose To comp.
org. sandy silt.

791 TO 764

30 TO 10
(blows) (blows)

Timber piles to Refusal
(at or around 760)

(The conditions are better
at B.W. #13.)

Pipe #1.

Ground level: 801.5

796 15 Blows/ft.

794 TO 782: 20 To 100+
Blows per ft.

Spread footings at 792.

~~790~~

G.L. 403.6.

EL.

R/A.

790 90

792 109

788 58

783 50

EXCAVATE ORG. MATERIAL.

SPREAD FOOTINGS

60-F-229

W.P. # 184-60

Hwy. # 403

HAMILTON DR.

UNDERPASS

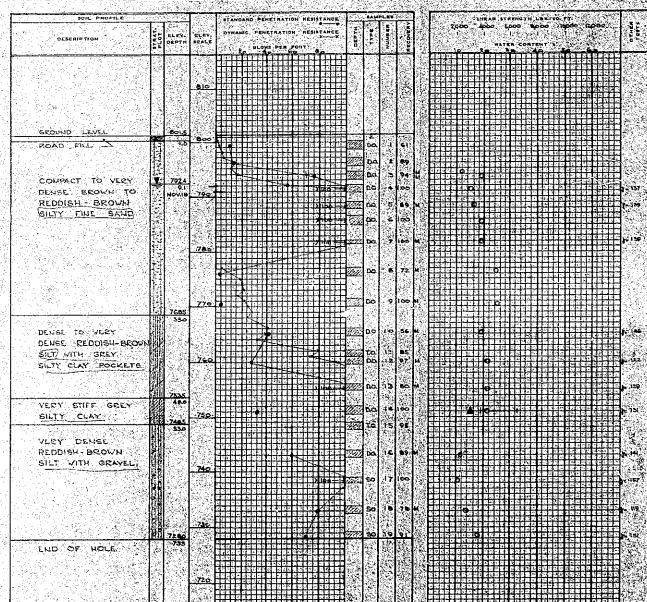
HAMILTON

FRANKI OF CANADA LTD.

BORING RECORD

 CONTRACT PC 963 BORING 1 BORING DATE OCT 19-21, 1960
 DATUM GEOIDETIC DIAM. 4" AUGER 1 BX HAMMER 140 LBS. DROP 30" IN

LABORATORY TESTS


 SAMPLE TYPES
 AS AUGER SAMPLE
 BS DRIVE OPEN
 CS SLEEVE OPEN VALVE
 DS SLEEVE FOOT VALVE
 ES TO THIN WALLED OPEN
 FS THIN WALLED PITCH
 GS WASHED SAMPLE

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

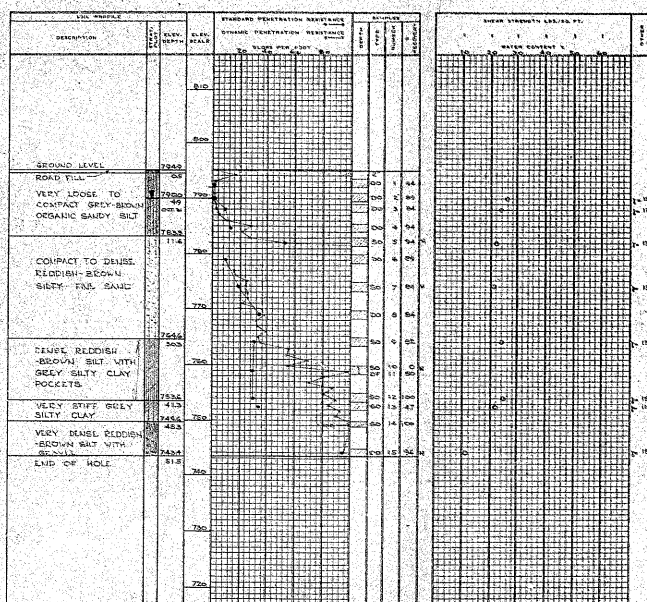
 D WATER CONTENT
 E ATMOSPHERIC LIMIT
 F LIQUID LIMIT
 G PLASTICITY INDEX
 H UNSATURATED WATER
 I UNSATURATED WATER
 J UNSATURATED WATER
 K UNSATURATED WATER
 L UNSATURATED WATER
 M UNSATURATED WATER
 N UNSATURATED WATER
 O UNSATURATED WATER
 P UNSATURATED WATER
 Q UNSATURATED WATER
 R UNSATURATED WATER
 S UNSATURATED WATER
 T UNSATURATED WATER
 U UNSATURATED WATER
 V UNSATURATED WATER
 W UNSATURATED WATER
 X UNSATURATED WATER
 Y UNSATURATED WATER
 Z UNSATURATED WATER

FRANKI OF CANADA LTD.

BORING RECORD

 CONTRACT PC 963 BORING 2 BORING DATE OCT 26-27, 1960
 DATUM GEOIDETIC DIAM. 4" AUGER 1 BX HAMMER 140 LBS. DROP 30" IN

LABORATORY TESTS


 SAMPLE TYPES
 AS AUGER SAMPLE
 BS DRIVE OPEN
 CS SLEEVE OPEN VALVE
 DS SLEEVE FOOT VALVE
 ES TO THIN WALLED OPEN
 FS THIN WALLED PITCH
 GS WASHED SAMPLE

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

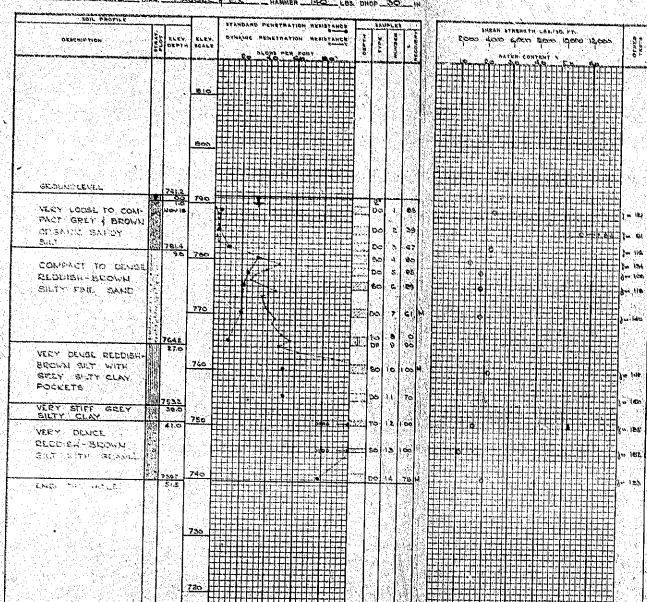
 D WATER CONTENT
 E ATMOSPHERIC LIMIT
 F LIQUID LIMIT
 G PLASTICITY INDEX
 H UNSATURATED WATER
 I UNSATURATED WATER
 J UNSATURATED WATER
 K UNSATURATED WATER
 L UNSATURATED WATER
 M UNSATURATED WATER
 N UNSATURATED WATER
 O UNSATURATED WATER
 P UNSATURATED WATER
 Q UNSATURATED WATER
 R UNSATURATED WATER
 S UNSATURATED WATER
 T UNSATURATED WATER
 U UNSATURATED WATER
 V UNSATURATED WATER
 W UNSATURATED WATER
 X UNSATURATED WATER
 Y UNSATURATED WATER
 Z UNSATURATED WATER

FRANKI OF CANADA LTD.

BORING RECORD

 CONTRACT PC 963 BORING 3 BORING DATE OCT 27-27, 1960
 DATUM GEOIDETIC DIAM. 4" AUGER 1 BX HAMMER 140 LBS. DROP 30" IN

LABORATORY TESTS


 SAMPLE TYPES
 AS AUGER SAMPLE
 BS DRIVE OPEN
 CS SLEEVE OPEN VALVE
 DS SLEEVE FOOT VALVE
 ES TO THIN WALLED OPEN
 FS THIN WALLED PITCH
 GS WASHED SAMPLE

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

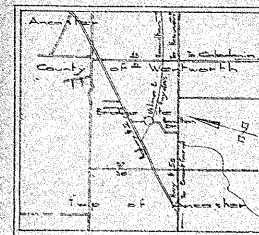
 D WATER CONTENT
 E ATMOSPHERIC LIMIT
 F LIQUID LIMIT
 G PLASTICITY INDEX
 H UNSATURATED WATER
 I UNSATURATED WATER
 J UNSATURATED WATER
 K UNSATURATED WATER
 L UNSATURATED WATER
 M UNSATURATED WATER
 N UNSATURATED WATER
 O UNSATURATED WATER
 P UNSATURATED WATER
 Q UNSATURATED WATER
 R UNSATURATED WATER
 S UNSATURATED WATER
 T UNSATURATED WATER
 U UNSATURATED WATER
 V UNSATURATED WATER
 W UNSATURATED WATER
 X UNSATURATED WATER
 Y UNSATURATED WATER
 Z UNSATURATED WATER

Con 3 Lot 30

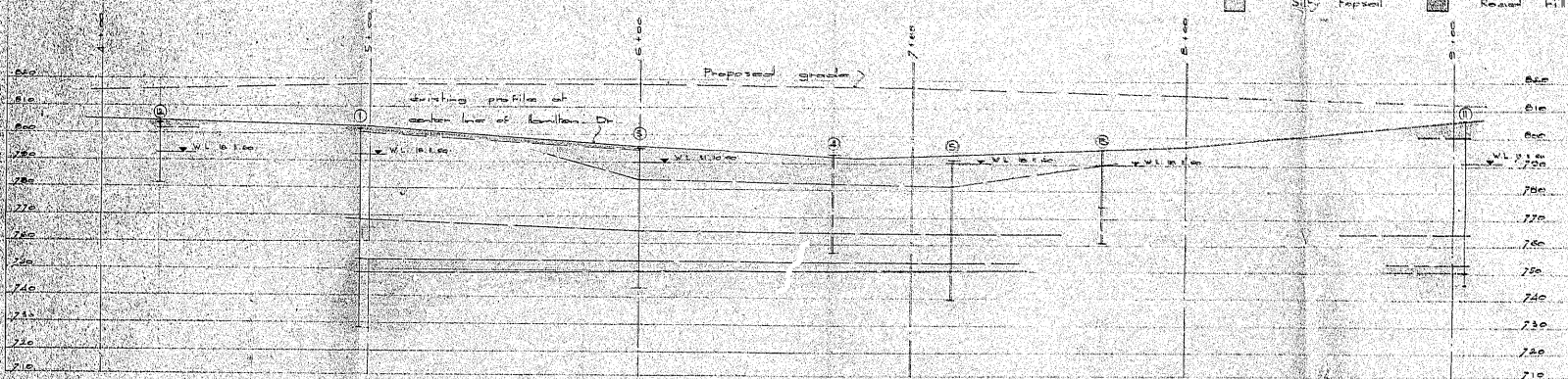
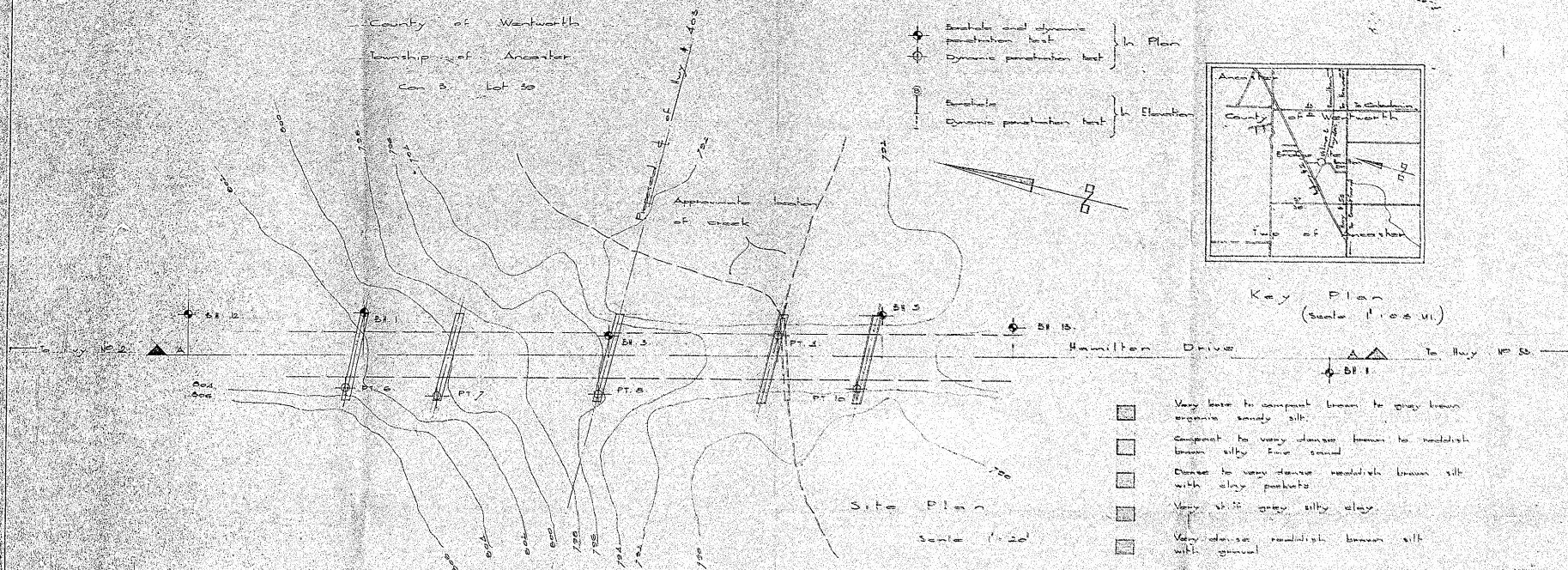
- Baseline and dynamic penetration test
- Dynamic penetration test

} In Plan

Barocholite } in Elevation
Dynamic penetration test }




Key Plan
(see 1108 vi.)



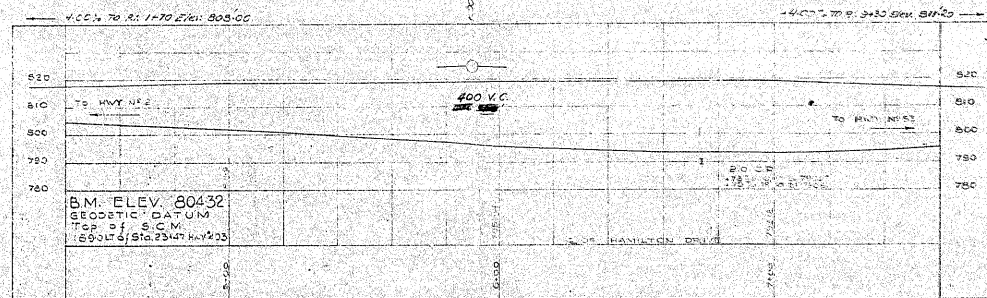
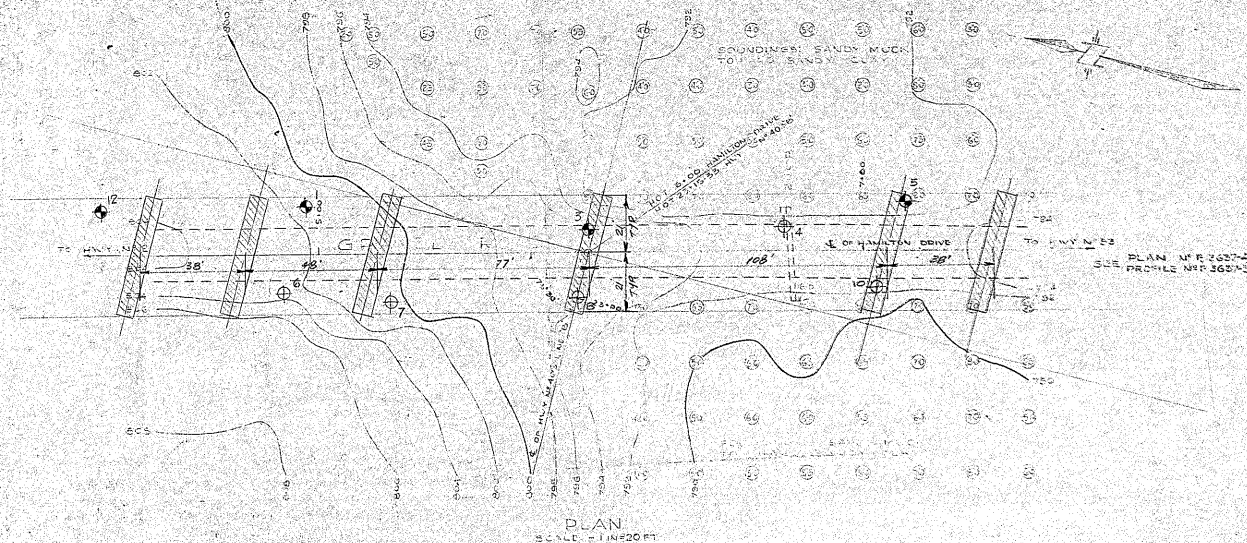
Three assumptions are built upon to tell a more "heavy"
 picture... is half shared... only at... bachelor
 position is... Between... bachelors... the
 boundaries... have... been... interpreted

Cross Section A-A

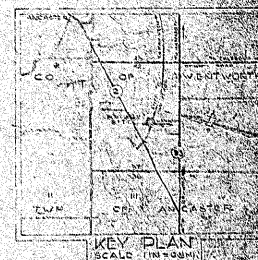
Scaler	10×20	Vert
	10×20	Horiz

JOB NO.	PER. ORDER NO.	Proposed Bridges int
DATE	21-1-66	Hampton Drive and
SCALE	1/4" = 1'	Kings Hwy NB 403.
DWG. BY	Checked by	
INTD BY		
APPROVED		
 FRANK J. O'CONNELL PROJECT SUPERVISOR		FRANKI OF CANADA LTD 214 MERTON ST TORONTO

COUNTY of VENTWORTH
TOWNSHIP of ANCASTER
CON III LOT 39



PROFILE
SCALE 1\"/>



N.P.N. 184-60 FRANK-4

DATE	REMARKS

DEPARTMENT OF HIGHWAYS - ONTARIO
PLANNING & DESIGN BRANCH

DISTRICT NO. 4

PROPOSED CROSSING

AT
HAMILTON DRIVE

AND
THE KINGS HWY N400 LINE

LOT 39

TOWNSHIP OF ANCASTER, COUNTY OF VENTWORTH

BRIDGE SITE 12

SURVEY BY CHIEF OF DISTRICT - H. H. HARRISON	APPROVED BY CHIEF OF DISTRICT - H. H. HARRISON
SUPERVISOR - J. E. COYNE	SCALE - AS SHOWN
DRAWN BY - J. E. COYNE	DATE OF DESIGN - 1960
CHECKED BY - J. E. COYNE	DATE OF PLAN - 1960
APPROVED BY - J. E. COYNE	PLANNING - E. J. COYNE
SUPERVISOR - J. E. COYNE	

SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS

60-F-229

W.P. # 184-60

Hwy. # 403

HAMILTON DR.

UNDERPASS

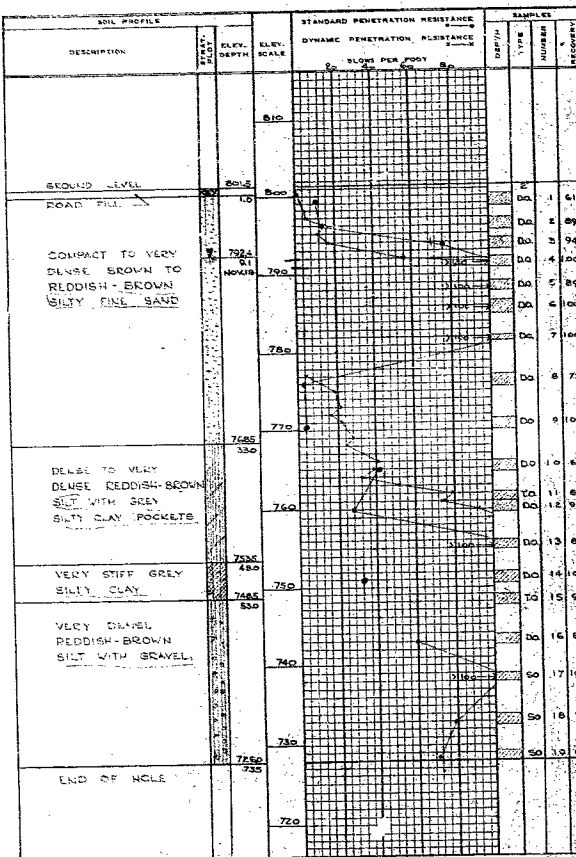
HAMILTON

FRANKI OF CANADA LTD.

BORING RECORD

CONTRACT PC 963 BORING 1 BORING DATE OCT 19-21, 1960
DATUM GEODETIC DIAM. 4" AUGER 1 BX HAMMER 140 LBS. DROP 30 IN

LABORATORY TESTS



SAMPLE TYPES
AS AUGER SAMPLE
DO DRIVE OPEN
DP 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_v FIELD PERMEABILITY TEST
↓ GROUND WATER LEVEL
AT TIME OF BORING
REMARKS

WATER CONTENT
A ATTENDING LIMITS
B IN SITU UNIT WEIGHT
C MECHANICAL ANALYSIS
K_p PERMEABILITY FIELD
K_v PERMEABILITY LAB.
R_s RELATIVE DENSITY
S_p SPECIFIC GRAVITY
P COMPACTION

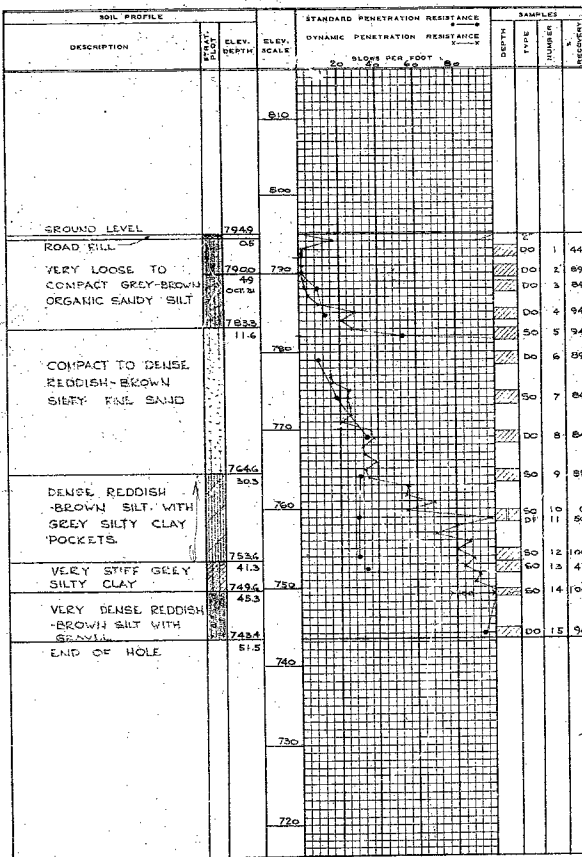
UNCONFINED
A UNDRAINED TRIAXIAL
B IN SITU VANE
C_u CONSOLIDATED UNDRAINED
C_u CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS
C_d CONSOLIDATED DRAINED
C CONSOLIDATION

FRANKI OF CANADA LTD.

BORING RECORD

CONTRACT PC 963 BORING 3 BORING DATE OCT 28-31, 1960
DATUM GEODETIC DIAM. 4" AUGER 1 BX HAMMER 140 LBS. DROP 30 IN

LABORATORY TESTS



SAMPLE TYPES
AS AUGER SAMPLE
DO DRIVE OPEN
DP 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_v FIELD PERMEABILITY TEST
↓ GROUND WATER LEVEL
AT TIME OF BORING
REMARKS

WATER CONTENT
A ATTENDING LIMITS
B IN SITU UNIT WEIGHT
C MECHANICAL ANALYSIS
K_p PERMEABILITY FIELD
K_v PERMEABILITY LAB.
R_s RELATIVE DENSITY
S_p SPECIFIC GRAVITY
P COMPACTION

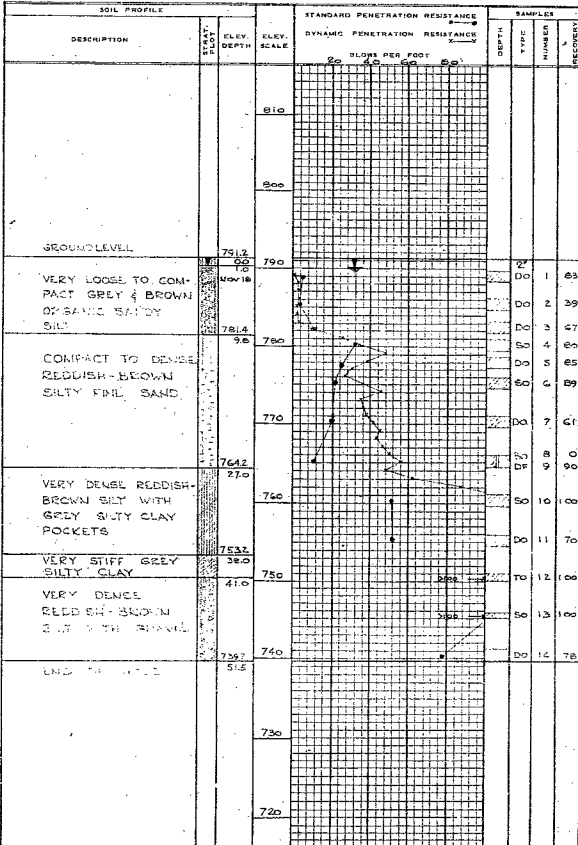
UNCONFINED
A UNDRAINED TRIAXIAL
B IN SITU VANE
C_u CONSOLIDATED UNDRAINED
C_u CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS
C_d CONSOLIDATED DRAINED
C CONSOLIDATION

FRANKI OF CANADA LTD.

BORING RECORD

CONTRACT PC 963 BORING 5 BORING DATE OCT 25-27, 1960
DATUM GEODETIC DIAM. 4" AUGER 1 BX HAMMER 140 LBS. DROP 30 IN

LABORATORY TESTS



SAMPLE TYPES
AS AUGER SAMPLE
DO DRIVE OPEN
DP 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_v FIELD PERMEABILITY TEST
↓ GROUND WATER LEVEL
AT TIME OF BORING
REMARKS

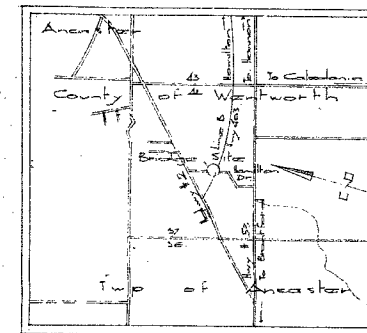
WATER CONTENT
A ATTENDING LIMITS
B IN SITU UNIT WEIGHT
C MECHANICAL ANALYSIS
K_p PERMEABILITY FIELD
K_v PERMEABILITY LAB.
R_s RELATIVE DENSITY
S_p SPECIFIC GRAVITY
P COMPACTION

UNCONFINED
A UNDRAINED TRIAXIAL
B IN SITU VANE
C_u CONSOLIDATED UNDRAINED
C_u CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS
C_d CONSOLIDATED DRAINED
C CONSOLIDATION

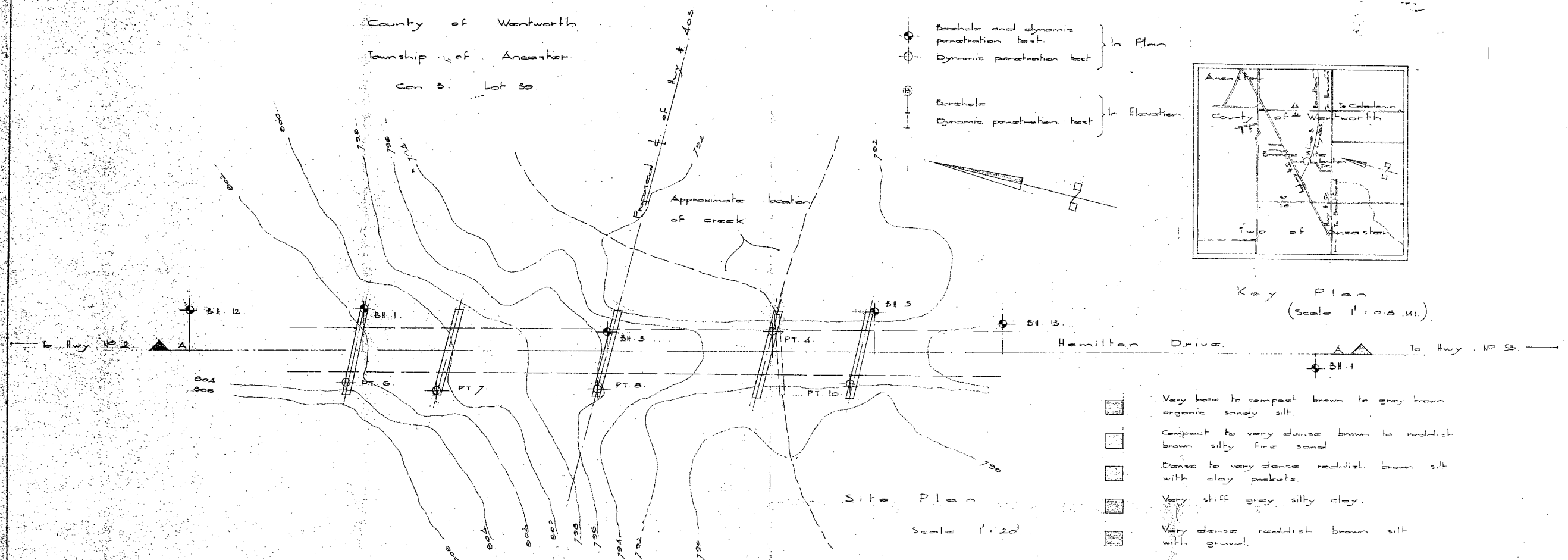
County of Westworth
Township of Ancaster
Con. B. Lot 30

Legend

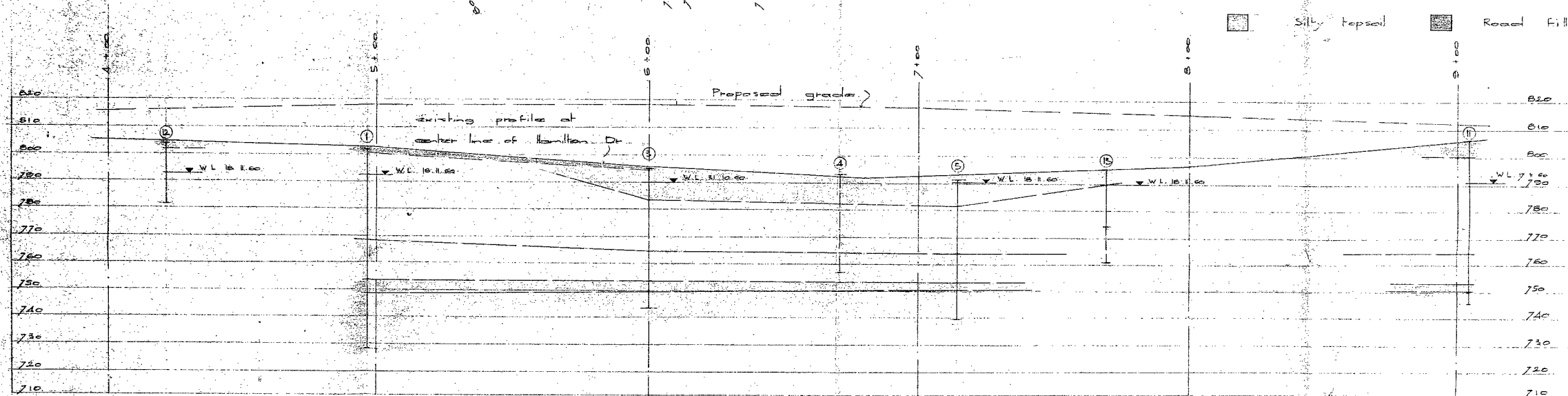
- Borehole and dynamic penetration test } In Plan
- ⊙ Dynamic penetration test } In Plan
- ⊙ Borehole } In Elevation
- ⊙ Dynamic penetration test } In Elevation



Key Plan
(Scale 1" = 0.5 MI.)



- Very loose to compact brown to grey brown organic sandy silt.
- Compact to very dense brown to reddish brown silty fine sand.
- Dense to very dense reddish brown silt with clay pockets.
- Very stiff grey silty clay.
- Very dense reddish brown silt with gravel.
- Silty topsoil
- Road Fill



Note

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries have been interpolated.

Cross Section A-A

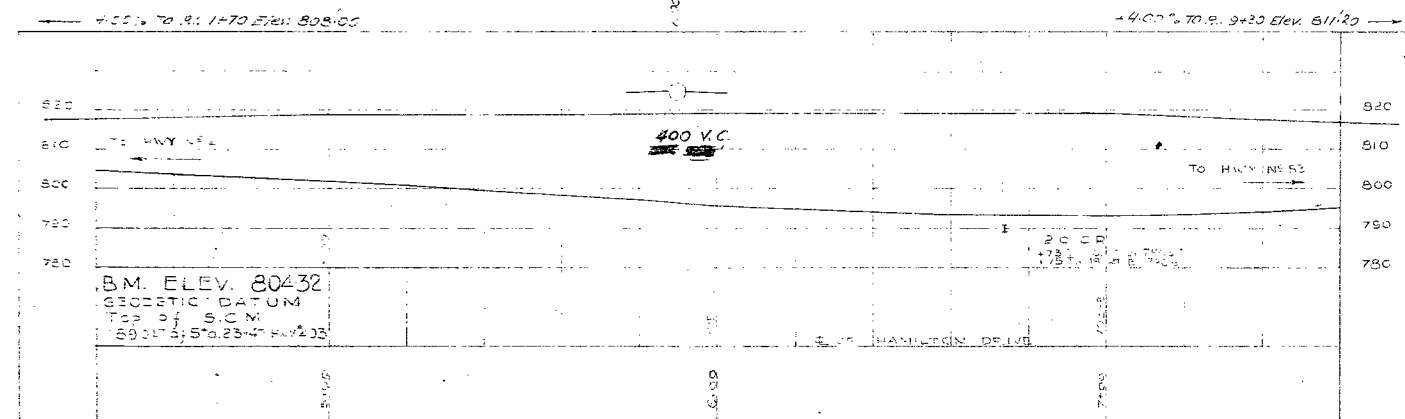
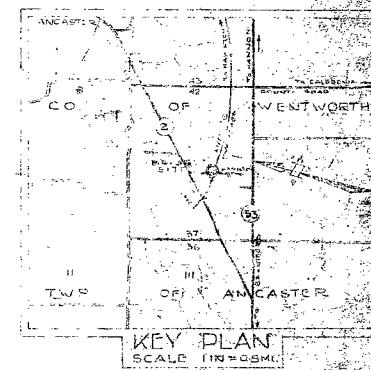
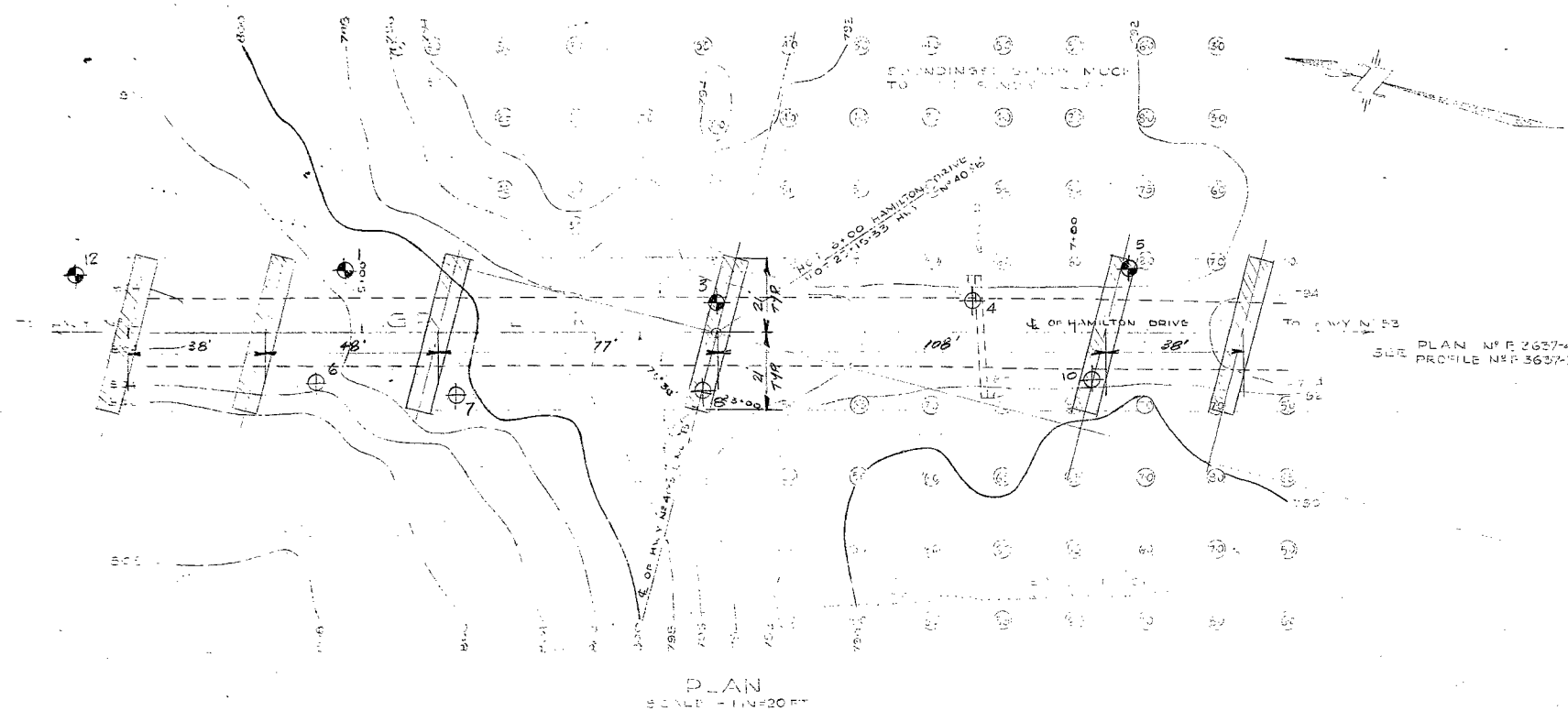
Scale { 1" = 20' Vert
1" = 20' Horiz

JOB NO.	PC 308-1	Proposed Bridge at
DATE	25.1.60	Hamilton Drive and
SCALE	Horizontal	Kings Hwy. No. 103
DWG. BY	W. J. J. J.	
CHECK BY	W. J. J. J.	
APPROVED		
FRANKI OF CANADA LTD.		
214 MERTON ST. TORONTO		

E-3913-1

E-3913-1

COUNTY OF WENTWORTH
TOWNSHIP OF ANCASTER
CON. III LOT 39

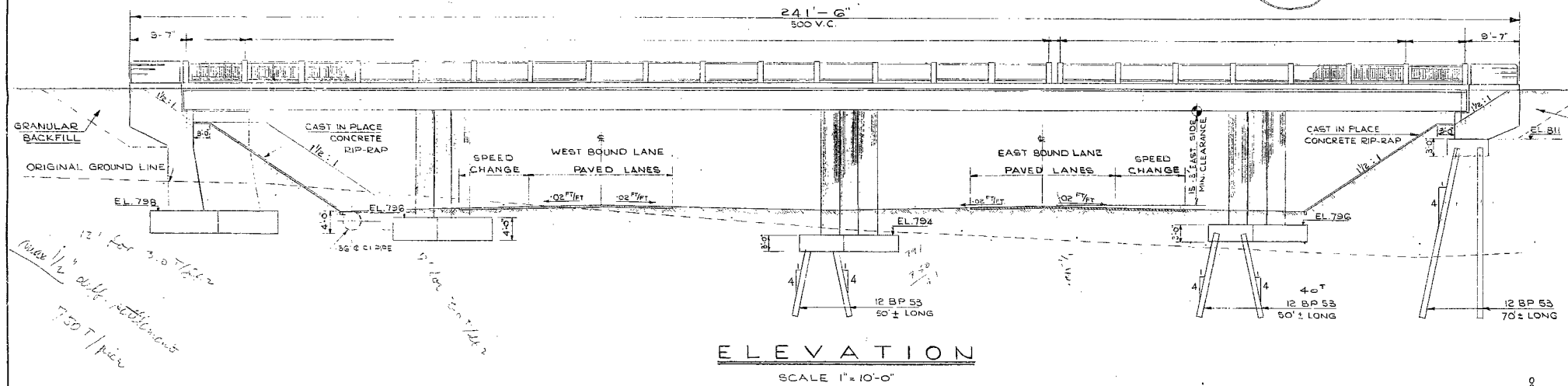
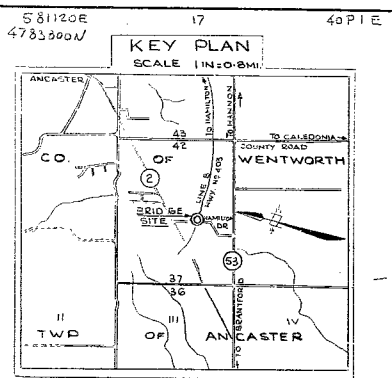
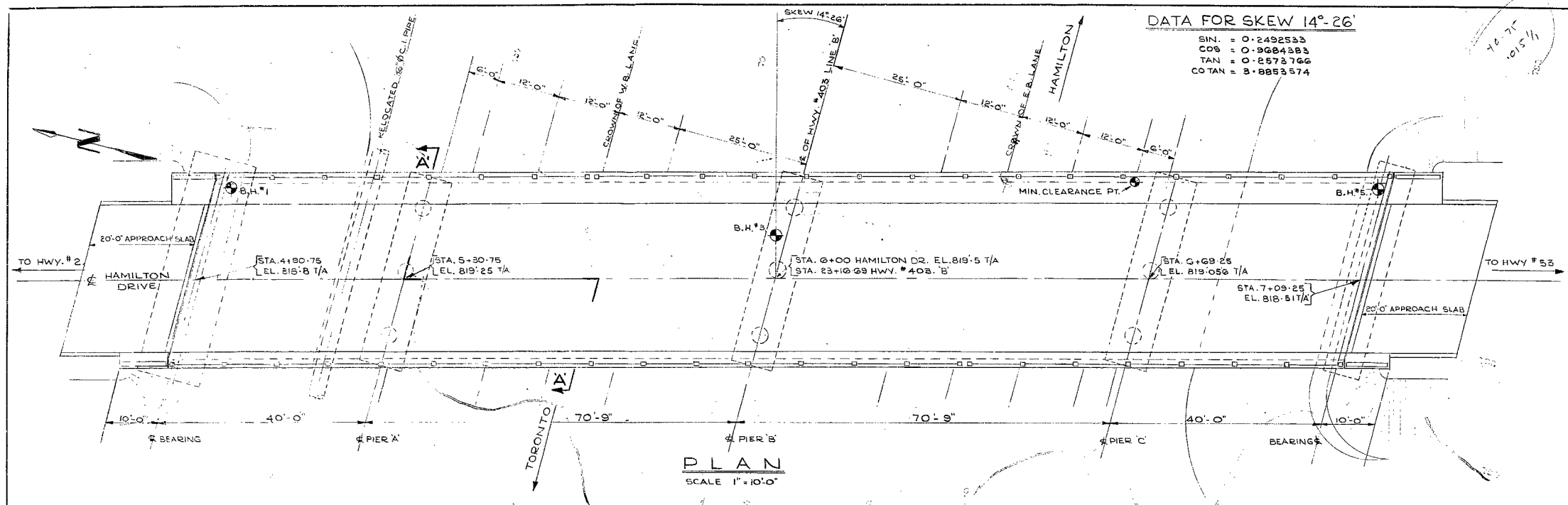


PLAN 184-60 FRANKI 66

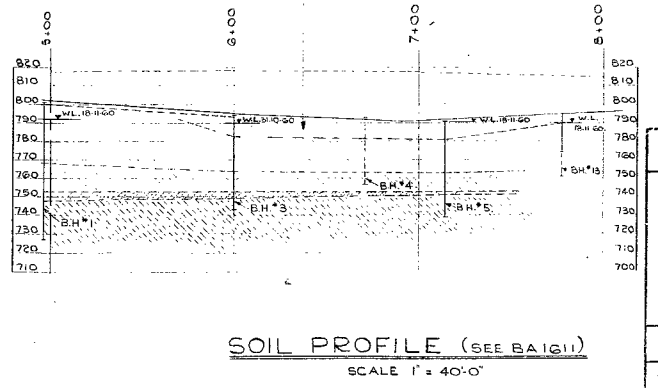
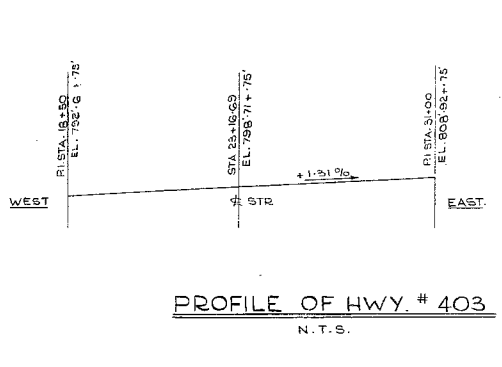
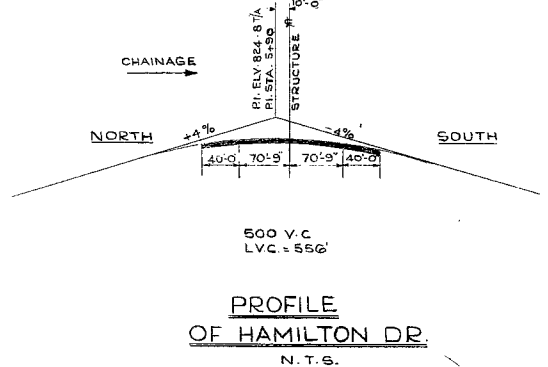
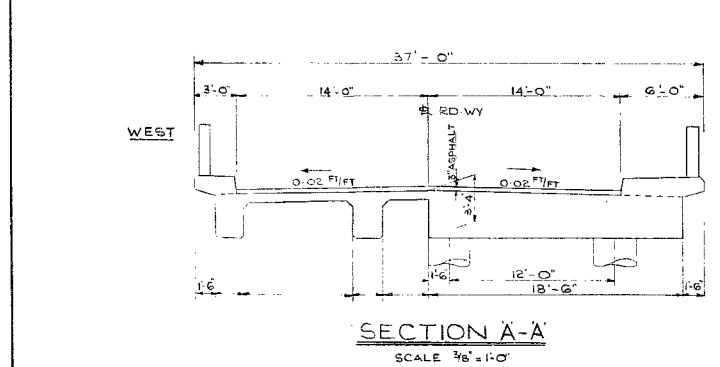
DATE	REMARKS	BY

DEPARTMENT OF HIGHWAYS - ONTARIO
PLANNING & DESIGN BRANCH
DISTRICT NO. 4
PROPOSED CROSSING
AT
HAMILTON DRIVE
AND
THE KINGS HWY. NO. 403 LINE B
LOT 39 CON. III
TOWNSHIP OF ANCASTER COUNTY OF WENTWORTH
BRIDGE SITE
SURVEY BY: A. BUCHANAN
SUPERVISOR: D. SCOTT
CHIEF OF PARTY: A. BUCHANAN
DIRECTOR: D. SCOTT
SCALE: AS SHOWN
DATE OF SURVEY: 1966
DATE OF PLAN: 1966
APPROVED BY: A. BUCHANAN
SUPERVISOR: D. SCOTT
PLAN NO. E-3913-1

SOME DEFECTS IN NEGATIVE DUE
TO CONDITION OF ORIGINAL DOCUMENTS



- LEGEND**
- VERY LOOSE COMPACT BROWN TO GREY BROWN ORGANIC SANDY SILT.
 - COMPACT TO VERY DENSE BROWN TO REDDISH BROWN SILTY FINE SAND.
 - DENSE TO VERY DENSE REDDISH BROWN SILT WITH CLAY POCKETS.
 - VERY STIFF GREY SILTY CLAY.
 - VERY DENSE REDDISH BROWN SILT WITH GRAVEL.



W.P. 184-60

DEPARTMENT OF HIGHWAYS-ONTARIO
 BRIDGE OFFICE-TORONTO

ANCASTER TWP UNDERPASS

THE KING'S HIGHWAY No. 403 DIST. No. 4
 CO. WENTWORTH
 TWP. ANCASTER LOT 39 CON. III

PRELIMINARY PLAN

APPROVED

BRIDGE ENGINEER DESIGN ENGINEER

DATE Jan. 1961

