

67-F-226M

Hwy. # 2

CREDIT RIVER BRIDGE

LAKE SHORE ROAD

67-F-226M

SOILS DESIGN REPORT

FOR

APPROACHES TO THE CREDIT RIVER BRIDGE

IN THE TOWN OF PORT CREDIT

LAKESHORE ROAD

HIGHWAY #2 - CONNECTING LINK

February 1967

McCORMICK & RANKIN LIMITED
Consulting Engineers
PORT CREDIT OTTAWA

GENERAL DATA

The Credit River bridge is located on Lakeshore Road, Highway No. 2, and spans the Credit River within the Town of Port Credit. The approaches have been settling ever since their construction and, at present, the pavement is distorted and uneven. In addition, several severe longitudinal cracks and differential settlements over the old bridge abutments (which were not removed) are present. (See photographs)

In order to restore a good driving surface, it is the intention of the Town of Port Credit to reconstruct these approaches together with the Highway #2 Connecting Link contract between Front Street and the west limit of Port Credit).

No investigations were carried out at this time as adequate information was received from Peel County for design purposes.

BACKGROUND

The existing structure and approaches were completed in November 1959 under the jurisdiction of Peel County. The foundation report prepared by Racey MacCallum & Associates for the structure anticipated settlement of the approaches in the order of 6" to 1 foot upon application of 4 to 5 feet of fill.

As predicted, the major part of the settlement (up to 7") took place during the following 4 years at the end of which a 2-3" layer of hot-mix asphalt

was placed over the cracked and deformed pavement. Settlements of decreasing magnitude have taken place since then. A time-settlement curve prepared by Peel County shows a virtually balanced condition in February 1966. In the spring of 1965, the Port Credit Yacht Club dredged approximately 2 feet in the dock areas approximately 25' south of the retaining wall. No changes were noted in the pavement elevation.

In August of 1966, the Town of Port Credit added 2 feet of fill over the parking area north of the approach embankment. Only light-weight fill was placed from the embankment to 30 feet north of the toe of slope.

Elevations taken on the approaches since then are erratic and it is almost impossible to assess the significance of the readings. It should be noted that there has been no shift in the horizontal alignment.

In February 1967, a discussion was held with Mr. A. G. Stermac, the Principal Foundation Engineer, and it was concluded that:

1. Settlement would be a continual problem and, therefore, a maintenance-free roadway should not be expected.
2. No interpretation could be made at this time of the readings taken between August 1966 and the present.
3. Further readings should be taken at regular intervals before and after reconstruction.

RECOMMENDATIONS

The following recommendations are based on the assumption that the embankment is relatively stable under the existing loading conditions (see the time-settlement curve) and that some settlements will be tolerated.

1. Depth of Excavation

In order to maintain the equilibrium under additional fills, the following depths of excavations measured from proposed finished pavement grade are recommended:

| | | |
|----------------------------|---|----------------------|
| Sta. 1 + 50 to Sta. 2 + 00 | - | Taper from 6" to 30" |
| Sta. 2 + 00 to Sta. 2 + 75 | - | 30" |
| Sta. 2 + 75 to Sta. 4 + 00 | - | 24" |
| Sta. 4 + 00 to Sta. 5 + 90 | - | 18" |
| West approach | - | 18" |

Provide taper of 25' between the different depths.

2. Width of Excavation

The approach embankment should be excavated for the full width.

3. Backfill of Excavation

The backfill is to consist of light-weight fill (slag, assumed weight of 100 lb/cu. ft.)

4. Granular Material

It is recommended that 6" of G. B. C. "A" be placed over the light-weight fill. Since the existing embankment has 6" of G. B. C. "A", there is no weight differential due to this layer.

5. Pavement Depths

The pavement depth is to consist of:

| | | |
|------------------------------|---|--------|
| Surface Course, H. L. 1 | - | 1 1/4" |
| Upper Binder Course, H. L. 6 | - | 1 1/2" |
| Lower Binder Course, H. L. 6 | - | 2" |
| Sand Asphalt | - | 3/4" |

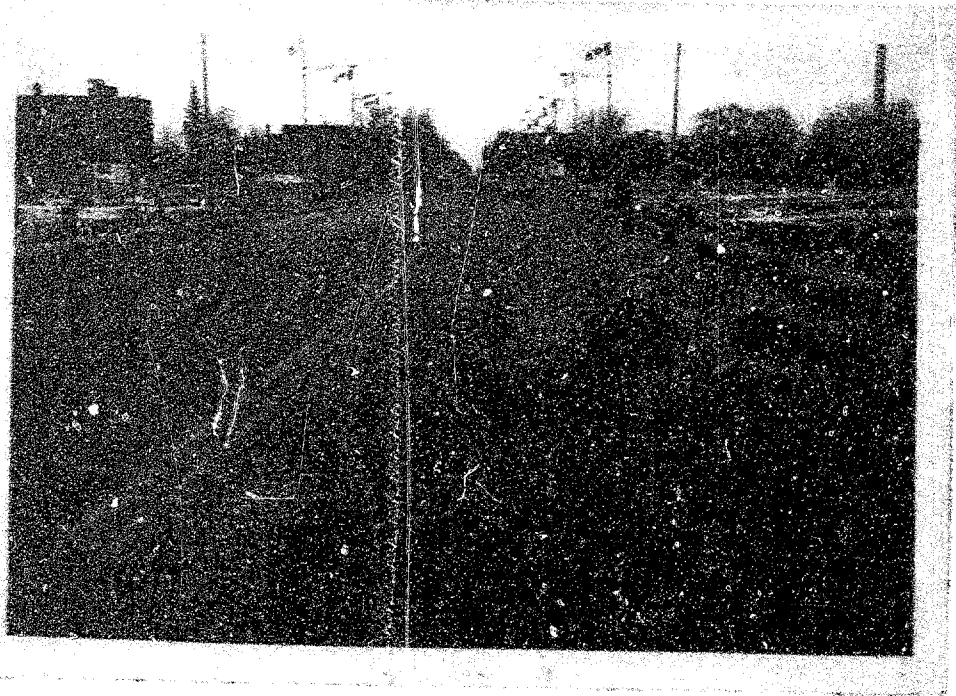
The total weight per square yard is equal to the weight per square yard of the existing 5 1/2" of pavement.

6. Removal of Old Bridge Abutments

It is recommended that the old bridge abutments between Station 3 + 60 and 4 + 50 be removed to at least 5 feet below profile grade to reduce their effect on any future settlement.

7. Alternate Treatment

An alternative treatment consisting of padding and resurfacing the existing pavement would not be satisfactory because distortion of the existing pavement is so severe that tolerable crossfall cannot be obtained without pavement removal or reconstruction of the curbs and sidewalks. Minimum cover over the centreline would require pads in excess of 6" at the edges. The additional weight of the padding may upset the equilibrium of the embankment and cause further settlement.



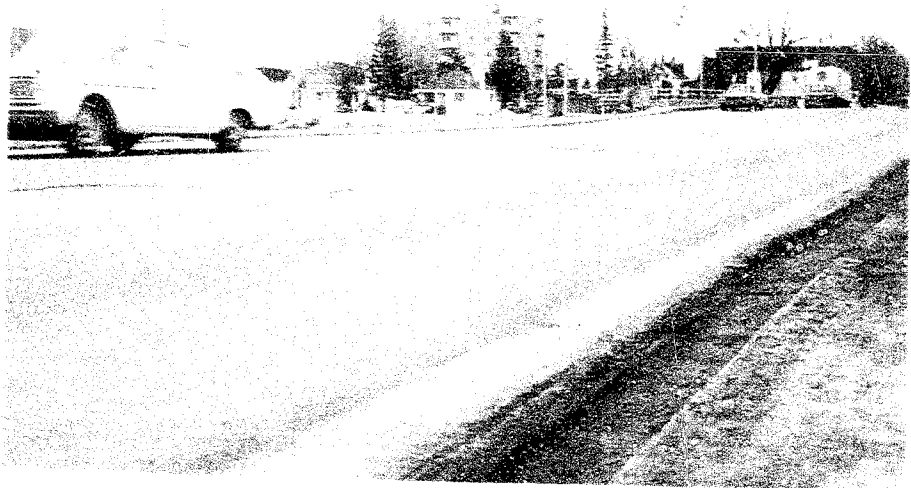
STATION 1+50 - View To The West Showing Longitudinal Cracks.



STATION 3+50 - View To Southwest Showing Old Abutments Reflecting Onto The Surface



STATION 1+50 - View To The West Showing Longitudinal Cracks.



STATION 3+50 - View To Southwest Showing Old
Abutments Reflecting Onto The Surface

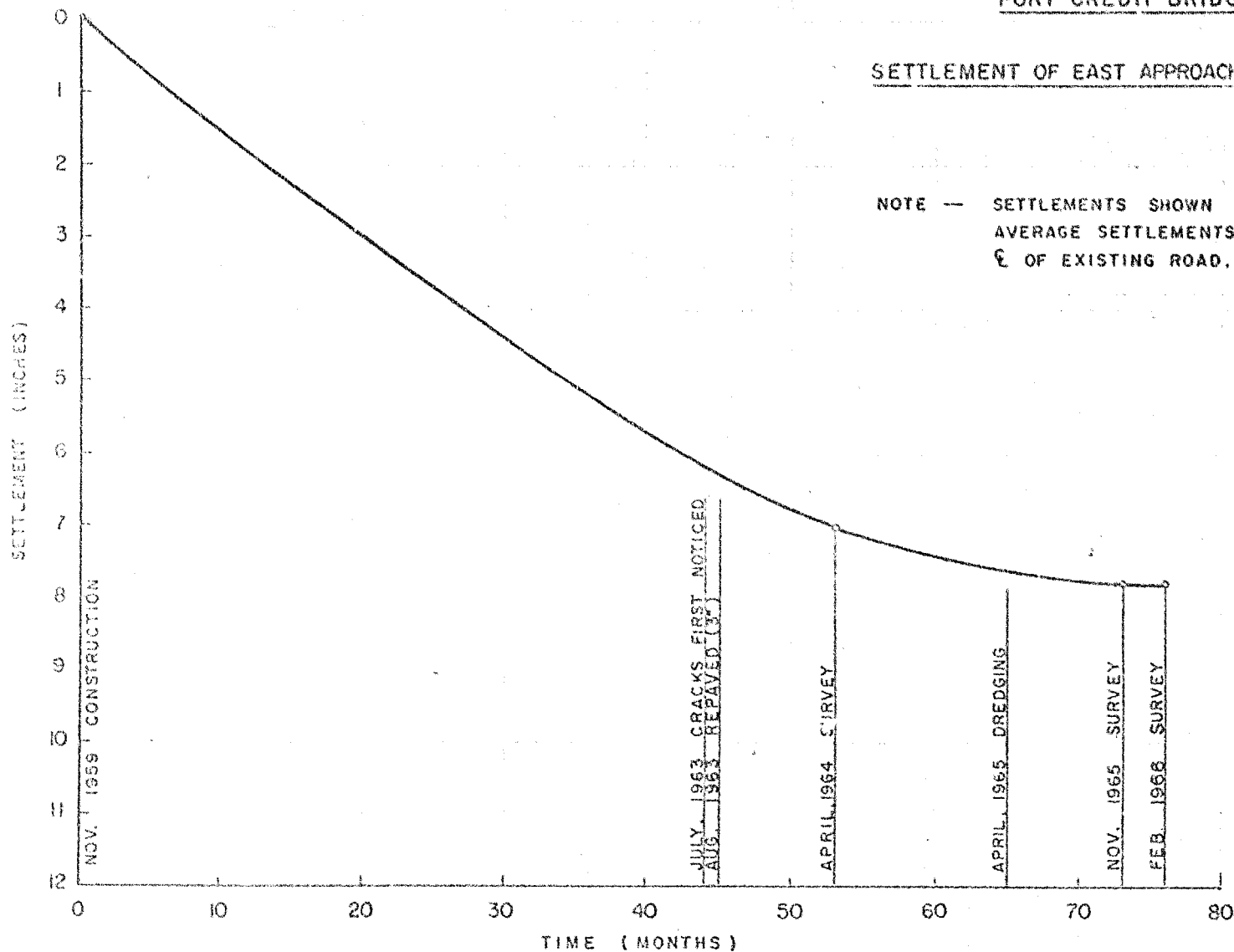
COUNTY OF PEEL
ENGINEERING DEPT.

PORT CREDIT BRIDGE

SETTLEMENT OF EAST APPROACH EMBANKMENT

FEB. 1966

NOTE — SETTLEMENTS SHOWN ARE
AVERAGE SETTLEMENTS ON
C OF EXISTING ROAD.

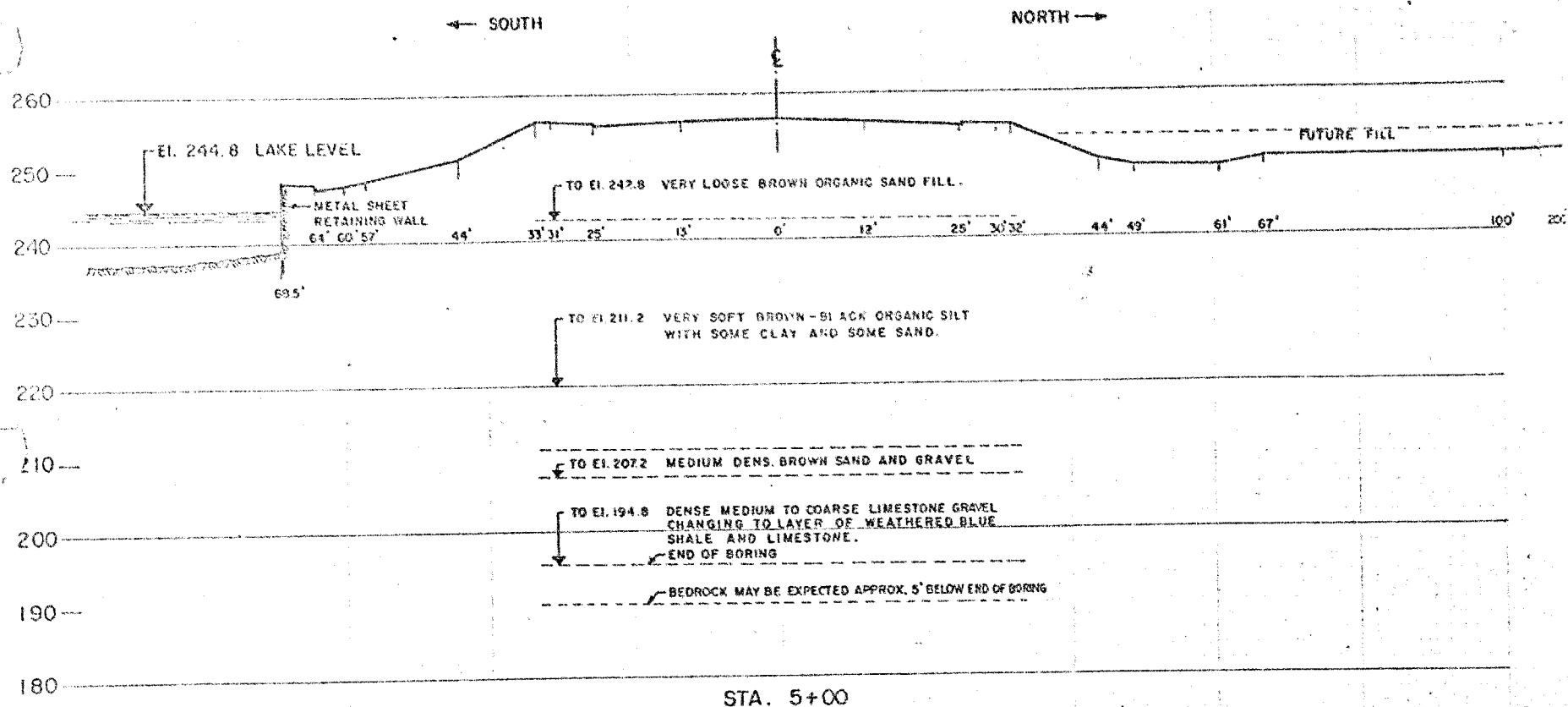


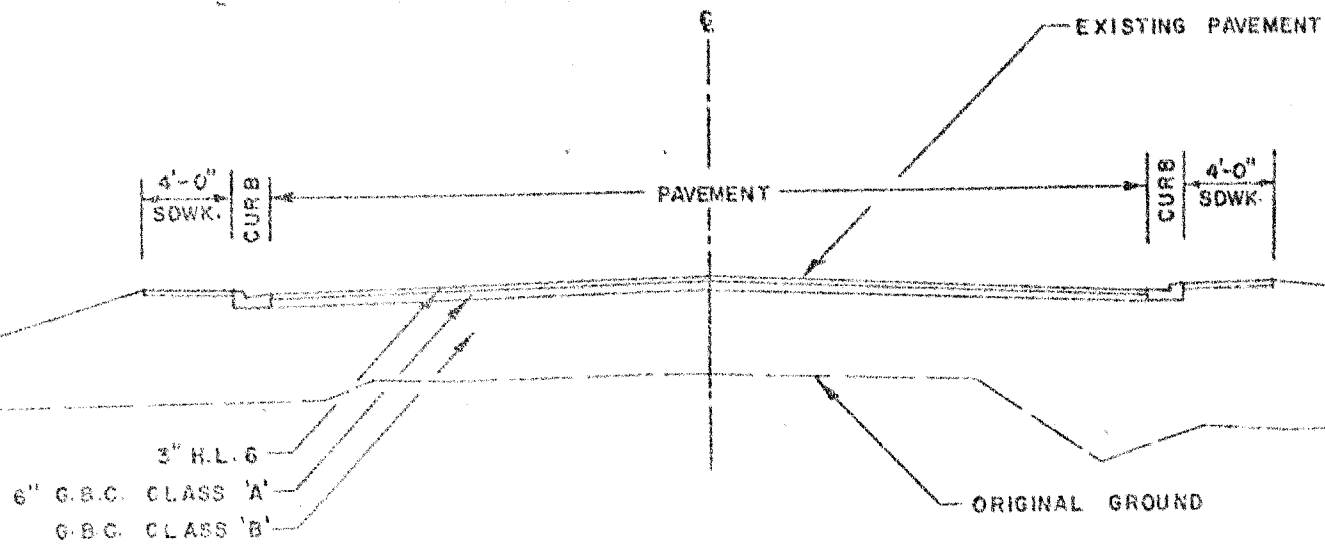
COUNTY OF PEEL
ENGINEERING DEPT.

PORT CREDIT BRIDGE

SECTION THROUGH EAST APPROACH EMBANKMENT

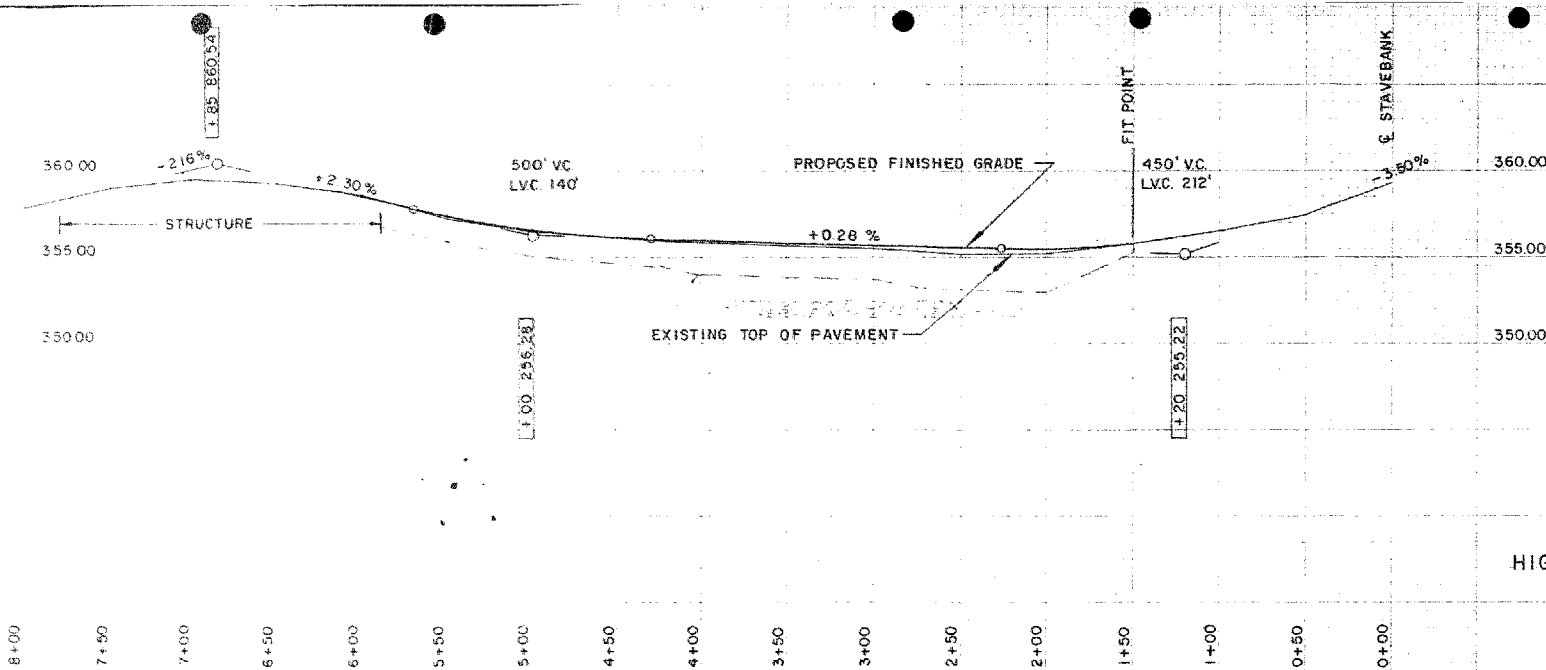
FEB. 1966.





TYPICAL CROSS - SECTION
SHOWING
COMPOSITION OF EMBANKMENT

HIGHWAY No. 2



SOILS PROFILE
OF THE
APPROACHES
TO THE CREDIT RIVER BRIDGE
IN THE
TOWN OF PORT CREDIT

HIGHWAY No 2 (LAKESHORE ROAD)

DATE: MARCH 9, 1967

Mr. H. Orlando,
District Municipal Engineer,
District 6 (Toronto),
Central Bldg.

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

March 1, 1967

Town of Port Credit,
Credit River Bridge Approach (East).

The undersigned visited the above mentioned site with Messrs. J. F. Beatson and G. Kitagawa of McCormick and Rankin Ltd. on February 27, 1967.

It is intended to carry out the reconstruction of the east approach to the bridge which has, over the past years, shown signs of distress manifested mainly in differential settlements.

From August 1966, to February 1967, elevations of the road surface were taken along a 450-ft. section of the east approach. The first readings were taken just prior to the placement of fill material on the north side of the highway. The fill material placed for a distance of 30 ft. from the highway, was lightweight slag.

Subsequent readings have shown that apparently, the road surface has risen somewhat. This is a rather surprising discovery which we find hard to understand or explain.

We are also advised by Mr. Beatson, that nails were found in the road surface, left there probably from a previous survey made some two years ago, and all these nails except one were found to be in perfect alignment. This indicates that apparently, no lateral movements (towards the lake) have taken place - which is certainly a comforting fact.

Readings taken in February 1967, indicate that in places, the heave of the road surface seems to have accelerated. A possible and probable reason for this could be frost action. Subsequent readings should either confirm or disprove this assumption.

We understand that if a major reconstruction of the approach to the bridge is undertaken, it will incorporate the raising of the highway in order to bring it to the right grade elevation. Presently, the road surface is not only in a rather

cont'd. /2 ...

March 1, 1967

poor condition, but the grade is too low due to previous settlements. The raising of the grade is to be achieved by partial subexcavation of the present road bed which is to be replaced by lightweight fill. In this way, a higher road elevation will be achieved without any load increase on the subsoil.

It is our opinion that this operation can be started in spite of the peculiar elevation recordings of the road surface. We would also recommend that the following be done in connection with this project:

a) Markers or bench marks should be established along the south side of the road, preferably at the bottom of the embankment slope and surveyed for elevation and alignment. The survey should be continued during construction and maintained on a monthly basis thereafter. Once it has been established that no movements are occurring, readings can be discontinued. We would, however, suggest that a period of one year be observed - possibly, with readings every two months at a later stage.

b) A pattern of nails be established on the road surface and an elevation and alignment survey be undertaken. A periodic check on these should be carried out.

c) The north half of the highway should be completed first.

On this occasion, we would like to point out that the previously described highway reconstruction will not necessarily eliminate all the problems. It is quite possible that further differential settlements may take place, resulting in pavement cracking which, in turn, may require certain maintenance work. It should be remembered that the subsoil in this location is of a rather poor quality and possibly, quite heterogeneous; therefore, further settlements and/or smaller movements cannot be entirely ruled out.

Major movements and possibly, a failure of the road embankment could be caused by either overloading of the area to the north of the road, or by removal of material on the south side of the road. The latter may occur if deepening of the marina is undertaken at any time in the future. Precautions should, therefore, be taken that such an operation never takes place.

We would appreciate it if we would be kept advised on the development of this project, and on the results of the readings of the above described observation points.

AGS/adeF

cc: Foundations Files ✓
Gen. Files

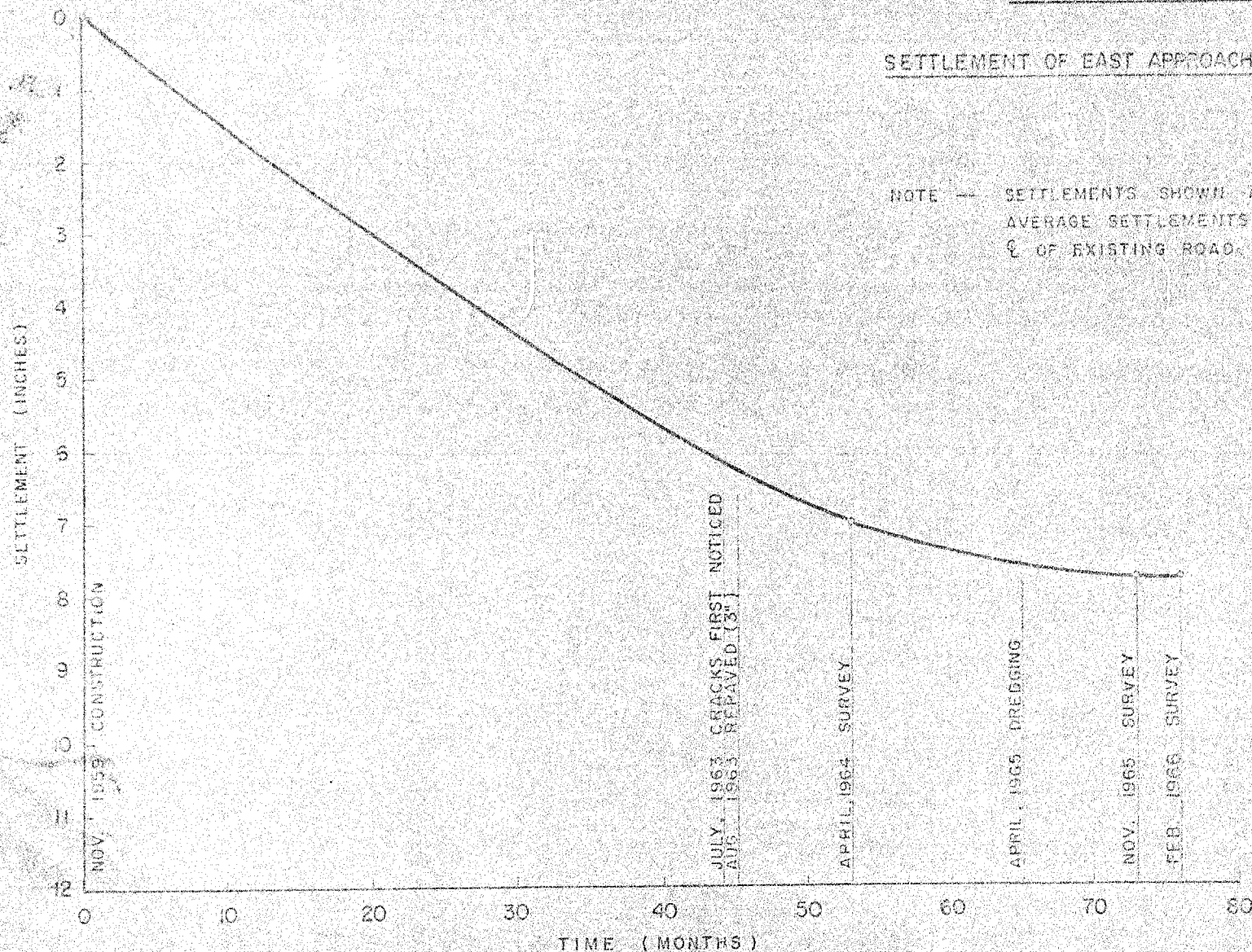
A. G. Sternac
A. G. Sternac
PRINCIPAL FOUNDATION ENGINEER

PORT CREDIT BRIDGE

SETTLEMENT OF EAST APPROACH EMBANKMENT

FEB. 1966

NOTE -- SETTLEMENTS SHOWN ARE
AVERAGE SETTLEMENTS ON
C OF EXISTING ROAD.



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

County of Peel,
Court House,
Brampton - Ontario.

FOUNDATION INVESTIGATION FOR PROPOSED
CREDIT RIVER BRIDGE, HIGHWAY NO 2, PEEL
COUNTY - ONTARIO.

Reference: S-610/T-1183

Racey, MacCallum and Associates
Limited.

29 May, 1958.

SUPER IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.

RACEY, MACCALLUM AND ASSOCIATES LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL  VANCOUVER
TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.SC., M.E.I.C., P.ENG.

A. ERIC RANKINE, B.SC., M.E.I.C., A.M.I.ELEC.E., P.ENG.

TORONTO DIVISION
27 CARLTON STREET

Our Reference: S-610/1-1188

29 May, 1958.

FOUNDATION INVESTIGATION FOR PROPOSED CREDIT RIVER BRIDGE HIGHWAY 2, PINK COUNTY, ONTARIO

This report describes the investigation carried out at the above site for the purpose of obtaining information for the design of the bridge abutments and piers. Following the description of the soil profile encountered at the site, recommendations are given regarding the suggested foundation type to be used.

Field work at the site commenced on 30 April, 1958, and was completed on 5 May, 1958, after two borings and seven cone penetration tests had been carried out. A standard diamond drill rig adapted for the taking of soil samples was used for the work. In the cohesive soils encountered at the site 2" inside diameter thin-walled Shelby tubing was used and 2" outside diameter split spoon samples were obtained in granular material. The number of blows of a 140 lb hammer dropping a fixed distance of one foot was recorded. This is known as a standard penetration test and is used in connection with empirical correlations to give the relative density of the granular soil encountered. A similar weight of hammer and dropping distance were used to drive the 2" diameter, 60° point anvil cone in the cone penetration tests, carried out to give a continuous record of the soil density variation.

SOIL PROFILE

The engineering data sheets, Enclosures 2 through 8, show the results of the two borings and the seven penetration tests which were made on each side of the Credit River at the site.

The soil at the site as shown by Boreholes 1 and 2, Enclosures Nos 2 and 3, consists of approximately 1 1/2 ft of river and lake sediments in a very loose condition overlying the shale and limestone beds of the Huronian formation. In Borehole No 1, 10 ft of core were taken in order to establish that bedrock had actually been encountered at the 1 1/2 ft depth, and it will be seen that the profile at this depth consists of a dense gravel gradually changing into the weathered blue shale and

Bur Ref: S-619/7-1188

29 May, 1958

SOIL PROFILE - Continued

limestone layers common to the area. The other Borehole, No 2, was carried down merely to refusal, which would probably take place at the top of this dense gravel layer overlying bedrock. It is felt likely that refusal indicated in the cone penetration tests would occur at the top of the same layer.

The two boreholes show a very similar soil profile, consisting of a few feet of extremely loose granular material overlying approximately 30 ft of a very soft brown-black organic silt containing some clay and some quantity of sand. The material, according to both the cone penetration tests and the number of blows recorded during the driving of the casing, appears to become slightly stiffer with depth. At a depth of 40 ft in Borehole No 1 and about 30 ft in Borehole No 2, the soil changes to a granular material consisting of a medium dense brown sand with some gravel in Borehole No 1 and a loose to medium dense brown sand and gravel in Borehole No 2. The split spoon penetration resistance results in both boreholes indicate the extreme looseness and softness of the material down to a depth of about 40 ft in both boreholes.

An examination of the profile demonstrated by the boreholes and cone penetration tests across the river shows that the material over all of the site appears to be very similar and the depth to bedrock is quite uniform. Refusal was reached in Borehole No 2 at Elevation 52.8, in penetration test No 5 at 50.2, in penetration test No 6 at 49.2, in penetration test No 7 at 51.2, and in penetration test No 4 at 54.8. According to the profile shown in Borehole No 1, it may be expected that bedrock in a fairly weathered condition would be encountered approximately 5 ft below refusal as indicated by the cone penetration tests.

FOUNDATION RECOMMENDATIONS

From the soil profile indicated in the borehole logs, it is obvious that the material is quite unsuitable for carrying any surface or near-the-surface foundations and that piles must therefore be resorted to. At this site, since water table is quite near the surface everywhere, it is possible to consider wood, concrete and steel piles as all being suitable for supporting the abutments and possible piers of the proposed structure; price alone would seem to be the determining consideration. The following table indicates the approximate price per foot and load carried by each of various types of piles. Where the loads indicated are in parentheses, the value of allowable load is dictated by the Toronto Building Code. It is assumed that in the case of steel pipe concrete-filled piles and steel H-piles the allowable load will be calculable on the basis of an allowable stress of 9,000 lbs. per sq.in. in steel and 600 lbs. per sq.in. in the concrete since the piles are to be driven to firm bearing in bedrock. Should the quality of the concrete be controlled by means of tests, it is possible that the allowable stress in the concrete could be raised. The values

Our Ref: S-610/T-1188

29 May, 1958

FOUNDATION RECOMMENDATIONS - Continued

have been calculated for these two types of piles on the basis of assuming them to act as short columns. The cost covers only on-shore driving. The table would indicate that little further consideration should be given to the use of either wooden piles or pre-cast concrete piles and that attention should be concentrated on the possibilities of using either steel pipe or steel H-piles.

Cost and Load Capacity of Different Types of Pile

| | Load, tons | Price, \$ |
|--------------------------------------------------------|---------------------------------------------|--------------|
| Wooden piles 12" dia. 50 - 70 ft long | (20) | 2.50 |
| Pre-cast Concrete Piles 20" Pre-cast | (44) | 3.00 |
| Steel Pipe, concrete filled, 1 1/2" diam. 1/4" wall | steel 19.5 concrete + 10.0 total 29.5 | 8.00 - 10.00 |
| 12" diam. 1/8" wall | steel 12.5 concrete + 28.5 total 41.0 | 7.00 - 9.00 |
| Points (for steel pipe) | | ca. 15.00 |
| Steel H-piles 10" x 10" WF, 60 lb. | 79.5 | 8.00 - 10.00 |

With regard to the abutments on each side of the bridge, either of these two types of pile would seem to be a possibility with the likelihood that the steel H-piles would be somewhat cheaper than the use of the smaller steel pipe concrete-filled piles. However, when consideration is given to the pier which it is proposed to found in the river, there are two alternative designs: either steel H-piles or steel pipe piles can be used, and a pier constructed on top of the piles entirely below water level for the purpose of carrying the bridge structure, or steel pipe piles can be driven and allowed to stick up 10 or 11 ft above water level to support the bridge directly by means of a concrete capping beam. Although the final decision may be based on aesthetic considerations, it is very obvious that the latter method would be much cheaper in total construction since the cost of pouring a concrete pier below water line would involve constructing a sheet pile coffer around the driven piles and clearing working space inside the sheet piling.

Our Ref: S-612/7-1159

29 May, 1953

At the approaches to the proposed bridge, it will be necessary to place approximately 4 or 5 ft thickness of fill. It will be seen from the boring logs that very compressible material underlies the bridge approaches, and it may be anticipated that approximately 6 ins. to 1 ft of settlement will take place in the underlying soil due to the application of 4 or 5 ft of fill to an area which has not been previously filled. If the approaches to the bridge remain the same as those of the present bridge, then additional fill will probably need to be placed on each side of the existing approaches to accommodate the greater width. Since the underlying soil has already been stressed by the application of the present fill for some years, it is unlikely that further settlements of a severe magnitude will take place unless greater heights of fill are placed on the site.

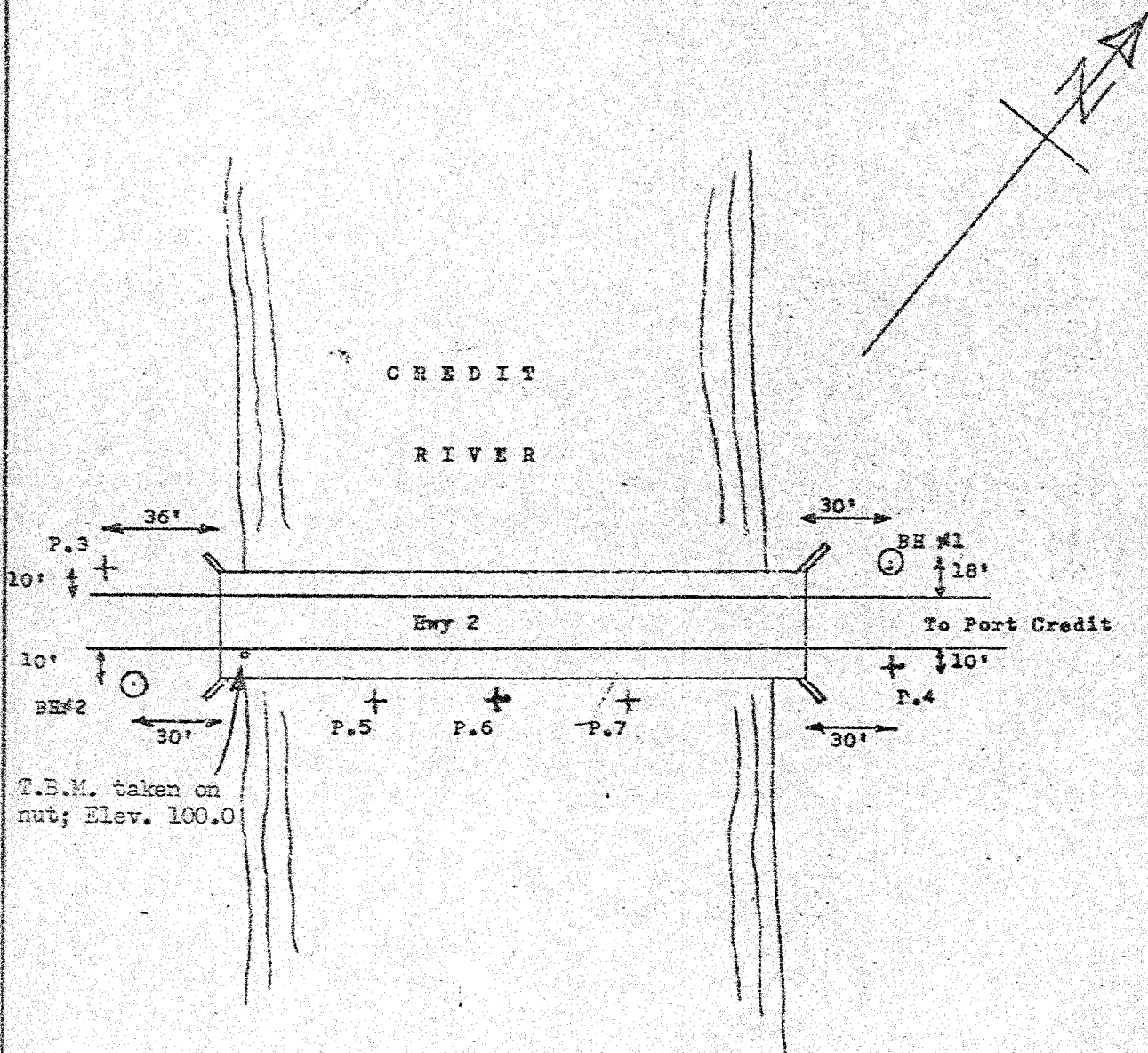
Ronald F. Scott

Ronald F. Scott, P. Eng.

RFK/EA



Prep. By R.F.S.



SITE PLAN

Foundation Investigation,
Credit River Bridge,
Peel County,
Ontario

○ Borehole

+ Cone Penetration Test

Not to scale

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: CREDIT RIVER BRIDGE,
 Location: PEEL COUNTY, ONTARIO.
 Hole Location: See Enclosure No 1.
 Hole Elevation and Datum: 95.0 feet. BM = 100.0
 Field Supervisor: H.G. Prep.: J.S.
 Driller: M.C. Checked: R.F.S. Date: 22.5.'58

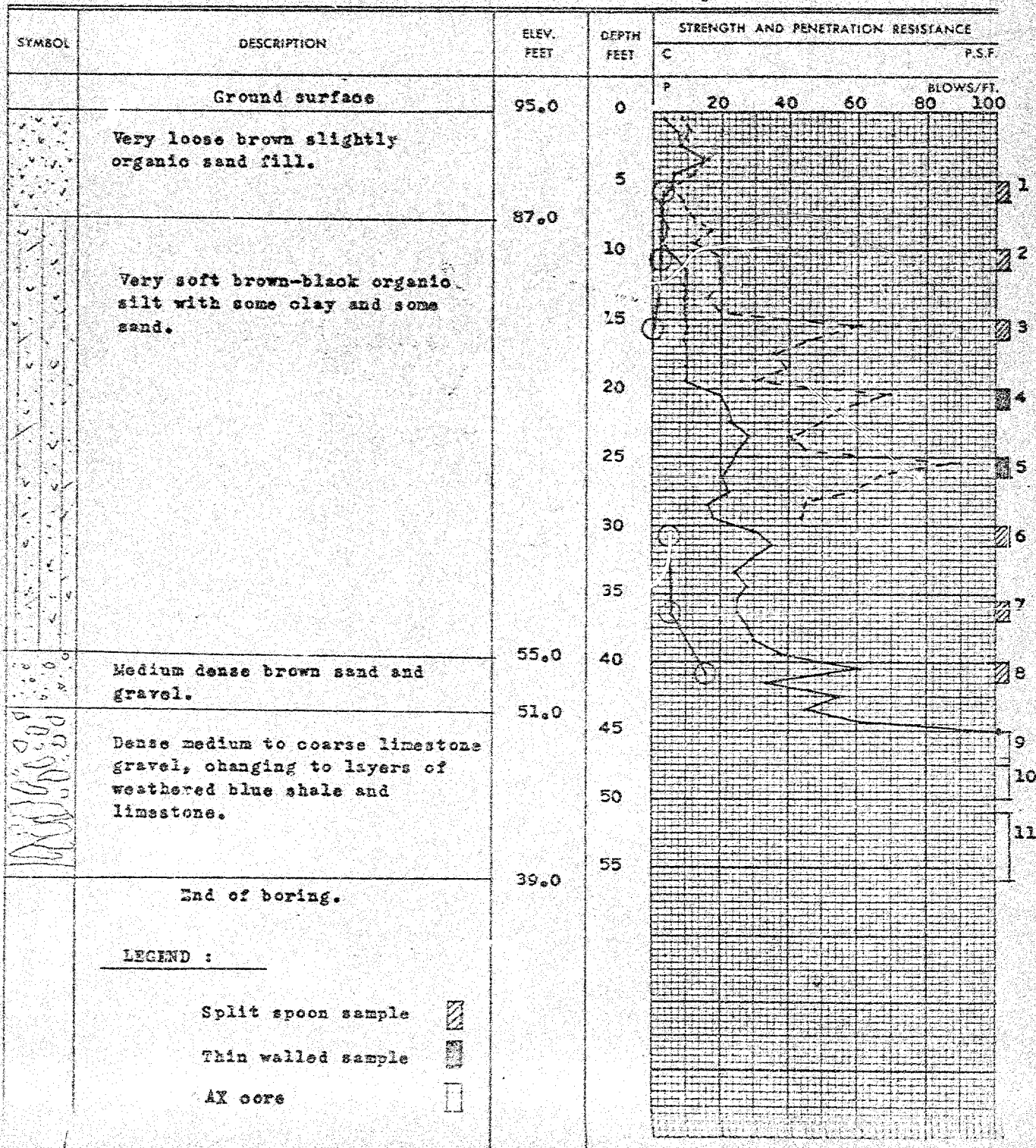
LEGEND

Shear Strength (C)

Unconfined compression
 Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube
 2" Dia. Cone
 Casing

⊕
+³⊕
⊕
⊕

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

Project: **CREDIT RIVER BRIDGE,**
Location: **PEEL COUNTY, ONTARIO.**
Hole Location: **See Enclosure No 1.**
Hole Elevation and Datum: **96.8 feet. BM = 100.0**
Field Supervisor: **H.G. Prop.: J.S.**
Driller: **M.C. Checked: R.F.S. Date: 22.5.58**

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

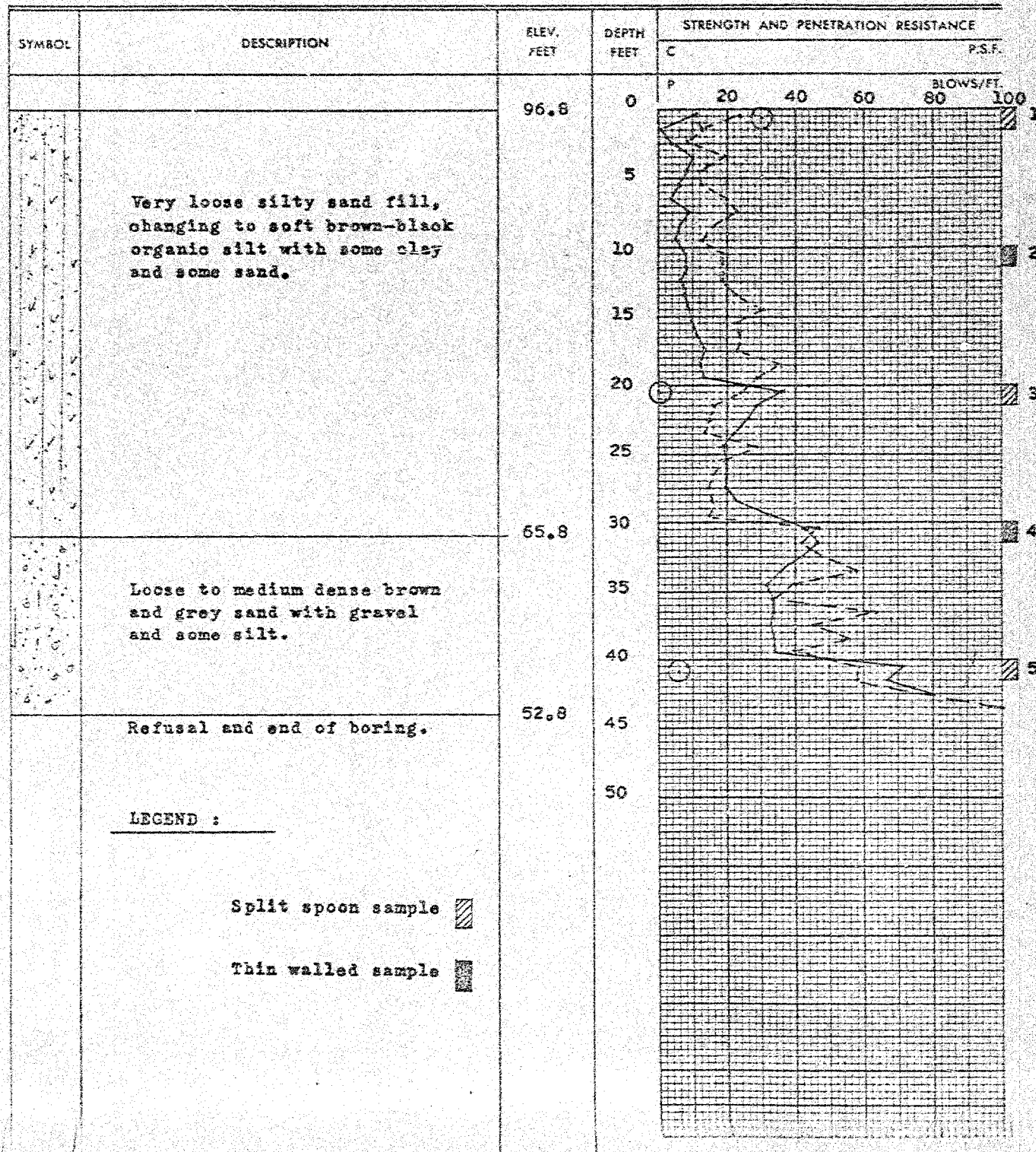
2" Split tube

2" Dia. Cone

Casing

⊕
+*

—○—○—



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Standard~~ Cone test: 3

Project: CREDIT RIVER BRIDGE,
 Location: PEEL COUNTY, ONTARIO.
 Hole Location: See Enclosure No 1.
 Hole Elevation and Datum: 97.3 feet. BM = 100.0
 Field Supervisor: H.G. Prep.: J.S.
 Driller: M.C. Checked: R.F.S. Date: 22.5.58

LEGEND

Shear Strength (C)

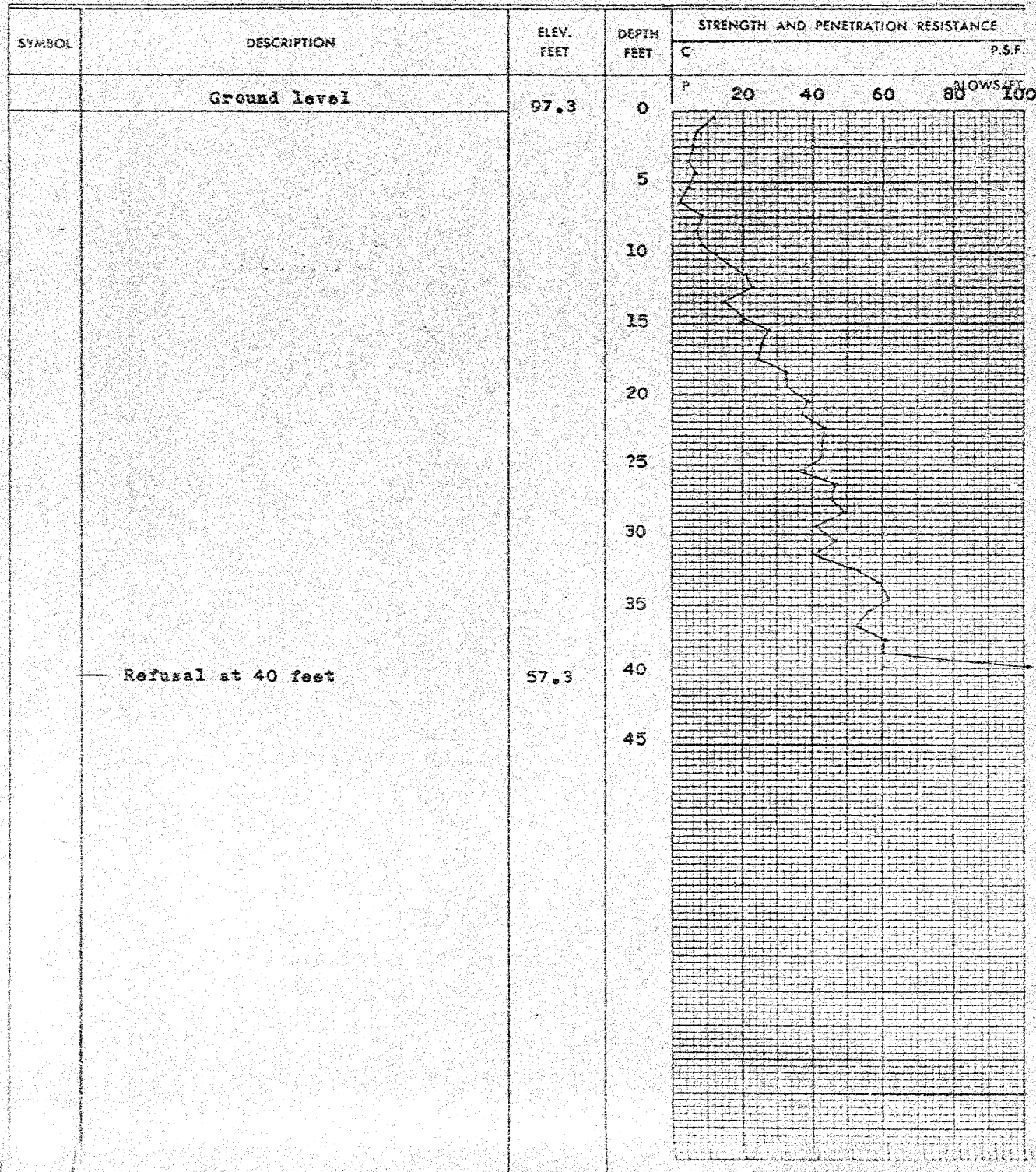
 Unconfined compression
 Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Standard~~ Cone test: 1

Project: CREDIT RIVER BRIDGE,
 Location: PEEL COUNTY, ONTARIO.
 Hole location: See Enclosure No 1.
 Hole Elevation and Datum: 96.8 feet. BM = 100.0
 Field Supervisor: H.G. Prep.: J.S.
 Driller: M.C. Checked: R.F.S. Date: 22.5.58

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

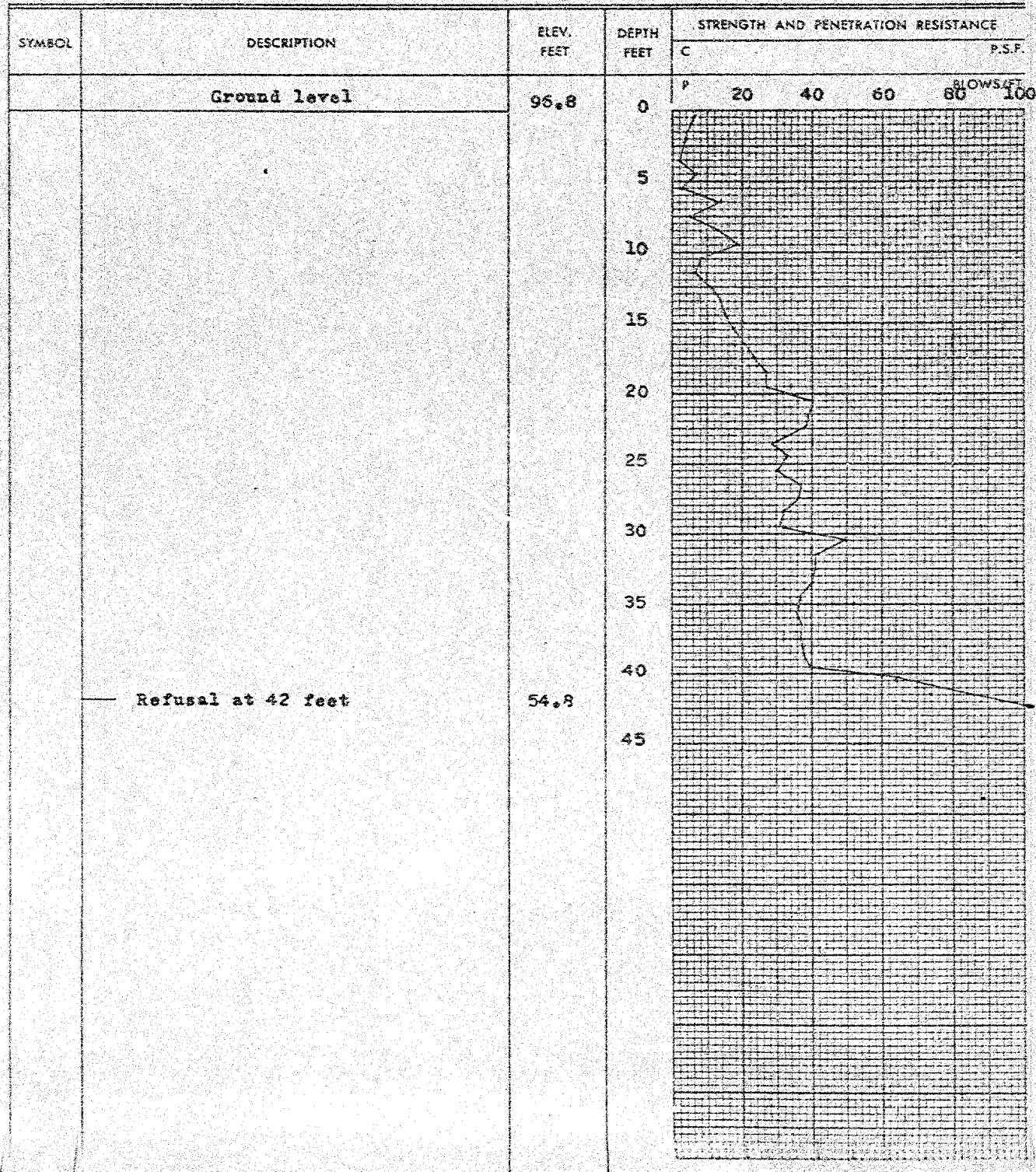
2" Split tube

2" Dia. Cone

Casing

⊕
+s

⊕ ⊕



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Standard~~ Cone test: 5

Project: CREDIT RIVER BRIDGE,
 Location: PEEL COUNTY, ONTARIO.
 Hole Location: See Enclosure No 1.
 Hole Elevation and Datum: 89.2 feet. BM = 100.0
 Field Supervisor: H.G. Prep.: J.S.
 Driller: M.C. Checker: R.F.S. Date: 22.5.58

LEGEND

Shear Strength (C)

Unconfined compression

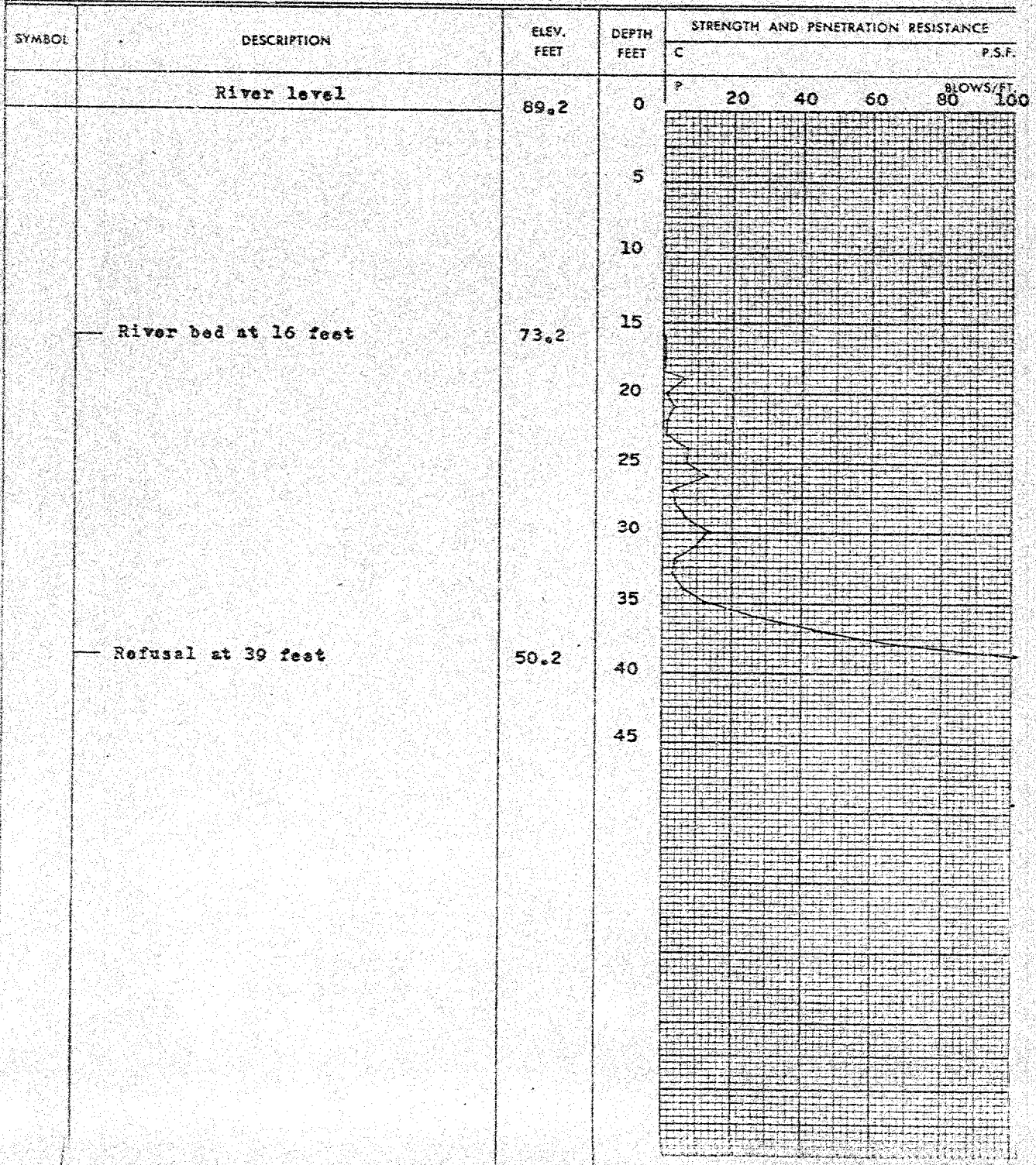
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

⊕
4"

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Standard~~ Cone test: 6

Project: CREDIT RIVER BRIDGE,
 Location: PEEL COUNTY, ONTARIO.
 Hole Location: See Enclosure No.1.
 Hole Elevation and Datum: 89.2 feet BM: = 100.0
 Field Supervisor: H.G. Prep.: J.S.
 Driller: M.C. Checked: R.F.S. Date: 22.5.158

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

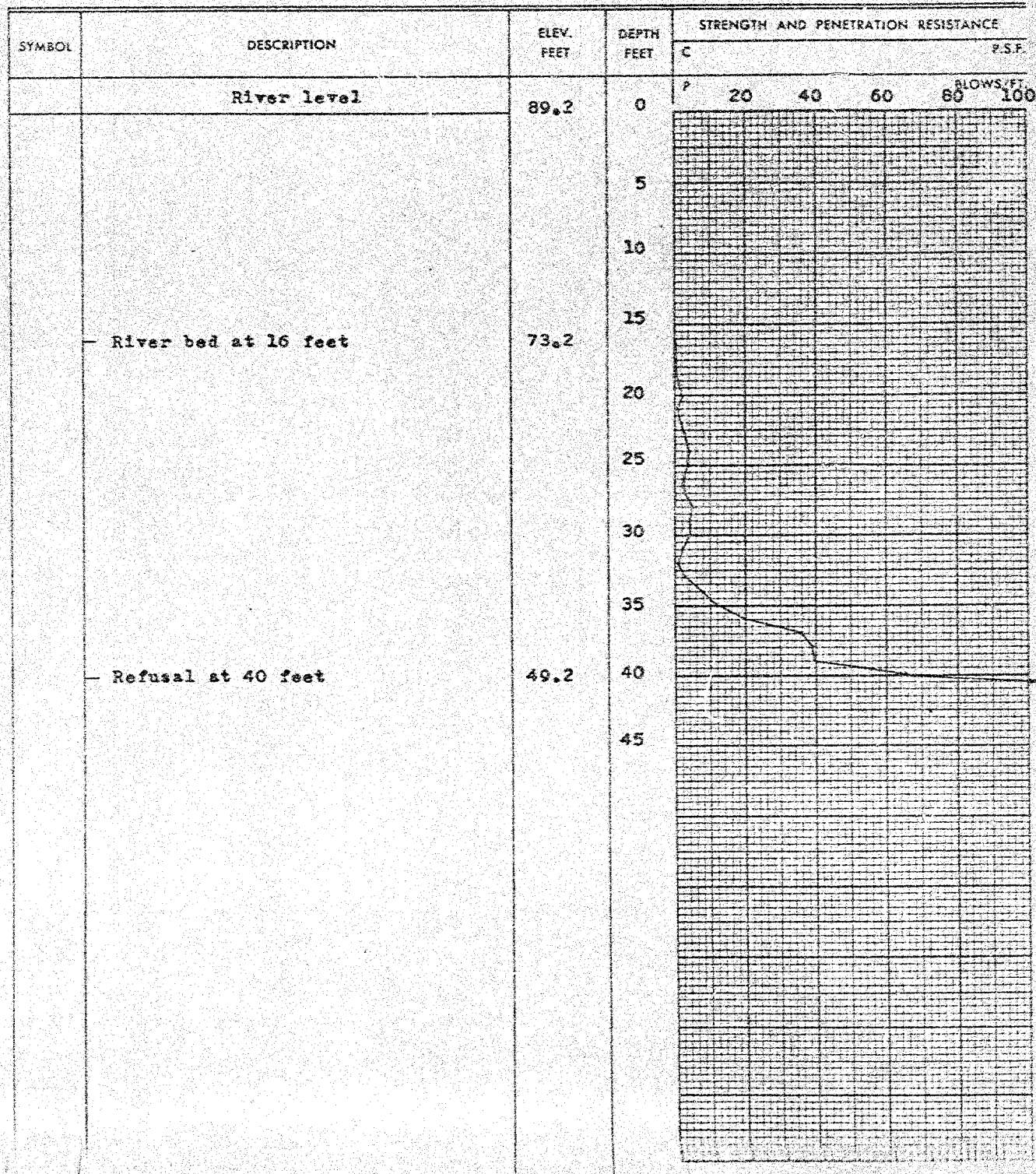
2" Split tube

2" Dia. Cone

Casing

⊕
+3

—○—○—



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for ~~Standard~~ Cone test: 7

Project: CREDIT RIVER BRIDGE,
 Location: PEEL COUNTY, ONTARIO.
 Hole Location: See Enclosure No 1.
 Hole Elevation and Datum: 89.2 feet: BM = 100.0
 Field Supervisor: H.G. Prep.: J.S.
 Driller: M.C. Checked: R.F.S. Date: 22.5.58

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

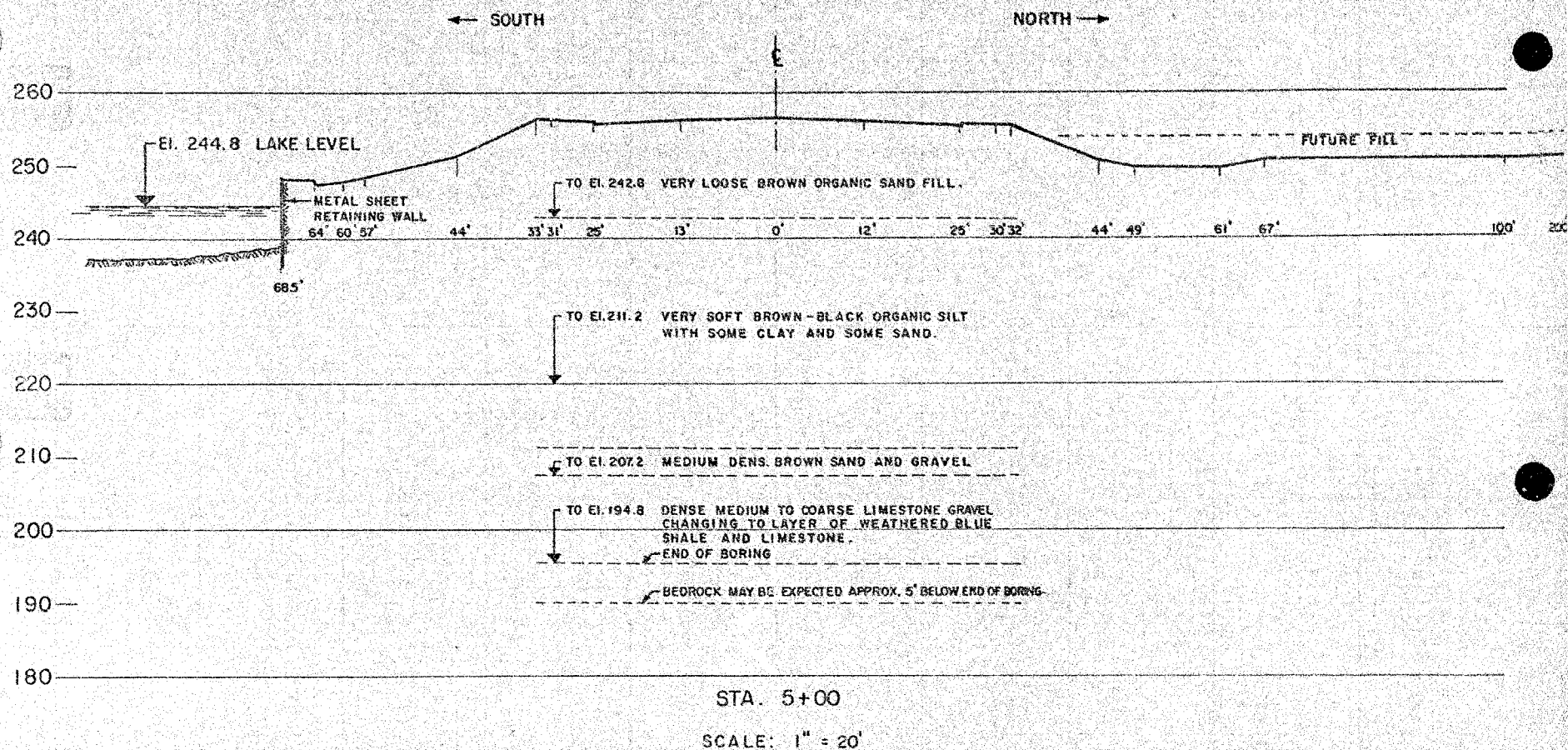
Casing

| SYMBOL | DESCRIPTION | ELEV. FEET | DEPTH FEET | STRENGTH AND PENETRATION RESISTANCE | | | | |
|--------|----------------------|---------------|---------------|-------------------------------------|--------|----|----|--------|
| | | | | C | P.S.F. | | | |
| | River level | 89.2 | 0 | P | 20 | 40 | 60 | 80 100 |
| | | | 5 | | | | | |
| | | | 10 | | | | | |
| | | | 15 | | | | | |
| | River bed at 19 feet | 70.2 | 20 | | | | | |
| | | | 25 | | | | | |
| | | | 30 | | | | | |
| | | | 35 | | | | | |
| | Refusal at 37 feet | 51.2 | 40 | | | | | |

PORT CREDIT BRIDGE

SECTION THROUGH EAST APPROACH EMBANKMENT

FEB. 1966.



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT.