

FRANKI OF CANADA, LIMITED

SOIL INVESTIGATIONS

214 MERTON ST. TORONTO
HU. 1-642-7

WP 221-61

R E P O R T

to

DEPARTMENT OF HIGHWAYS, ONTARIO

on

SOIL CONDITIONS

PROPOSED MIMICO CREEK BRIDGE

DIXON ROAD, TORONTO, ONTARIO

Distribution :

14 copies : Department of Highways,
Ontario

2 copies : Franki of Canada Limited

Our Reference

PC 1056
OP 20161

Your Reference
WP 221-61

27th December 1961

I N D E XSOIL REPORT

Introduction	Page 1
Procedure	" 1
Soil Conditions	" 1
Water Conditions	" 4
Discussion	" 4
Conclusions and Recommendations	" 9

BORING RECORDS

DRAWING PC 1056-1 : Borehole locations and
Inferred Soil Stratigraphy

FIGURES : Results of Laboratory Testing

INTRODUCTION

Franki of Canada Limited has been retained by the Department of Highways, Ontario, by letter of November 15th 1961, to carry out a soil investigation at the site of a proposed new bridge where Dixon Road crosses Mimico Creek, in Toronto, Ontario.

The object of the investigation was to determine and interpret the soil conditions at the site as they affect the foundation design of the proposed bridge.

PROCEDURE

The field work was commenced on November 28th, 1961, and completed on November 30th 1961, using two machines, and consisted of two detailed boreholes with adjacent dynamic penetration tests.

The locations of the boreholes are shown on Drawing PG 1056-1 which also shows a section of the inferred soil stratigraphy. A detailed log for each hole is given on the Boring Records.

Soil testing on samples obtained was carried out in our laboratory and the results are given on the Boring Records and on the Figures. Samples remaining after testing will be stored until July 1st, 1962, and then discarded unless other instructions are received.

Elevations referred to in this report are Geodetic and are related to a D.H.O. benchmark at the location shown on the Drawing. The elevation of this benchmark was given as 505.00.

SOIL CONDITIONS

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

2.

Fill

Fill was encountered to a depth of 16 and 15 feet in boreholes 1 and 2 respectively. The fill consists of clayey till which is generally very organic and contains pockets of pure organic material.

Standard penetration or 'N' values obtained in the fill ranged from 5 to 19 blows per foot. The average consistency of the fill is soft to firm, as estimated from visual examination. The natural moisture content obtained ranged from 7 to 20 percent with a median value of 15 percent.

Topsoil

About 1 foot of black silty topsoil was encountered below the fill in borehole 1.

Clayey to Silty Very Fine Sand

Beneath the topsoil in borehole 1 is a layer of brown clayey to silty very fine sand, about 2 feet in thickness. A standard penetration or 'N' value of 14 blows per foot was obtained for the sand and a moisture content of 18 percent.

Compact to Very Dense Grey Silty to Sandy Till

Beneath the sand in borehole 1 and below the fill in borehole 2 is a stratum of grey till extending to about elevation 455. The till is composed of a heterogeneous mixture of sand and gravel in a matrix which is generally silty near the surface of the stratum, becoming coarser with depth. The gravel content also increases with depth. Typical grainsize distribution curves showing this trend of becoming coarser with depth, are shown on Figure 1.

In borehole 1 the 'N' values obtained were all in excess of 80 blows per foot and with the exception of one, in excess of 120 blows per foot, indicating the very high density of the till. In borehole 2, similar results were obtained

3.

below elevation 480. Above elevation 480 in borehole 2, the till is of similar composition but of lesser density as indicated by the 'N' values which averaged 30 blows per foot.

The natural moisture content of the upper part of the till is about 13 percent with an in-situ unit weight ranging from 139 to 145 pounds per cubic foot. The till in both boreholes below elevation 480 has a natural moisture content of the order of 7 to 8 percent and an average un-situ unit weight greater than 145 pounds per cubic foot.

Grey Silt and Varved Silty Clay

At about elevation 471 in borehole 1 a layer of grey stratified silt was encountered, about 2.5 feet in thickness. An 'N' value of 34 blows per foot was obtained in the silt layer, indicating its dense nature.

At about elevation 478 in borehole 2 is a 2.5 foot thick layer of complex composition. The upper part of the layer consists of a very heterogeneous mixture of silty clay, sand and gravel; this is followed by laminated silt, clayey silt and sandy silt with thin sand partings; the remainder of the layer resembles a varved silty clay. Individual laminations in the above material are from 1/8 inch to 1 inch in thickness.

From visual examination, the shear strength of the more cohesive lower part of the layer in borehole 2 is between 1000 and 1500 pounds per square foot. An average natural moisture content of 20 percent was obtained.

The above materials are interglacial lacustrine deposits of probably different periods. These deposits usually occur in lenses and are also in this case considered to be non-continuous.

Bedrock

At about elevation 455, the till rests on grey shale bedrock. The shale is considerably weathered as indicated by the comparatively low resistance to augering in borehole 1.

4.

The rock was core drilled in borehole 2 for a depth of 5 feet and a core recovery of 45 percent was obtained confirming the weathered nature of the shale.

WATER CONDITIONS

The boreholes were advanced dry. Ground water in borehole 1 was encountered during sampling at elevation 581.5 where it remained throughout the boring operations. This level coincides with the water level of Mimico Creek at the bridge location.

Borehole 2 remained dry during the sampling operations and after completion a water level observation pipe was inserted in the borehole for later observation. Between December 1st and 13th 1961, the ground water level fluctuated between elevations 483.4 and 585.2 or in average 2.8 feet higher than the level of the creek.

DISCUSSION

The bridge was originally planned to be a rigid frame and the boreholes were put down on either side of the creek with this design in mind. After completion of the field work it became known that partly because of hydraulic considerations the design had changed and it is now understood that the structure will consist of three simply supported spans, 40 feet long.

It is further understood that the northern half of the structure will be built first, while traffic will be maintained on the existing structure. After completion of the northern half, traffic will be detoured onto it; the existing bridge will then be demolished and the remainder of the proposed structure built.

It is known that in the present design, the piers

5.

will carry about 18 kips per foot run. The abutments would be loaded to about one half this value.

It has been proposed to found both piers and abutments on piles. It is understood that this type of foundation under the abutments has been chosen because of settlement considerations. The main consideration for piled foundations under the piers is understood to be to safeguard against scour.

It is considered that spread footings should not be excluded as a possible type of foundation. In the following paragraphs therefore, both types of foundation are discussed :

(a) Piled Foundations

Piles should be carried into the very dense silty till stratum below elevation 480. In order to provide adequate protection against scour, piles should penetrate sufficiently far into the till stratum below the creek bottom to a minimum depth depending partly on the depth of scour which may be expected. A minimum depth of penetration below the creek bottom of 15 feet has been proposed. The tips of the piles below the piers would therefore have to be at about elevation 465.

It is considered that no prefabricated pile would penetrate the till stratum to this depth. The only type of pile that would penetrate some distance would be a steel H pile. Maximum penetration would be determined to some extent by the pile section and the driving energy employed, but it is considered that the maximum penetration would be to an elevation between 470 and 475. In addition, the pile would be subject to damage, which could not be ascertained nor controlled. It is therefore recommended that a type of bored pile be used. The most feasible and economical method would probably be to pre-bore a hole by auger, possibly lined in the upper few feet and back-filled with concrete. Alternatively, a pipe may be churn drilled to the required depth and concreted while the pipe is being withdrawn. The ultimate

6.

bearing capacity of a 12 inch diameter pile may be conservatively taken as 50 tons and a factor of safety of 2 would be adequate for design, giving an allowable load per pile of 25 tons. The total load per pier is about 900 tons and about 36 piles at 3 foot spacing in a single row would be required. Similar treatment may be given to the abutment foundations. For larger diameter piles, the capacity may be computed using the ratio of the diameters squared. Larger capacity will decrease the number of piles and increase their spacing. Too great a spacing would require heavier reinforcement in the walls. The choice will therefore depend on economic considerations.

The foregoing is based on the assumptions that settlement under the abutments and scour in the river will pose difficulties. In connection with the latter it should be stated that this assumption was made without the knowledge of the soil conditions.

If it is assumed that the till stratum encountered in boreholes 1 and 2 extends beneath the river bottom, then the possibility of scour to the degree assumed is decreased. Moreover if scour took place to such a depth, the stability of the embankments would be seriously endangered and at failure the sliding soil mass would in all probability shear off or at least damage the piled foundations. It is understood that the maximum recorded high water level is at elevation 496 and occurred during Hurricane Hazel. At that time some undermining of the existing structure which is believed to sit on footings presumably took place. The present free width between abutments is 44 feet and the available cross-section at elevation 496 is about 600 square feet. In the present design an equal cross-section would be available at about elevation 488, being the approximate new high water level under severe conditions. This corresponds with a rise in water level of about 6 feet and in connection with the assumed soil conditions, it is unlikely that serious scour would occur in the relatively short period of time over which severe

7.

conditions would persist. It is therefore recommended that this aspect be further studied. It would also be necessary to carry out additional work to check the soil conditions in the river bed by one or two shallow boreholes. It may then be feasible to adopt a spread footing type of foundation.

(b) Spread Footings

The footings for the piers should be founded at a depth of several feet below the maximum possible scour. For added safety, the river could be channelized beneath and upstream of the bridge.

The footings then would be placed in the very dense silt till stratum. In this stratum an allowable bearing pressure of 6000 pounds per square foot may be used. The pier load is of the order of 18 Kips per foot run, and the minimum footing width practical would probably induce a foundation pressure less than the allowable. Settlement under these pressures should be very small to negligible.

The footings for the abutments should also be founded in the silt till stratum. The footing for the west abutment should be founded at elevation 480 where an allowable bearing pressure of 6000 pounds per square foot may be used. For the east abutment, the footing could be placed at elevation 487 where, based on the results of borehole 2, an allowable bearing pressure of 1800 pounds per square foot may be used. Under this pressure, settlement will be small. The above has been based on the results of borehole 2 and it would be advisable to carry out another shallow borehole at the location of the east abutment.

(c) Excavations

The excavations for the piers must be carried several feet below present river level. It is considered that the excavations could be made without major difficulties, if

8.

the river is temporarily detoured to an excavated channel at the opposite side or in the centre of the river bed. A few shallow boreholes with in-situ permeability tests would check the feasibility of this method. The excavations for the piers should not interfere with the existing structure.

The excavations for the abutments will be well outside the existing abutment and should not directly interfere with the existing structure. Where the excavations touch the existing approach fills, temporary sheeting against these fills may be used. For the west abutment there will only be shallow excavations. At the east abutment new fill has recently been dumped and the excavations will encounter this fill which is of unknown composition. If it is of doubtful quality for embankment fill, it may have to be partially removed, thereby decreasing possible difficulties in the abutment excavations. Alternatively, short caissons resting on the till may be employed. The north wing walls of the existing structure could serve as temporary retaining walls during the second phase of construction.

(d) Embankments

The present approach fills are composed of soft to firm organic clayey till fill and would not be suitable under conditions of heavy traffic. In any case, it is recommended that it be removed behind the abutments and replaced with compacted clean granular fill. Drainage behind the abutments should be provided. Elsewhere, the existing approach fills may be improved by rolling in granular fill until high density is obtained. The recent fill outside the existing structure on the north east side is of unknown quality and should be further investigated.

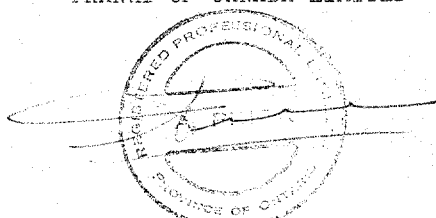
The end slopes of the embankments will be 1 vertical to $1\frac{1}{2}$ horizontal. The slopes should be rip-rapped or paved. If paving is used, drainage should be provided between the slope and the protective layer.

9.

CONCLUSIONS AND RECOMMENDATIONS

1. The site is covered by fill, underlain by dense to very dense silt till resting on bedrock at about elevation 455.
2. The river level at the time of the investigation was at about elevation 481.
3. Both piled foundations and spread footings have been discussed for the piers and abutments.
4. It is recommended that the possibility of serious scour be further studied.
5. Excavations and embankments have been discussed.
6. The two boreholes were put down at the time when a rigid frame structure was planned. In the discussions under (3) and (5) certain assumptions were made, based on extra-polation of the results of the 2 boreholes. It is therefore recommended that additional shallow boreholes be carried out to verify the soil conditions both at the location of the east abutment as well as in the river bed.

FRANKI OF CANADA LIMITED



A. Prior, P. Eng.
Divisional Soils Engineer

AP/DRB

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 stratigraphic plot is in accordance with the standard symbols of the National Research Council. The elevations given are referred to the Datum shown on the general heading.

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.

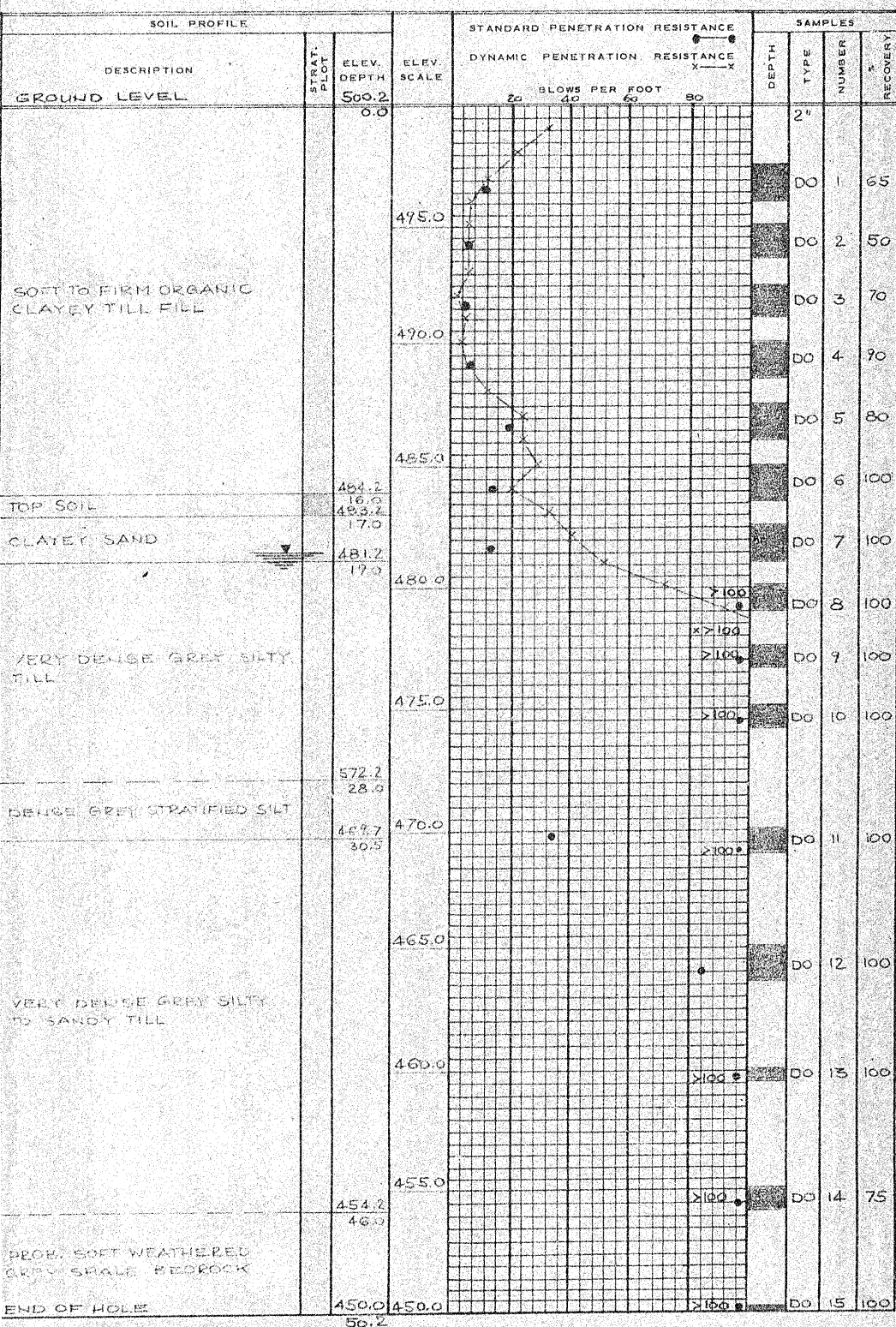
SAMPLES

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 a 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 1056 BORING I BORING DATE NOV 28 - 61
 DATUM GEOD. DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN



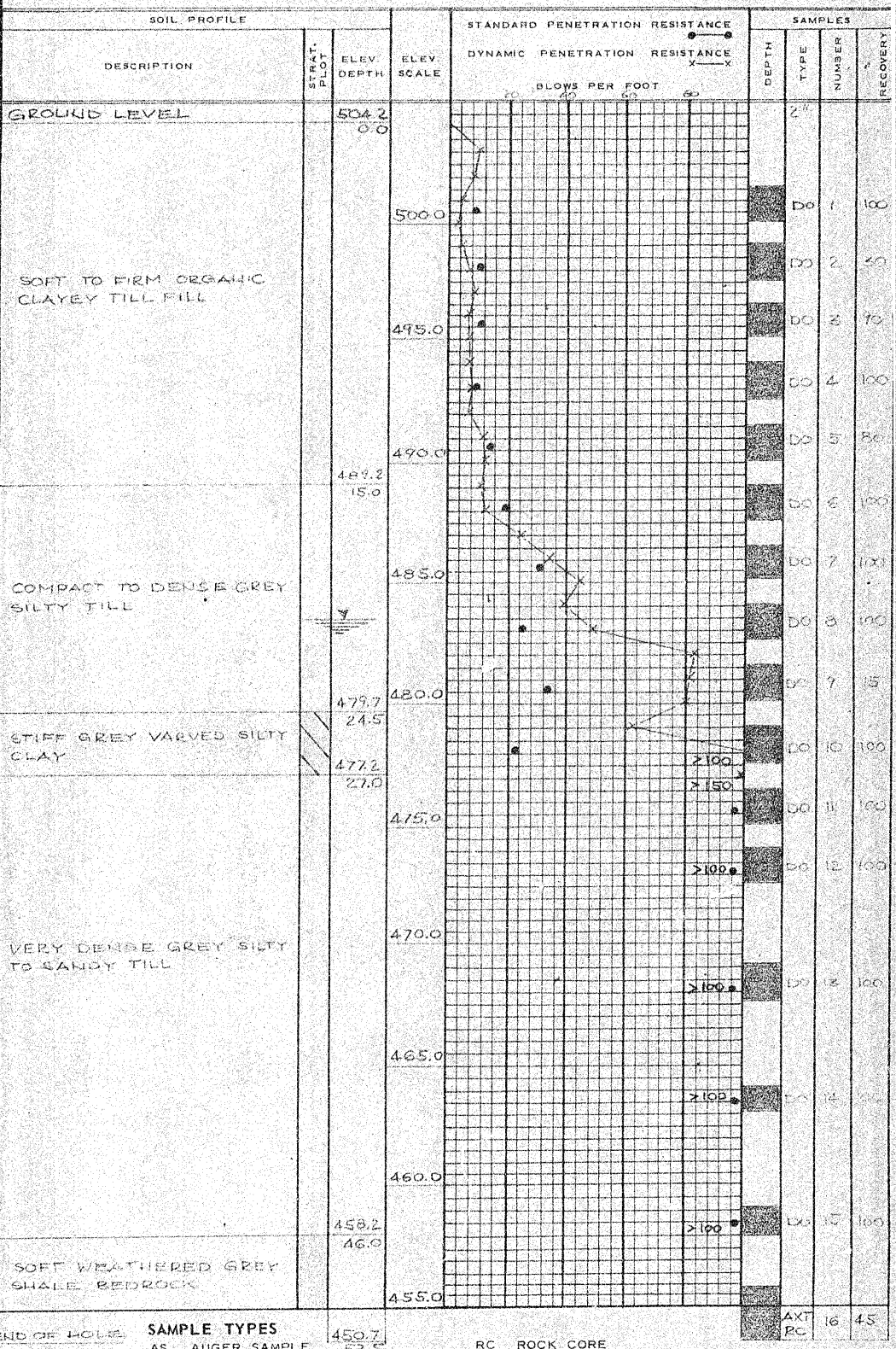
SAMPLE TYPES

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
 KF FIELD PERMEABILITY TEST
 T GROUND WATER LEVEL
 AT TIME OF BORING

REMARKS

CONTRACT PC 1056 BORING 2 BORING DATE NOV 28 - 61
 DATUM GEOD. DIAM. 4 1/2" HAMMER 140 LBS. DROP 30 IN



SAMPLE TYPES

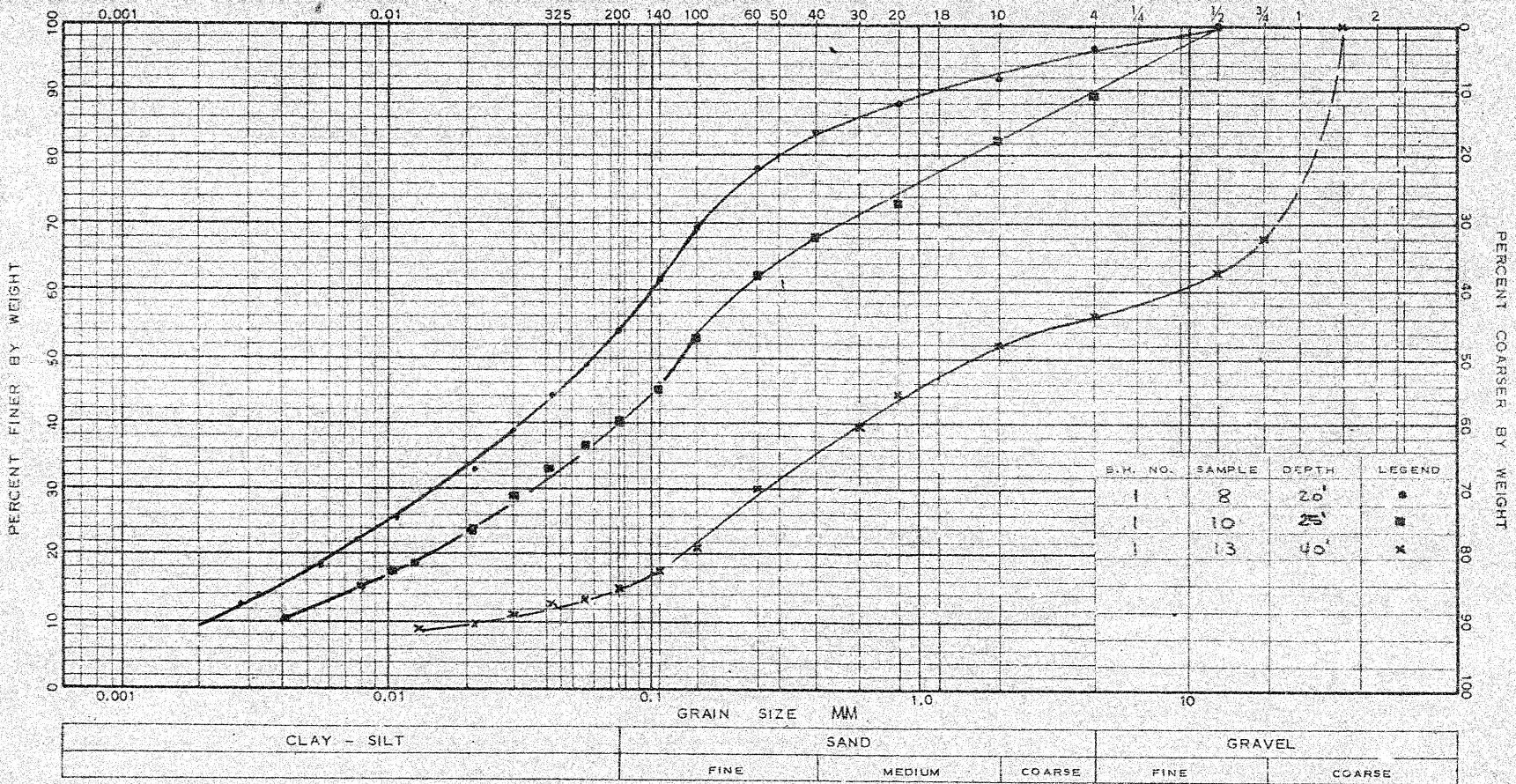
AS AUGER SAMPLE
 DO DRIVE OPEN
 DF DRIVE FOOT VALVE
 SO SLEEVE OPEN
 SF SLEEVE FOOT VALVE
 TO THIN WALLED OPEN
 TP THIN WALLED PISTON
 WS WASHED SAMPLE

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

REMARKS

FRANKI OF CANADA
SOIL INVESTIGATIONS
MECHANICAL ANALYSIS

P.C. NO. 1056 LAB NO. 1
PROJECT DHO MINICO CREEK
BOREHOLE NO. SAMPLE NO.
TESTED BY DATE
CHECKED BY DATE



Friday - March 1st 1963

Visit of construction requested by Al Kekim because the contractor has, during lowering of piles by boring (augering) encountered boulders which hindered further progress.

On site difficulties in augering were witnessed. However, because of past poor evidence the designed depth of piles should be reached under all circumstances and, therefore, if necessary a churn drill, should be brought in. This was outlined and subsequently agreed upon by Mr Harold Gilbert, District Construction Engineer.

Ag. Stinson,

OFFICE LOCATION -

DOWNSVIEW AVE.
KEELE ST. - HIGHWAY 401
TORONTO, ONTARIO.



ONTARIO

DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS -

DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS,
TORONTO 5, ONTARIO.

Bridge Division,
January 22, 1962.

File with 1103-1-61

MEMORANDUM TO:

Mr. A. Stermac,
Principal Foundation Engineer,
Department of Highways,
Room 107, Lab. Building,
DOWNSVIEW, Ontario.

*Len & Marty
Jan 24. 1962*

RE: W.P. 221-61
Mimico Creek Bridge *No Comments*
W.P. 222-61
Dixon Road Overpass
W.P. 225-61
Kenforth Dr. Access O'Pass *No Comments*
District #6

Attached you will find Preliminary Plans for the
subject structures. Would you please let us have
your comments at your earliest convenience.

F. DeVisser

FDeV/ea

F. DeVisser,
Bridge Location Engineer.

Note: Considered on 1/24/62. The drawings on contracts submitted for the Mimico Creek Bridge and Dixon Road Overpass are acceptable. The drawings for the Kenforth Dr. Access O'Pass are not acceptable. The drawings for the Kenforth Dr. Access O'Pass are not acceptable. The drawings for the Kenforth Dr. Access O'Pass are not acceptable.

HYDROLOGY REPORT

Mimico Creek at Dixon Rd.
W.P. 221-61 B.W. 581
District 6

The size of the watershed upstream from the Dixon Rd. Bridge is approx. 17.2 sq. mi. The gradient of the stream bed is fairly steep. Part of the land draining into the creek is being urbanized at a fast rate. (Toronto and Brampton Subdivisions, partial drainage from Toronto International Airport).

The existing structure has a clear span of 43'-9". During spring run-off and rain storms considerable scour under the bridge takes place, as well as on the west bank, which is receding downstream of the bridge due to undercutting. A 9 ft. deep scour hole under the east abutment developed during the 1954 October storm, and the bridge was closed during repairs. A diver was employed to assist the repair crew.

Water draining from the ditches in the north east corner causes considerable washout of the bank. Metro have informed us that a 3 ft. C.I. Pipe was installed several years ago, but washed out during a rain storm.

It was noted that downstream at Hwy. 27 and 401 there is a series of 45 ft. span barrel arch culverts. One of these was undermined and damaged during the 1954 October storm.

In view of the above, and considering the estimated calculated volume of water, that has to pass through the bridge, it is recommended that a 50 ft. clear span bridge be built. Sheet piling should be driven in front of the abutments to a depth of 15 ft. below present creek bed, except when footings on piles are used. Two feet thick random rip-rap to be placed on the west bank, approx. 25 ft. upstream and downstream of the bridge.

Summary of recommendations:

1. Span to be 50 ft. clear, or equivalent.
2. No skew required.
3. Bridge to be built in same location.
4. Unless footing are supported by piles, sheet piling should be driven in front of the abutments 15 ft. below creek bed. Exact limits of sheet piling will be given after the bridge site plan has been received.
5. Two feet thick random rip-rap to be placed on the creek banks to elevation 497.5 for a distance of 25 ft. upstream and downstream from the bridge.

7. D.

FDeV/bn

F. DeVisser,
Bridge Location Engineer.

FRANKI OF CANADA, LIMITED

SOIL INVESTIGATIONS

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

8th January 1962

Department of Highways,
Ontario,
Materials and Research
Section,
Downsview, Ontario.

Atten: Mr. A. Stermac, P. Eng.

Dear Sirs,

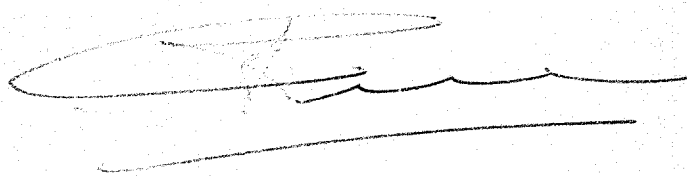
Re: Soil Investigation
W.P. 221-61
Mimico Creek

After submission of our report on the above soil investigation we noticed an omission on our part. Under "Soil Conditions" we discuss the results of laboratory tests; however, we did not present those results graphically on the Boring Records.

We are enclosing 14 copies of the results of the laboratory testing and we would appreciate it if you would attach these to the reports.

We apologise for this error and we hope that it has not inconvenienced you in any way.

Yours very truly,
FRANKI OF CANADA LIMITED



Enclosures (14)
AP/DRB

A. Prior, P. Eng.
Divisional Soils Engineer

Mr. A. M. Toye,
Bridge Engineer.

January 5, 1962.

FOUNDATION INVESTIGATION REPORT

Materials & Research Division,

By: Franki of Canada, Ltd.

(Foundation Section)

Attention: Mr. E. McCombie.

Re: Proposed Mimico Creek Bridge -
Dixen Road, Toronto, Ontario.
S.P. 221-61 -- District No. 6

Attached, we are sending you the Foundation Investigation report for the above-mentioned structure, submitted by Franki of Canada, Ltd.

We have reviewed the report and have found the factual data and information well presented. As far as the conclusions and recommendations are concerned, we herewith submit our comments for your consideration:-

In view of the relatively low allowable bearing capacity of 12" pre-bored concrete piles (25 T/pile), the use of spread footings seems to be a more favourable solution.

The original subsoil is a compact to very dense silt - (till stratum), where the minimum Standard Penetration Test value of 20 was recorded. Except for the East abutment, a safe bearing capacity of 3.0 T/sq.ft. can be used. At the East abutment, a safe bearing capacity of 2.0 T/sq.ft. at or below elevation 437.0 is recommended. The recommended elevation for the West abutment is 480.0, or below.

The pier footing elevations should be determined on hydrological requirements.

We believe that these additional comments, together with the information contained in the report, will prove to be adequate for your future design work. However, should there be any additional information required, please feel free to contact our Office.

ND/MAGF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
B. D. McMillan
I. C. Campbell
C. Fraser
A. J. Kovich
J. Roy
J. E. Grusnier
B. F. Saint
F. Norman
A. Watt

A. G. Sternac,
PRINCIPAL FOUNDATION ENGR.

Per:

M. Kovata
(M. Kovata,
CH. PRINCIPAL FOUNDATION ENGR.

Foundations Office -- Gen. Files.

RESULTS OF LABORATORY TESTING

<u>Borehole</u>	<u>Sample</u>	<u>Depth ft.</u>	<u>Moisture Content %</u>	<u>Unit weight p.c.f.</u>
1	1	4	6.6	
1	2	6	11.1	
1	3	8	15.2	
1	4	11	15.6	
1	5	13	19.8	
1	6	16	25.8	
1	7	18	17.8	
1	8	21	7.9	140
1	10	25	7.0	
1	12	36	8.5	
1	13	40	7.4	
2	1	4	16.4	
2	2	6	13.1	
2	3	8	13.0	
2	4	11	14.4	
2	5	14	14.9	
2	6	16	12.0	145
2	8	21	13.4	139
2	10	26	19.7	
2	11	29	6.9	148
2	13	26	8.9	

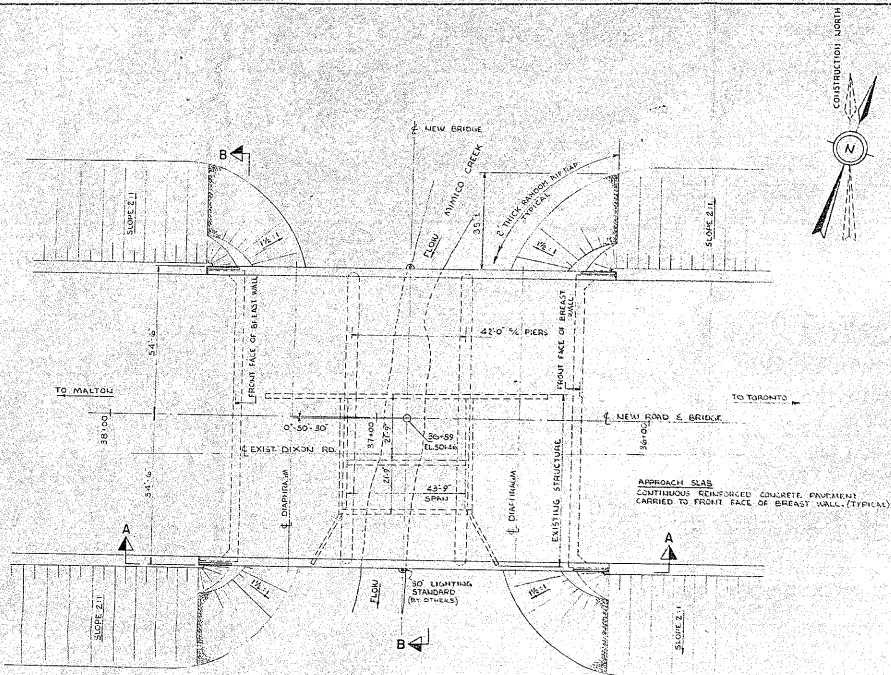
61-F-207-C

W.P. # 221-61

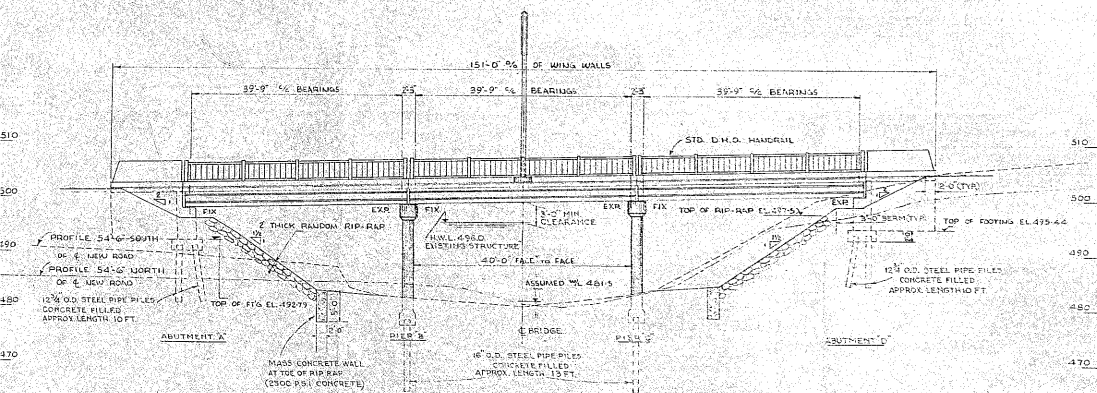
Dixon Rd. &

MIMICO CREEK

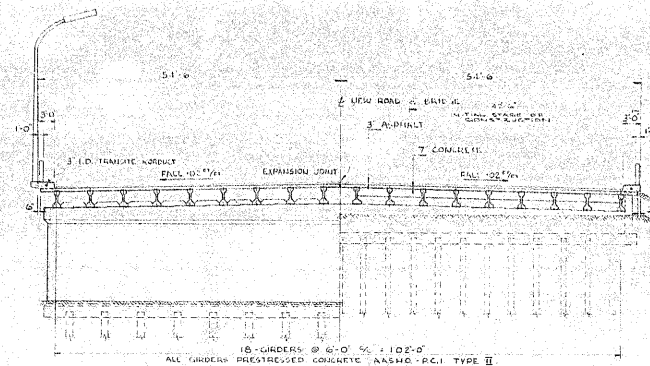
BRIDGE



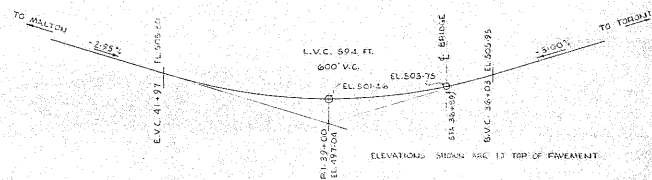
PLAN
SCALE: 1" = 20'-0"



ELEVATION A - A
SCALE: 1" = 10'-0"

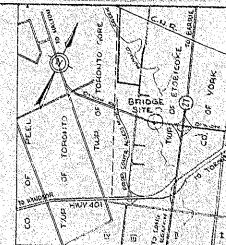


SECTION B - B
SCALE: 1" = 10'-0"



FINISHED CROWN PROFILE ALONG NEW DIXON ROAD

SCALE: HORIZ. 1" = 100'-0"
VERT. 1" = 10'-0"



LOCATION PLAN
SCALE: 1" = 0.8 M

WP 221-61

DE LEUW, CATHNER & COMPANY OF CANADA LIMITED
CONSULTING PROFESSIONAL ENGINEERS TORONTO
DEPARTMENT OF HIGHWAYS, ONTARIO
BRIDGE OFFICE, TORONTO

MIMICO CREEK BRIDGE

DIXON RD. 0.5 MI. W. OF HWY No. 27 DIST. NO. 6
SD. YORK LOT 21-22 CON. III
TWP. ETOBICOKE

GENERAL ARRANGEMENT

APPROVED JAN 19 1962

BRIDGE ENGINEER DESIGN ENGINEER

REVISIONS

DATE

DESCRIPTION

DATE JANUARY 1962

BRIDGE NO. D 4969-1

DATE

DESCRIPTION

DATE

DESCRIPTION

DATE

DESCRIPTION

