

**H. Q. GOLDER & ASSOCIATES LTD.**

**CONSULTING CIVIL ENGINEERS**

H. Q. GOLDER  
V. MILLIGAN

2444 BLOOR ST. W.  
TORONTO 9  
HO. 7-9201

February 4, 1962

Morrison, Hershfield, Millman & Huggins Ltd.,  
96 Bloor Street West,  
TORONTO 5, Ontario.

Attention: Mr. J. F. Gregg, P. Eng.,

RE: HUMBER RIVER BRIDGE AT BOLTON  
HIGHWAY NO. 50, DISTRICT NO. 6  
WP35-60.

Dear Sirs:

Further to the meeting between your Mr. Gregg and the writer on February 2, 1962, we have reviewed the data supplied to us concerning the soil conditions at the site of a proposed bridge on Highway No. 50 in Bolton, Ontario. The data supplied included copies of correspondence between your firm and the Department of Highways, Ontario, drawings Nos. D4724-1 and D4724-2, of the proposed bridge foundations and detailed memoranda concerning the soil conditions at the site and the soil conditions at each of two bridges located on King Street, Bolton.

The proposed bridge foundations are to be spread footings founded in a stratum of stratified silt, clayey silt and clay, some 11 to 12 feet below normal river water level. Because of the depth below water level to foundation grade excavation in the dry cannot be guaranteed and a "piping" condition could well occur. It was therefore proposed by the Foundation Section of the Department of Highways, Ontario, that interlocking steel sheet piling be driven around the footing perimeter and the excavation carried out within a strutted cofferdam. The sheet piling would be driven to a penetration of about 8 to 10 feet below the footing elevation and would be left in place as scour protection. If during construction "boiling or piping" should develop, the cofferdam would be flooded and the excavation then carried out under water. The base of the excavation would be covered with tremie concrete seal prior to final dewatering. All the existing evidence

concerning the soil conditions at this site indicates that this proposed method of construction, as incorporated in the design, is a sound engineering solution.

Now, it has been suggested by the Department of Highways, Ontario in a letter by Mr. A. Selby dated January 18th, 1962 and in a memorandum by Mr. S. McCombie dated January 29th, 1962, that the proposed sheet piling procedure would add approximately \$50,000.00 to the cost of the structure. Whether this sum is correct or not, I do not know. It is being proposed, in the two letters referred to above, that the sheeting should not be used at the start of the construction and should be used only as experience in the dewatering of the excavations for one of the wing walls proves it to be necessary.

On February 5th, 1962 I had a discussion of this recommended construction procedure with Mr. A. Sterns and Mr. A. Selby of the D.H.O., Foundation Section, they pointed out that this method of construction may be unsafe but they considered it to be justified on a trial basis in view of the fact that the two King Street bridge foundations close to the site had been constructed without the use of sheeting. With Mr. Selby, I examined laboratory analyses and soil samples from the sites to which they referred. The results of D.H.O. borings are shown in an air wing No. 61-8-116, dated January 18th, 1962. It is significant to note that the soil conditions at the sites of the bridges on King Street are not comparable with soil conditions for the proposed Highway #50 structure. The King Street structures are founded in a relatively uniform clayey silt of high relative density. Typical classification tests for this material indicate that the liquid limit of the clay silt is of the order of 25 to 31, the plastic limit approximately 12 to 17 and the water content generally well below the liquid limit, usually 2 percent.

On the other hand, the highly stratified sub-soil conditions at the proposed bridge are anything but uniform. Individual layers of silt and clayey silt range in thickness generally from 2 to 6 inches and are often quite erratic; individual clay layers are relatively thin and of the order of 1 inch or less in thickness. Six Atterberg limit tests only were carried out on typical clayey silt layers from samples at foundation grade and some 10 feet below it. These tests are listed below:

<u>L<sub>1</sub></u>	<u>P<sub>1</sub></u>	<u>w</u>
21.5	18.2	22.9
21.0	19.3	22.6
22.5	18.2	23.2
22.2	19.2	23.2
22.0	20.1	22.9
22.7	19.0	23.6

It may be noted that in every instance the water content of these layers is above the liquid limit. Consequently, while the presence of the thin clay layers (which were not tested in the laboratory) will tend to cut down the possibility of piping or boiling, the presence of the clayey silt layers, (probably of low shearing strength as we may infer from the Atterberg limit tests) raises the question of instability at the base of the excavation during construction. This question of instability is of some importance when we consider that the north abutment of the proposed bridge is located close to the relatively steep slope at Queen and Hickman Streets and the south abutment will be within some 10 feet of the existing Humber River.

Thus, should instability develop, it is possible that sheeting can not then be used and the bridge may have to be founded on friction piles. We understand that in this case, the cost of the structure would be increased by approximately \$50,000.00. Instability could have more severe results than simply a change in the foundation type but in view of the stratified character of the sub-soil it is difficult to make an accurate analysis of what these effects would be.

This possibility of instability was discussed with Mr. Stermac and Mr. Selby at our meeting and it was agreed that to make an initial trial excavation at one wing wall without sheeting posed a risk which could not at the present time be accurately assessed. To have this risk, provided that construction was carefully controlled may be possibly justified should a saving of \$50,000.00 result, but to have this risk for a much lesser possible saving, say \$25,000.00, was not justified.

This then represents the substance of my review of this problem. Whether the exact sums of money referred to above are \$50,000.00 or \$25,000.00 I leave to you to estimate.

Yours faithfully,

H. A. GOLDBER & ASSOCIATES LTD.

*H. A. Goldber*  
H. A. Goldber, P. Eng.

VW/jb  
6207

February 2, 1962.

Mr. J. L. Davis,  
Department of Highways, Ontario,  
Parliament Buildings,  
Toronto, Ontario.

Attn: Mr. F. L. Hewson, P.Eng.  
Consultant Liaison Engineer  
Re: W.P. 35-62 - Luther River Bridge  
at Selby - Sheet Piling

Dear Sir:

This will acknowledge receipt of your letter dated January 30, 1962 containing reference to the hydrology and foundation reports in addition to your instructions regarding the elimination of the sheet piling and alterations to the wingwalls.

During our telephone conversation of yesterday with your Mr. Bruce Davis we indicated that we are not satisfied that these alterations are desirable, remembering that, as your consulting engineers, we are obliged to accept responsibility for the adequacy of the design.

It was agreed that we should further study the latest soils information and following that, if we are still concerned about the elimination of the sheeting, we will then discuss our misgivings with you at a future meeting. We informed Mr. Davis that we propose to have our own soils consultants, E. Q. Collier & Associates, review the available soils data at our own expense.

Briefly, our concern is based on the fact that water is most likely to be present in the future excavations. The soil is largely composed of stratified layers of silt and clayey silt. Our excavation extends some twelve feet below existing river bottom, and may be six feet as was the case with the King Street Structures. Your own soils people report that soils conditions at these sites are not comparable to conditions at our previous sites and hence construction procedures used on these other bridges are not necessarily adaptable to our needs.

We are satisfied that the proposed revisions are practical and will not adversely affect the stability of the structure or give the contractor an opportunity to claim for extra work. We will gladly make the necessary revisions. We understand that we will be compensated for the cost of this additional work on the basis of salary costs plus 10%.

Yours very truly,

MORRISON, HERSHFIELD, MILLMAN AND HUGGINS, LIMITED

J. T. Gregg, P.Eng.

410/62  
60-117

cc. Mr. B. Davis  
Mr. J. Selby

Bridge Division,  
Department of Highways, Ontario,  
Parliament Buildings,  
TORONTO 2, Ontario.

December 21, 1961.

Attn: Mr. F. I. Howson, P.Eng.  
Consultant Liaison Engineer

Re: Sheet Piling  
Humber River Bridge at Bolton  
WP 35-60

Gentlemen:

This letter is to place on record our discussion of yesterday between representatives of the D.H.O. Bridge Office, Mr. Selby of your Soils Group, and Messrs. Moore and Gregg of this company.

The meeting was called at the request of the Department following discussions with the local county engineers on construction procedures used on several local bridges just downstream of this structure.

Mr. Kleinstiber reported that on these bridges, which were built within the last few years, no steel sheet piling was used during the excavation for foundations nor was any provided for scour protection. Two of these structures were founded on spread footings and the third structure (King Street middle bridge) was supported on end leaving timber piles. These piles sank, virtually under their own weight, for 35 feet but held up at about 40 feet below the footings. No water problems had been reported during construction of these foundations and no special measures had been found necessary. In view of this, it was considered prudent to reconsider the necessity of sheeting on this proposed bridge for Highway #50.

Mr. Selby reported that the Department had put down additional bore holes at the site and had taken more frequent samples than had Racey taken during the original investigation. These confirmed the original analysis of the underlying soil which is a clay silt (80-85% silt 15 to 20% clay approx.) with intermittent layers of silt (varying from a few inches up to 30" thick). In addition, the presence of a water bearing gravel seam above footing elevation has been confirmed. However, Mr. Selby was of the opinion that the clay silt was more cohesive and less permeable than was previously thought and he doubted if the sheeting needed to be as extensive (i.e. deep) as shown on the drawings.

A general discussion followed on the various ways and means of constructing the footings with and without sheeting. No agreement on procedure was reached but the possibility of constructing the cofferdam by force account if it is required at all, is to be investigated and discussed with the D.H.O. Chief Engineer, Mr. W. A. Clarke.

Mr. Selby has not yet completed testing his samples and a copy of his report is to be forwarded when available to us for our consideration.

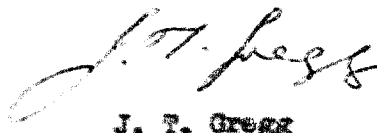
Mr. F. I. Hewson

- 2

To sum up our general impression at this time, we are of the opinion that the design as it stands is eminently suited to the site conditions. To abandon the sheeting completely is to take a gamble and the maximum possible saving involved is in the order of \$25,000.00 out of a total estimated cost approx. \$150,000.00. (An erroneous figure of \$40,000 has been talked of). If the sheeting as designed is not required for scour protection (to be checked with the Hydraulics Section) it is possible that we may be able to raise the tip elevation by a few feet. However, we are certainly willing to look into this matter again, taking recent developments into consideration.

Yours very truly,

MORRISON, HERSEFIELD, MILLMAN AND HUGGINS, LIMITED



J. T. Gregg

a

JTG:md

60-147

cc. B. R. Davis, D.H.O. Bridge Office ✓  
cc. Al McKim, D.H.O. Bridge Office  
cc. Ken Selby, D.H.O., Materials & Research Section

Mr. A. M. Toye,

January 16, 1962.

Bridge Engineer.

Materials and Research Division,

(Foundation Section)

Attention: Mr. S. McComble.

Re: Humber River Bridge at Bolton,  
Hwy. No. 50, District No. 6,  
W.P. 35-60.

The subsoil conditions for the above-mentioned structure were first investigated in June and July, 1960, by the soil consultant, Messrs. Racey, MacCallum & Associates, and a report containing factual information with recommendations for the footings was submitted. There were subsequent discussions between the Bridge Design Consultant and the representatives of this Section pertaining to the design and execution of the foundation work.

To clarify the question of piled foundations, two pile loading tests were carried out by this Section and the results reported and discussed. As a final result of all the above-mentioned investigations and discussions, spread footings with a safe bearing load of 3.0 T/sq.ft., placed 6 ft. below the new diverted creek bottom, were decided upon. Because of the proximity of free water and the required depth of footing excavations, danger of piping within the excavation was considered as possible, and interlocking steel sheet piling driven to about ten feet below the proposed excavation bottom was incorporated in the design.

After the design was completed and submitted for review, it was brought to our attention that the proposed and above-mentioned sheet piling would add about \$50,000 to the cost of the structure. In the light of this fact, the Foundation Section has carried out some additional field and laboratory work in order to explore the possibility of eliminating this costly construction procedure.

cont'd. /2 ...

At the site of the proposed structure on Hwy. 50, an additional borehole was drilled at the South-west corner of the proposed South abutment. Continuous sampling was carried out from a depth of 17.0' to 29.0'. The samples were carefully identified and classified. In the laboratory, Atterberg limits and comparative permeability tests were carried out.

The investigation has shown that from 14.0 ft. downward, the subsoil is stratified, consisting of layers of clayey silt, silt, and silty clay. The stratification is shown on the accompanying sketch.

Comparative permeability tests have shown different coefficients of permeability for the silt material - ( $K = 3.5 \times 10^{-4}$  in/sec) and the clayey silt material - ( $K = 1.26 \times 10^{-7}$  in/sec). These values cannot be taken as absolute, but rather as indications of permeability differences.

Two boreholes were also drilled, one at each of the two bridges located on King Street's crossings of the Humber River. These structures were built on spread footings, supposedly some 6 ft. below river bottom and during construction, neither dewatering difficulties nor piping were encountered. If the soil conditions at these sites were found to be comparable with those at the site of the proposed structure on Hwy. 50, the extrapolation of experience would be justified. However, the soil conditions were not found to be comparable, the subsoil here, being clayey silt of very to extremely high density, 'N' values being in a range of 40 to 160 blows/ft.

The above-mentioned facts and newly-gathered evidence point to the choice of sheet piling as incorporated in the design as a good and safe engineering solution. However, the stratified character of the subsoil, the difference in permeability of different layers are valid evidence that no homogeneity exists and therefore, deviations from ideal conditions for piping exist. It is practically impossible to incorporate these factors into any kind of computation or analyses, and only field trial sections could provide a reliable and true answer.

It is therefore our recommendation that work at the bridge site be started without steel sheeting and this be used only if field evidence proves it necessary. It is recommended that first the new river bed at the bridge site be excavated to its final elevation. If this is done while the water in the river is controlled and kept as low as possible (which, we understand, is possible), dewatering of the excavation is not expected to present any difficulty. After this stage is completed, excavation with simple dewatering of one of the wing walls should be commenced.



- 3 -

Here, evidence will be gathered if sheeting will or will not be necessary. In case that this proposal is accepted, it is suggested that the Foundation Section be advised of the commencement of construction in order that assistance, guidance and advice could be given to the District's supervisory personnel.

We believe that the above information will enable you to arrive at the final conclusions and also make the final decisions. However, if there are any additional questions that you would like to discuss, please feel free to call on our Office.

A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.  
Per:

KGS/MdeF  
Attach.

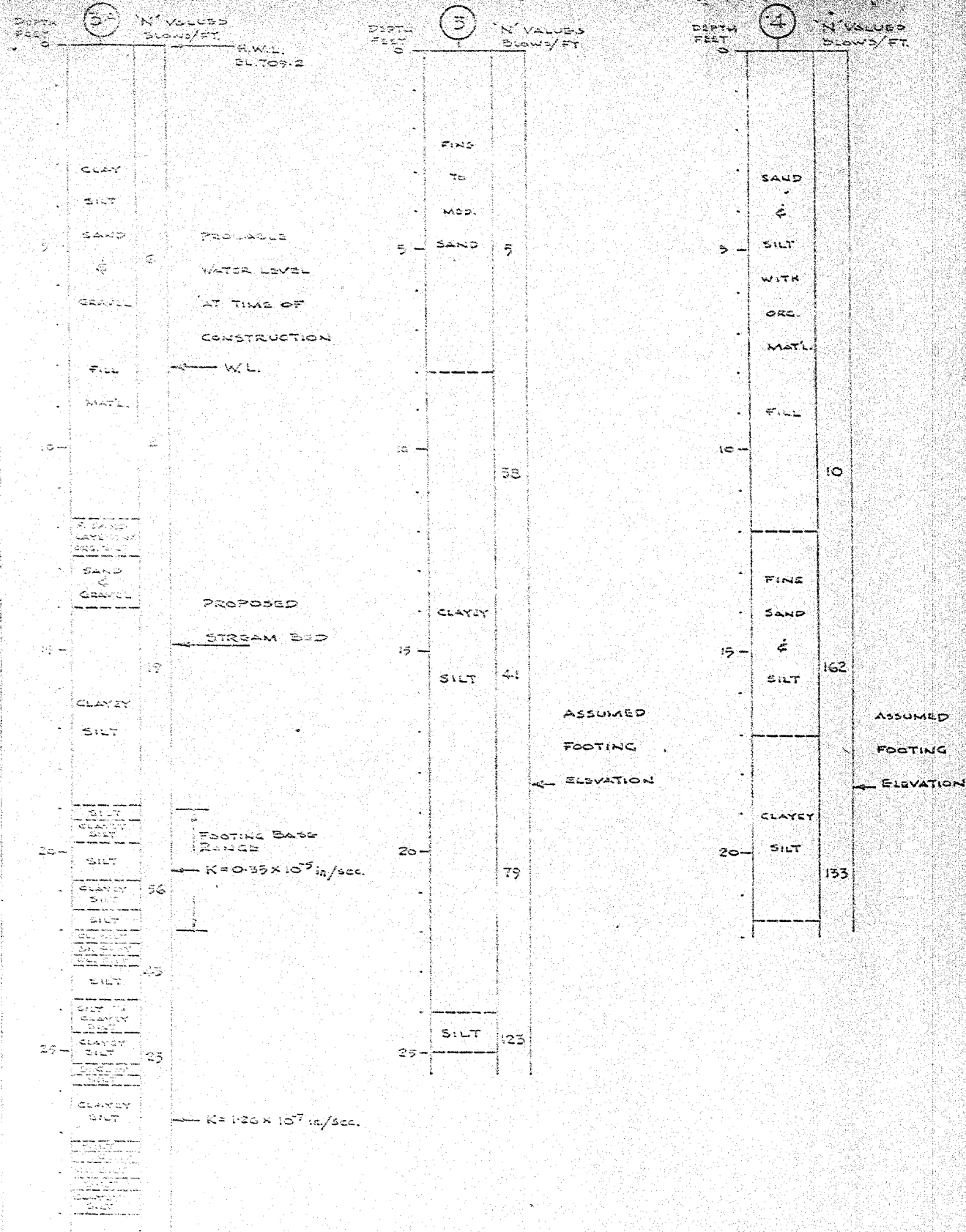
  
(K. G. Selby,  
SR. PROJECT FOUNDATION ENGR.)

cc: Foundations Office  
Gen. Files.

# PROJ. HWY. 50 BRIDGE

# KING ST. BRIDGE NO. 1

# KING ST. BRIDGE NO. 2



ORIGINATED BY: K. SELBY  
 DRAWN BY: R. D. REED  
 CHECKED BY: [blank]  
 APPROVED BY: [blank]  
 DATE: 16 JAN. 1962

DEPARTMENT OF HIGHWAYS - ONTARIO  
 MATERIALS & RESEARCH SECTION  
 COMPARISON OF SUB SOIL  
 CONDITIONS AT THREE SITES  
 IN BOLTON

SCALE: 3 INCHES = 1 FOOT  
 W. P. NO. 33-60  
 JOB NO. 61-F-116  
 DWG. NO. 61-F-116 A

Mr. A. M. Toye,  
Bridge Engineer.

Materials & Research Section.

July 12, 1960.

FOUNDATION INVESTIGATION REPORT

by: Eacey, MacCallum & Associates,  
Ltd.

Attention: Mr. S. McCombie.

Re: Proposed Crossing of Humber River and  
Hwy. No. 50 at Bolton, Ont., Dist. 6.  
W.P. 35-60.

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Attached, we are forwarding to you, the above mentioned report submitted by the Consultants, Eacey, MacCallum and Associates, Ltd. We have reviewed the presented factual data and believe that the given conclusions and recommendations will be adequate for your future design work.

Should there be any other questions or problems in connection with the above mentioned structure that you would like to discuss, please feel free to call on our Office.

AS/MdeF  
Attach.

cc: Messrs. A. M. Toye (2)  
E. A. Fregaskes  
D. C. Ramsay  
I. Campbell  
C. Fraser  
T. J. Kovich  
A. Watt

L. G. Soderman,  
PRINCIPAL FOUNDATIONS ENGINEER.  
Per:

(A. Stermac,  
FOUNDATIONS OFFICE ENGINEER.)

Foundations Office  
Gen. Files.

RM 110

Department of Highways  
COPY  
For the Information of

Mr. A. G. Stermac  
Principal Foundation Eng.  
Room 107, Lab. Bldg.  
DOWNSVIEW, Ontario.

Mr. B. R. Davis

Bridge Design Engineer

Mr. F. I. Hewson

March 30, 1962

W.P. 35-60

Humber River Bridge at Bolton  
Hwy. 50 District 5

The expense of the foundations proposed for this bridge has been criticized so further studies have been made. The trouble has been caused by the depth of the excavations below river level (11 feet) and the nature of the soil (a fine grained silt with seams of clay and sand).

While it is possible that these excavations could be carried out by ordinary means, our foundation engineers and their consultants feel that it is quite likely the material would "pipe" or become quick. If this did occur construction would be delayed and extras and claims could be expected on the contract.

Foundation experts agree that this can be prevented by driving sheet piling to 10 feet below the excavation. They expect well-points and chemical grout would be ineffective in such a fine grained soil. Excavation under water and a tremie foundation would be possible but our experience with tremie concrete has not been favourable and the saving would be small. Piled foundations would resist scour but the silt would liquify during driving creating construction problems.

Since sheet piling is required anyway, it was decided to leave it in place where it might serve as scour protection. Although the ends of the wingwalls could be founded at a higher elevation, the stepped footing would cost almost as much as the deeper foundations so this is not being changed.

It is estimated that these extra protections increase the cost by less than 20% and the added assurance of ease of construction and permanence are worth this.

*F. I. Hewson*

F. I. Hewson,  
Consultant Liaison Engineer.

FIH/et

cc. A.G. Stermac  
cc. S.T. Gregg  
cc. A.E. McKim

Notes of Meeting - Humber River Bridge at Bolton - D.H.C. W.P. 35-60

Date: March 21, 1962

Place: Office of Morrison, Hershfield, Millman & Huggins, Ltd.  
96 Bloor Street West, Toronto

Present for:

Dept. of Highways, Ontario - Mr. H. F. Gilbert  
Mr. A. Sternac  
Mr. K. Selby  
Mr. F. I. Hewson  
Mr. A. McKim

Morrison, Hershfield, Millman & Huggins, Ltd. - J. T. Greig

Purpose of Meeting

To review the design, estimate and construction methods for the above structure.

History

The bridge, as designed, has assumed that sheet piling will be necessary in order to excavate for the footings, some of which to remain in place, to provide against scour.

Since originally designed, additional soils information indicates that soil conditions may be more favourable than assumed for design. In addition, the D.H.C. Hydraulic Engineers are now of the opinion that sheeting is not required for scour protection.

It has been estimated (questioned?) that a saving of approximately \$50,000 could be <sup>a</sup>made if sheeting were abandoned completely. The consultants have contended that this is too high a figure and that the actual saving would be more in the order of \$20,000 to \$25,000. Mr. Sternac has stated that for a saving of \$50,000 it would be worthwhile taking a chance in eliminating the sheeting but not if the saving were in the order of only \$20,000. To eliminate the sheeting might give the contractor a good opportunity to claim an extra if the conditions proved to be other than ideal, and there is a good chance that this may be the case.

Recent weather conditions and loading restrictions on Hwy. #50 have prevented the Dept. from digging a test hole at the site.

Discussion

Some preliminary discussion took place as to the method of constructing the sub-structure without the use of sheet piling. Mr. Sternac proposed that the initial excavation be made down to El. 701 approx., the sides of the excavation should stand at 1 : 1. The final excavation from El. 701 to approx. El. 690 would then proceed under water if necessary (and most likely) using a side slope of  $1\frac{1}{2}$  : 1. A drag line may be required for this work and it may be necessary to provide a 10 ft. min. berm for equipment and working space at El. 701. Without actually making any calculations it was agreed that this procedure involved at least twice the amount of excavation. It may also be necessary to place a 3 ft. loose tremie seal over the bottom of the excavation so that dewatering can be carried out.

The meeting then considered the relative costs of the various proposals which are as follows:

(1) D.H.O. EA Estimate

Item No.	Item	Quantity	Unit Price	Total
1	Earth Excavation	1,705 c.y.	3.00	5,115
4	Supply Equipment		L.S.	1,500
5	Supply Sheet Piling		L.S.	26,000
6	Construct Cofferdam		L.S.	10,000
7	Excavate inside Cofferdam		L.S.	2,000
8	Dewater Foundations		L.S.	6,000
				<u>\$ 52,615</u>

(2) Consultant's EA Estimate (original)

1	Supply Piling Equipment		L.S.	10,000
2	Gen. Ex. to 702	1,090 c.y.	2.50	2,725
3	Supply & Drive Perm. Sheetpiling	4,870 s.f.	2.95	14,350
4	Earth Ex. for footings	1,705	5.00	8,525
5	Dewater Cofferdams (incl. temporary sheetpiling)		L.S.	10,000
				<u>\$ 45,600</u>

(3) Revised Estimate for Open Ex. as discussed

1	General Excavation	3,410 c.y.	3.00	10,230
2	Ex. below 701		L.S.	4,000
3	Dewater Excavation		L.S.	6,000
4	3 ft. Trans. Conc. ?	380 c.y.	20.00	7,600
				<u>\$ 27,830</u>

.....

(4) Revised Consultant's Estimate

N.B. This assumes that all sheeting is temporary and therefore new steel not required. Sheeting may be hired but for estimate we have considered new sheeting and then salvage value is deducted. It should also be assumed that the contractor only needs to work on one footing at a time and need only get a total of 7,500 sq. ft. of sheeting, i.e. enough for one cofferdam plus extra for damaged sections. Extent of sheeting may also be further reduced if initial excavation is taken below 702 (not recommended).

Item No.	Item	Quantity	Unit Price	Total
1	Supply Piling Equipment		L.S.	1,500.00
2	Gen. Ex. to 702	1,090 c.y.	2.50	2,725.00
3	Supply Temporary Sheeting	7,500 s.f.	1.75	13,125.00
4	Drive & Recover Temp. Sheeting	13,644 s.f.	0.80	10,915.20
5	Ex. for Footings (inside cofferdam) (bracing incl.)	1,705 c.y.	3.00	5,115.00
6	Dewater Cofferdams	2	1,000.00	<u>2,000.00</u>
				<u>\$-35,380.20</u>

Conclusions and Recommendations

It was agreed that estimates (3) and (4) should be further studied. From above figures it would appear that the maximum saving in abandoning sheeting is approximately \$13,100.

If the sheeting can be hired from a piling contractor or if it becomes necessary to place trends in an open excavation the apparent difference between (3) and (4) could be even less than the above figure. However, we feel safe in saying that the actual difference in price between open excavation and a sheeted excavation will certainly be less than \$25,000.

Soils Report

Copies of a letter to the consultants from their soils consultant, H. Q. Golder and Associates, were given to Mr. Starnac and Mr. Rowson. The letter is a summary of the various soils investigations which have been made to February 8, 1962.

*J. T. Gregg*  
J. T. Gregg

60-117

cc. All Present  
Mr. B. Davis  
Mr. V. Milligan

23-62-70  
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.

GEORGE L. HOUGHTON, A.M.I.MECH.E., M.E.I.C., P.ENG.

TORONTO DIVISION  
27 CARLTON STREET

Reference: S-500/T-2335  
- Report -

12th July, 1960

Department of Highways for Ontario,  
Materials and Research Section,  
C/o Parliament Buildings,  
TORONTO - Ontario.

Attention: Mr. K. Peaker.

RE: FOUNDATION INVESTIGATION  
FOR PROPOSED CROSSING OF  
HUMBER RIVER AND HIGHWAY  
NO 50 AT BOLTON, ONTARIO.  
W.P. 35 - 60.

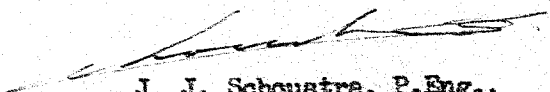
Dear Sirs,

The enclosed report presents the results of our  
foundation investigation at the above location.

We hope the report is satisfactory to you; if you have  
any questions about it please do not hesitate to get in touch with  
us.

Thank you for this opportunity of being of service to  
you.

Yours very truly,  
RACEY, MACCALLUM AND ASSOCIATES LIMITED,

  
J. J. Schoustra, P.Eng.,  
Divisional Soil Engineer.

JJS/YDP

EM 6-5641



Department of Highways for Ontario,  
Materials and Research Section,  
C/o Parliament Buildings,  
Toronto - Ontario.

FOUNDATION INVESTIGATION  
FOR PROPOSED CROSSING OF  
HUMBER RIVER AND HIGHWAY  
NO 50 AT BOLTON, ONTARIO.  
W.P. 35 - 60.

Reference: S-500/T-2335  
- Report -

Racey, MacCallum and Associates  
Limited.

12th July, 1960.

# 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.

GEORGE L. HOUGHTON, A.M.I.MECH.E., M.E.I.C., P.ENG.

TORONTO DIVISION  
27 CARLTON STREET

Reference: S-500/T-2335  
- Report -

12th July, 1960

**FOUNDATION INVESTIGATION  
FOR PROPOSED CROSSING OF  
HUMBER RIVER AND HIGHWAY  
NO 50 AT BOLTON, ONTARIO.  
W.P. 35 - 60.**

**INTRODUCTION :**

The field investigation of the above site was carried out from 13th to 23rd June, 1960. A detailed description of the field work and results, together with foundation recommendations, is given in the following paragraphs.

**SCOPE OF FIELD WORK AND LABORATORY WORK :**

Boring was started with a continuous flight power auger which was later replaced as the boreholes were filling-in at certain depths. In Borehole No 2 the auger was successful down to approximate depth of 79 feet, whereas in the other three boreholes it went down only to approximately 30 to 40 foot depths before being stopped by fill-in. A standard diamond drill rig was then used to advance Borehole No 2 by driving BX casing and washing out. Sampling was done using a 2-inch outside diameter split spoon. A dynamic cone penetration test was carried out adjacent to Borehole No 1 and Borehole No 2. The driving energy for both the split spoon and cone penetrations was a 140 lb hammer at 30 inches drop.

As the standard penetration resistance recorded was quite low, it was felt that some static penetration tests were necessary. These were carried out by the Dutch Deep Sounding Apparatus, using a 60-degree cone of 1.4 inch base diameter.

The locations of the borings and probings are shown on Enclosure No 1, and the results on Enclosures No 2 to 5.

On arriving at the laboratory some samples were subjected to grain-size distribution tests and moisture tests. The results

Reference: S-500/T-2335  
- Report - Continued.

12th July, 1960

are shown on Enclosures No 6, 7, 8 and 9, and on the Engineering Data sheets. A subsoil profile has been prepared, see Enclosure No 10.

SUBSOIL CONDITIONS :

Under the embankment fill the original subsoil profile consists mainly of water-lain silt with some traces of very fine sands and silty-clays. The silty-clay occurs only occasionally as a very thin clay "varve" varying in thickness from a fraction of an inch to approximately one inch. At greater depths the silt becomes coarser and is finely layered with fine sands. The silt deposit extends down to the maximum depth of 111.5 feet reached in Borehole No 2.

In some instances some minor lenses of gravel and organic silt were found to exist between the overlying fill and the original subsoil. These occur at approximately river level.

The ground water level is at approximately the same level as the river, or at Elevation 700.0 feet.

DISCUSSION OF RESULTS :

As mentioned previously, the dynamic penetration readings were quite low, which would indicate that the silt was only of medium density. The static penetration readings, however, revealed the material to be much more dense. This may be explained by the fact that in a reasonably dense submerged silt the dynamic test has a tendency to loosen or "liquefy" the material with the shock from the hammer blows. This phenomenon is aggravated for the split spoon as the result of water seepage from the bottom of the hole. The static test, however, does not have this tendency and therefore may safely be assumed as giving more correct results. The static cone tests could not be continued to greater depth because the friction on the casing presented a resistance in excess of the capacity of the machine.

The river invert elevation is approximately 697 to 698 feet. If the standard D.H.O. scour allowance of 8 feet should be used, the founding elevation would be approximately 690 feet. On the basis of the static penetration test the allowable soil bearing capacity for footings at this elevation may be taken as 8000 paf.

As the footing excavation will be below the water table there will definitely be problems of seepage and "quick" conditions. Therefore, a well-point system should be installed prior to excavation. The excavation sides either should be cut back to at least a 1 : 1 slope

Reference: S-500/T-2335  
- Report - Continued.

12th July, 1960

or be sheeted and braced. The former solution is probably the more economical.

An alternative solution would be a foundation on piles driven some distance into the dense silt. A displacement-type pile, driven to approximately Elevation 780 - 785 feet would have an ultimate end-bearing capacity of the order of 150 tons per square foot of tip area. Some additional frictional resistance would certainly develop along the bottom 10 feet of the shaft, averaging at least 1 taf. Hence, the ultimate bearing capacity of, for instance, 10 inch diameter tube piles driven 10 feet into the dense silt would be about 110 tons. Normally a factor of safety of 2.0 is considered adequate if applied to the results of static penetration tests, hence the safe bearing capacity of such piles would be of the order of 55 tons. The actual set during driving might not indicate such a high capacity; in this type of soil the set is no reliable measure except when time is allowed for "freeze up".

Steel H-piles driven to refusal into the silt could also be used of course. Judging from the static and dynamic cone penetration results such piles would have to penetrate deeper than displacement piles, possibly down to Elevation 750 - 760 feet. It is not felt, however, that this type would be ideally suited as no sudden increase in density does prevail.

The silty soil found at or close to river level is not expected to present a stability or settlement problem for embankment fills. Because of the expected liquefaction phenomenon during pile driving it may be advisable, however, to complete all pile driving prior to excavating the new river bed.

CONCLUSIONS :

1. The subsoil to a depth of at least 110 feet consists mainly of dense grey silt with some fine sand and traces of silty clay.
2. A footing foundation (allowing for the usual 8 feet of scour protection depth) at approximately Elevation 690 feet could safely support 8,000 psf. Very careful excavation drainage measures would be required to prevent deterioration of the soil bearing properties.
3. A more attractive solution would seem to be a foundation on end-bearing piles to approximate

/ Continued :

Reference: S-500/T-2335  
- Report - Continued.

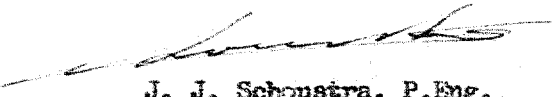
12th July, 1960

CONCLUSIONS :

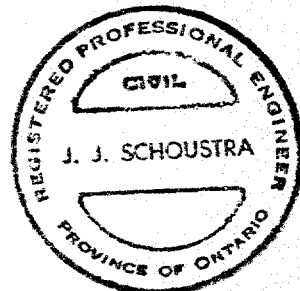
3. Continued -

Elevation 685 - 680 feet. Safe bearing capacities of up to 75 taf in end-bearing and 0.5 taf in frictional bearing may be allowed. Hence for a 10-inch diameter tubular closed-end pile the bearing capacity would be 55 tons.

4. No stability or settlement problems are anticipated for embankment fills.
5. Excavation for the new river channel should preferably be done after pile driving, to prevent minor stability problems as the result of temporary soil liquefaction.

  
J. J. Schoustra, P.Eng.,  
Divisional Soil Engineer.

JJS/YDP

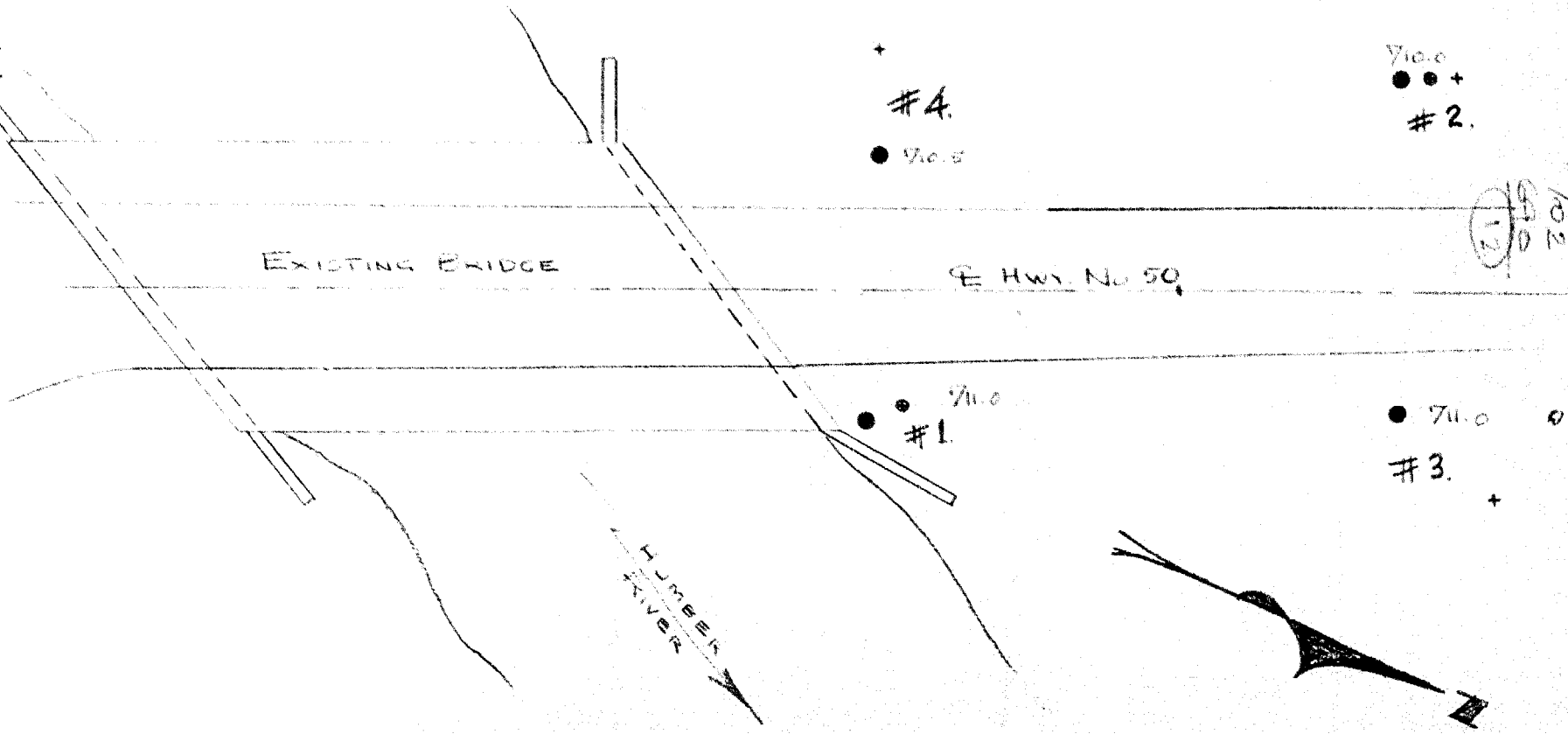


# LOCATION PLAN OF BORINGS & DYNAMIC & STATIC CONE PENETRATION TESTS — HUMBER RIVER BRIDGE HWY NO. 50, EOLTON, ONTARIO

• SCALE: 1" = 20'

- LEGEND: ● BORINGS  
⊕ DYNAMIC CONE PENETRATION TEST  
+ STATIC " " "

Racey, MacCallum & Associates Ltd.  
EOLTON



THE EASTERN...  
HAY AND...  
...  
...  
...



## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: HUNTER RIVER BRIDGE.  
 Location: HIGHWAY #50, BOLTON, ONTARIO.  
 Hole Location: See Enclosure No 1.  
 Hole Elevation and Datum: 711.0 feet M.S.L.  
 Field Supervisor: M.I.R. Prep: L.P.W.  
 Driller: H.G. Checked: J.S. Date:

## LEGEND

Shear Strength (C)

Unconfined compression

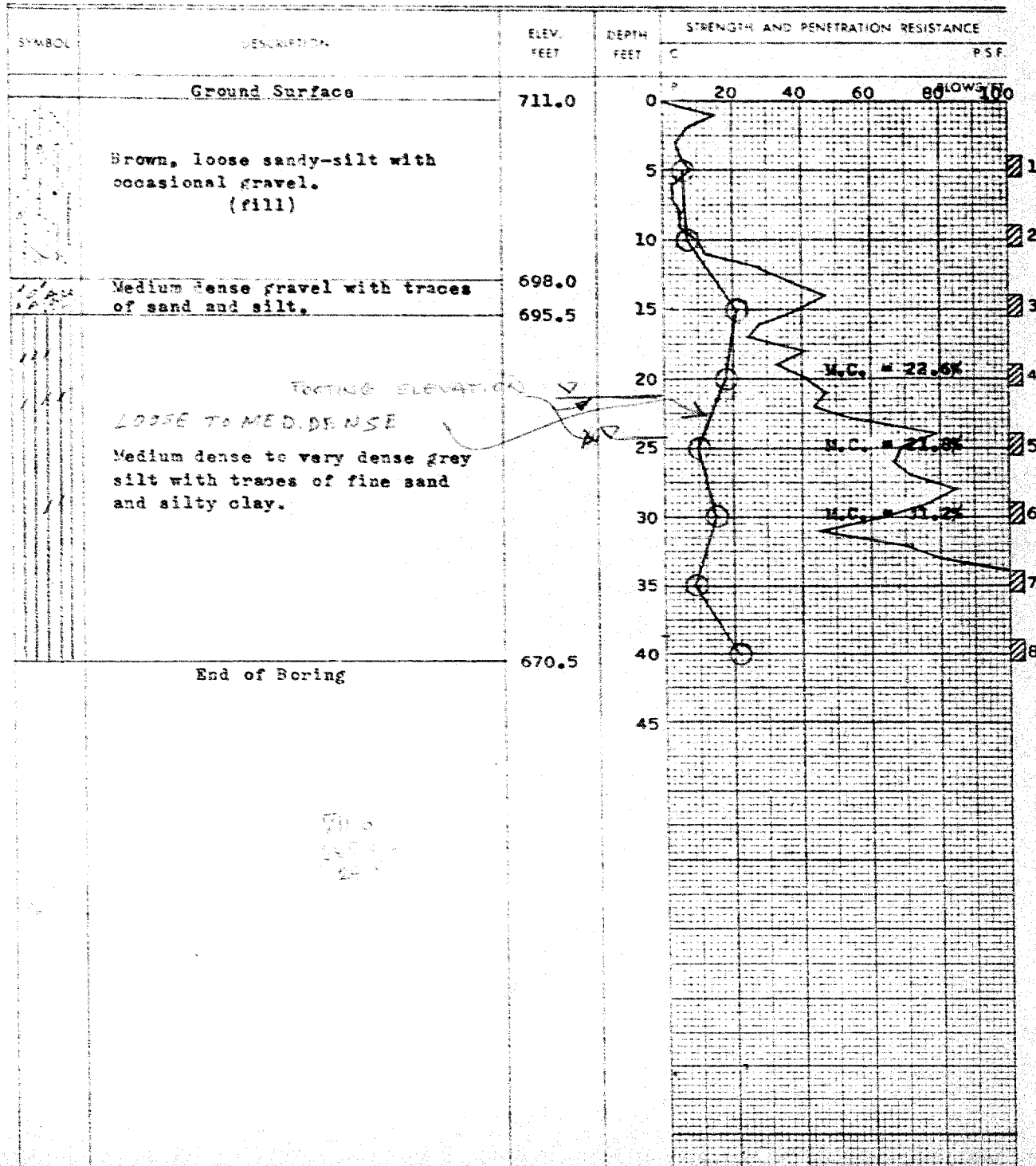
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing

⊕  
43⊕  
⊕  
⊕



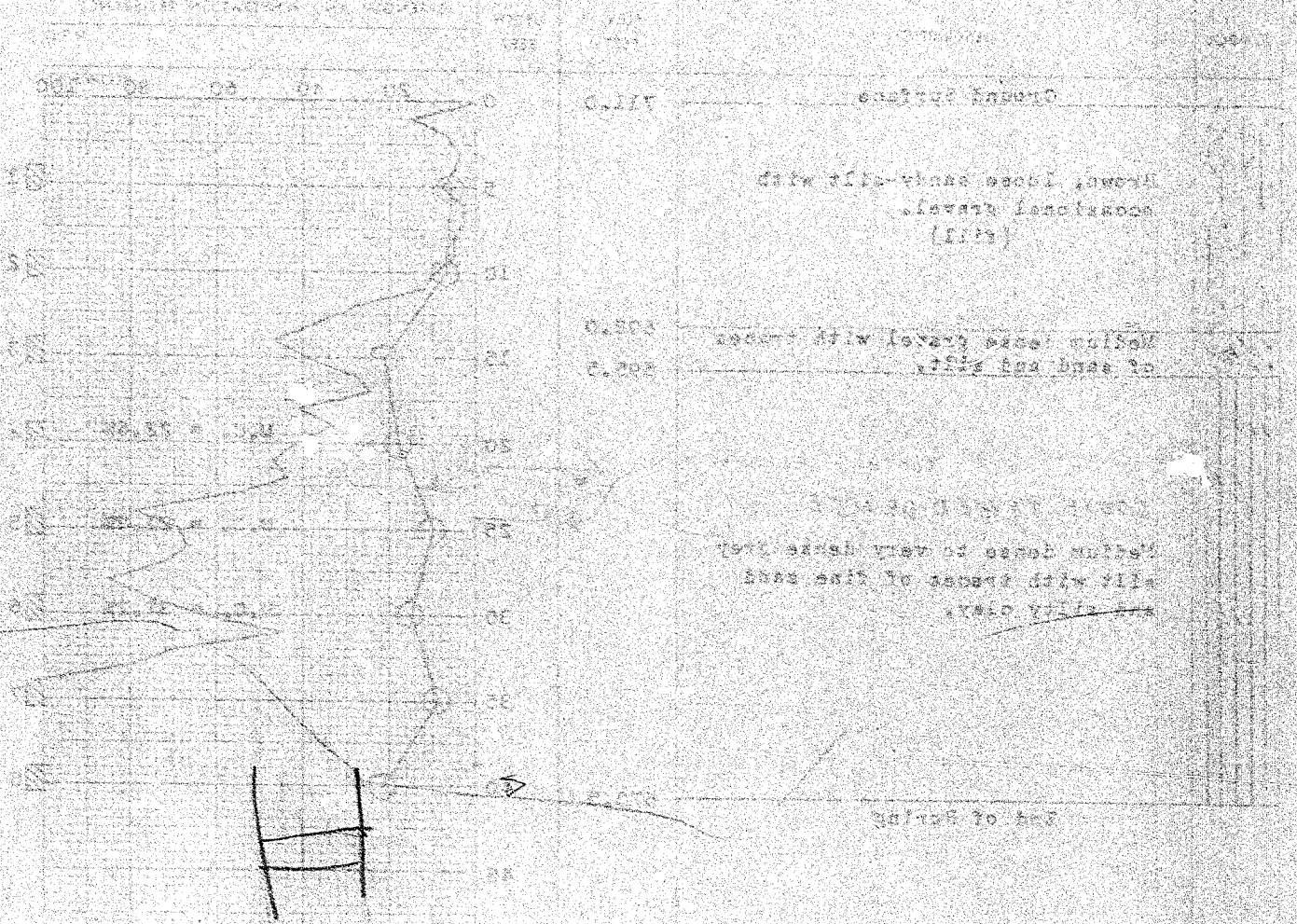
# RACEY McCALLUM AND ASSOCIATES LTD.

Geotechnical Engineering Division

For: [illegible] Street [illegible] [illegible]

Date:

Project: [illegible]  
 Location: [illegible]  
 Notes: [illegible]  
 Date: [illegible]



## RACEY MacCALLUM AND ASSOCIATES LTD.

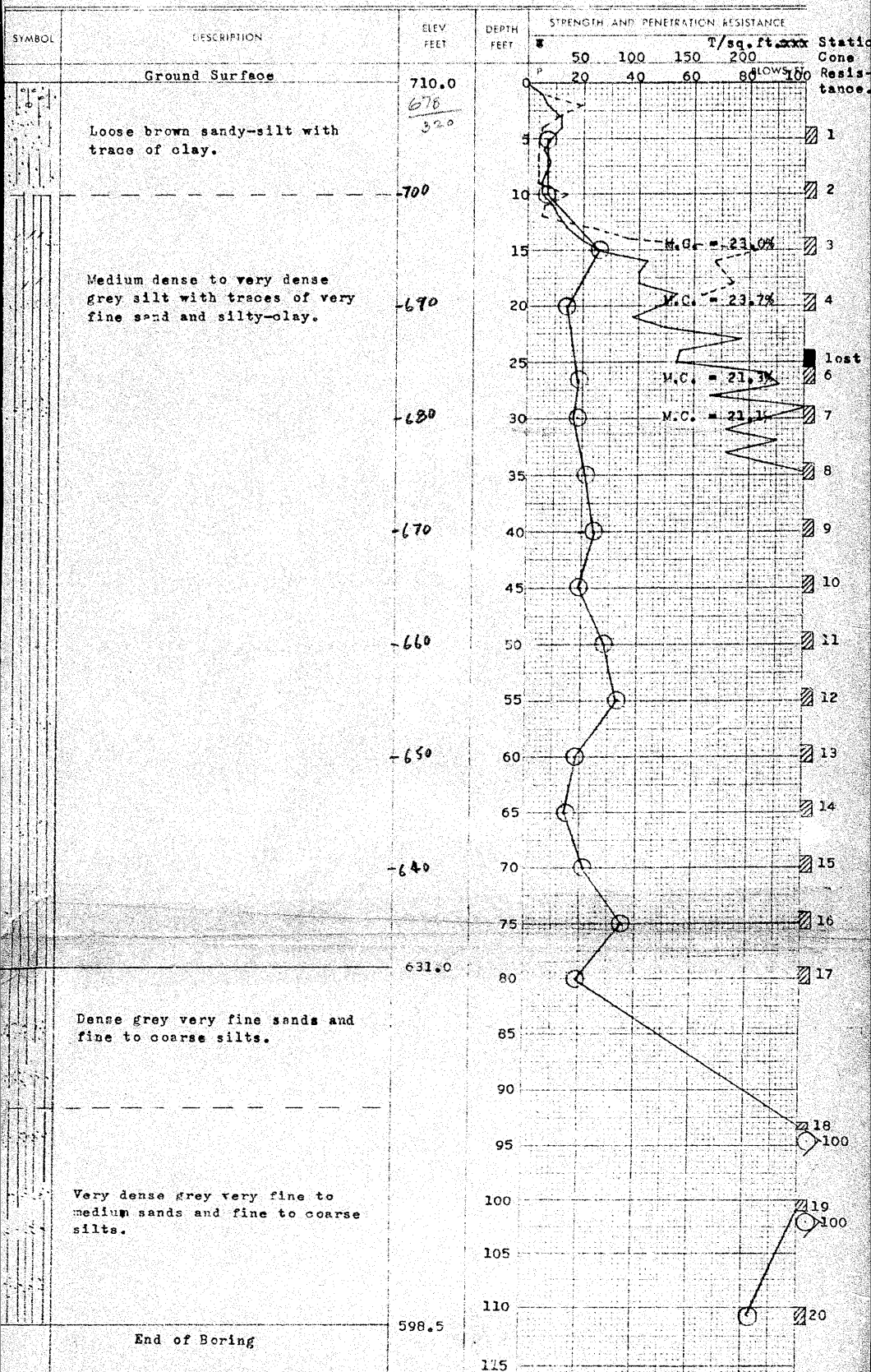
Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

Project: HUMBER RIVER BRIDGE.  
 Location: HIGHWAY #50, BOSTON, ONTARIO.  
 Hole Location: See Enclosure No 1.  
 Hole Elevation and Datum: 710.0 feet M.S.L.  
 Field Supervisor: M.I.B. Prep.: L.P.W.  
 Driller: H.G. Checked: J.S. Date:

## LEGEND

Shear Strength  $C$   
 Unconfined compression  
 Vane test and sensitivity  $S_u$   
 Penetration Resistance  $P$   
 2" Split tube  
 2" Dia. Cone  
~~Static~~ Static Cone Test



**RACEY MacCALLUM AND ASSOCIATES LTD.**

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

Project: **WINNIPIC RIVER BRIDGE.**  
 Location: **HIGHWAY #50, BOLTON, ONTARIO.**  
 Hole Location: **See Enclosure No 1.**  
 Hole Elevation and Datum: **711.0 feet N.S.L.**  
 Field Supervisor: **L.P.W. Prep: L.P.W.**  
 Driller: **H.G. Checked: J.S. Date:**

**LEGEND**

Shear Strength (C)

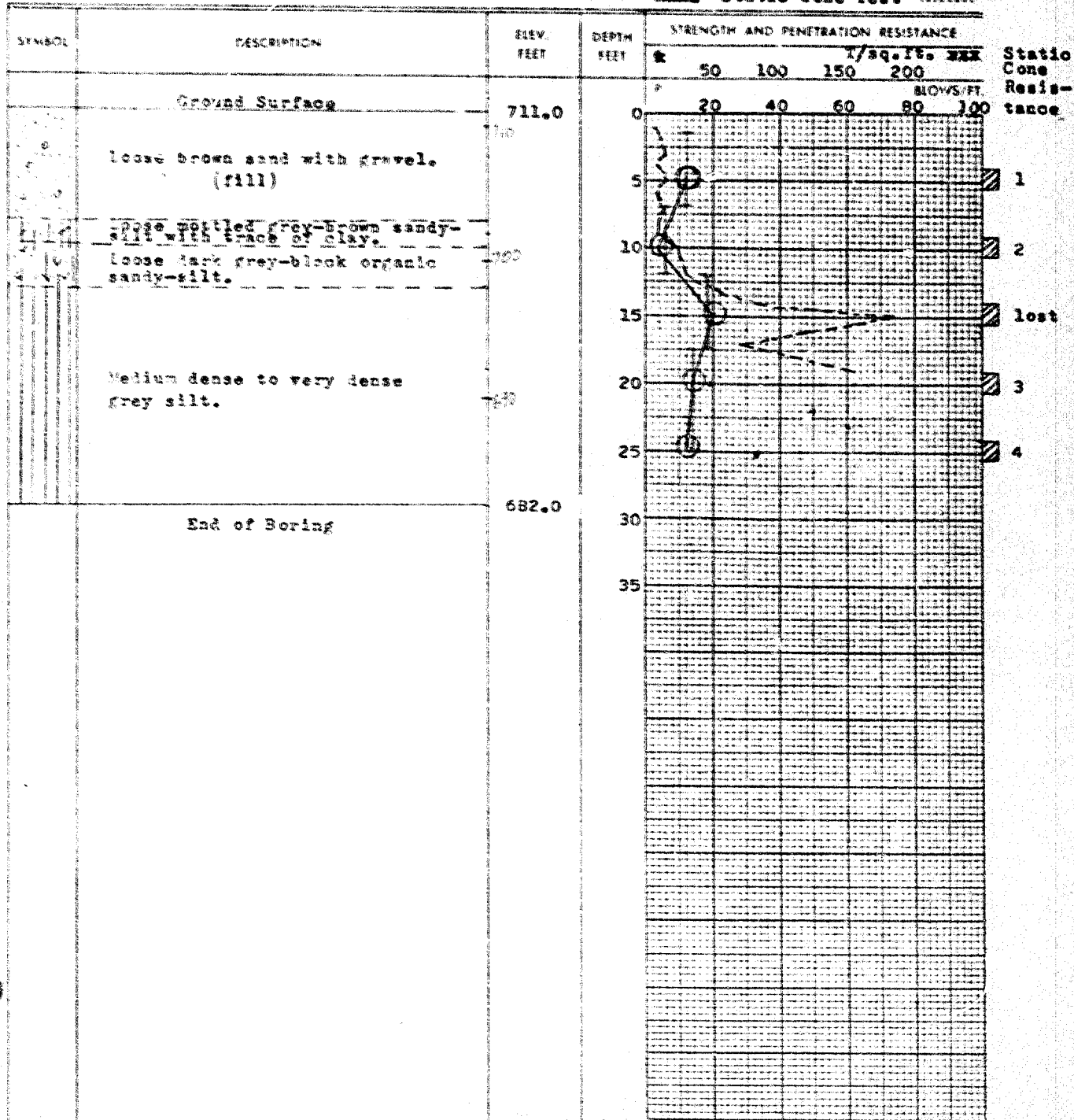
Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (B)

2" Split tube

2" Dia. Cone

**Static Cone Test**

## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 4

Project: HUNGER RIVER BRIDGE.  
 Location: HIGHWAY #50, BOLTON, ONTARIO.  
 Hole Location: See Enclosure No 1.  
 Hole Elevation and Datum: 710.5 feet  
 Field Supervisor: L.P.X. Prep: L.P.W.  
 Driller: H.G. Checked: J.S. Date:

## LEGEND

Shear Strength (C)

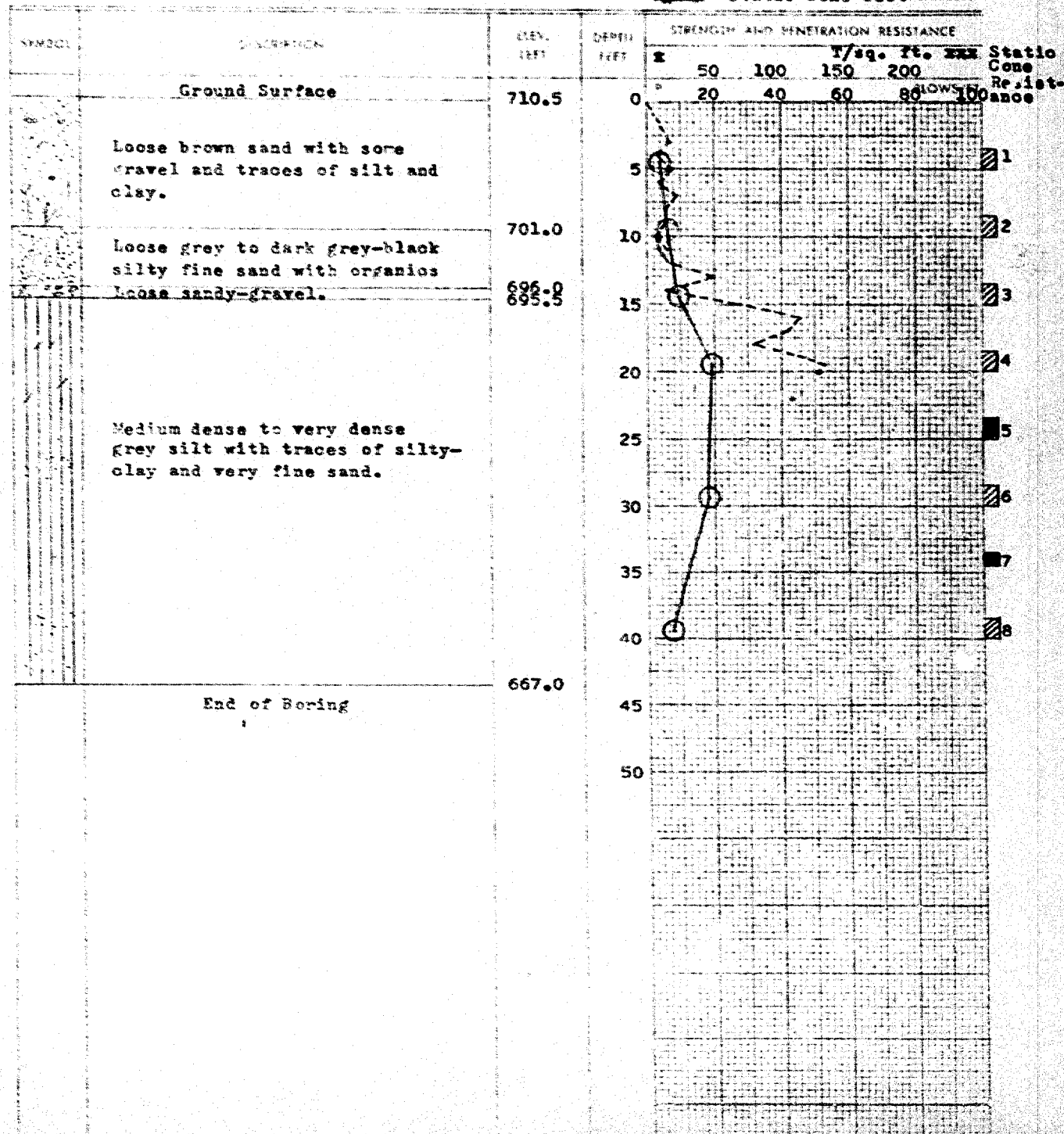
Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

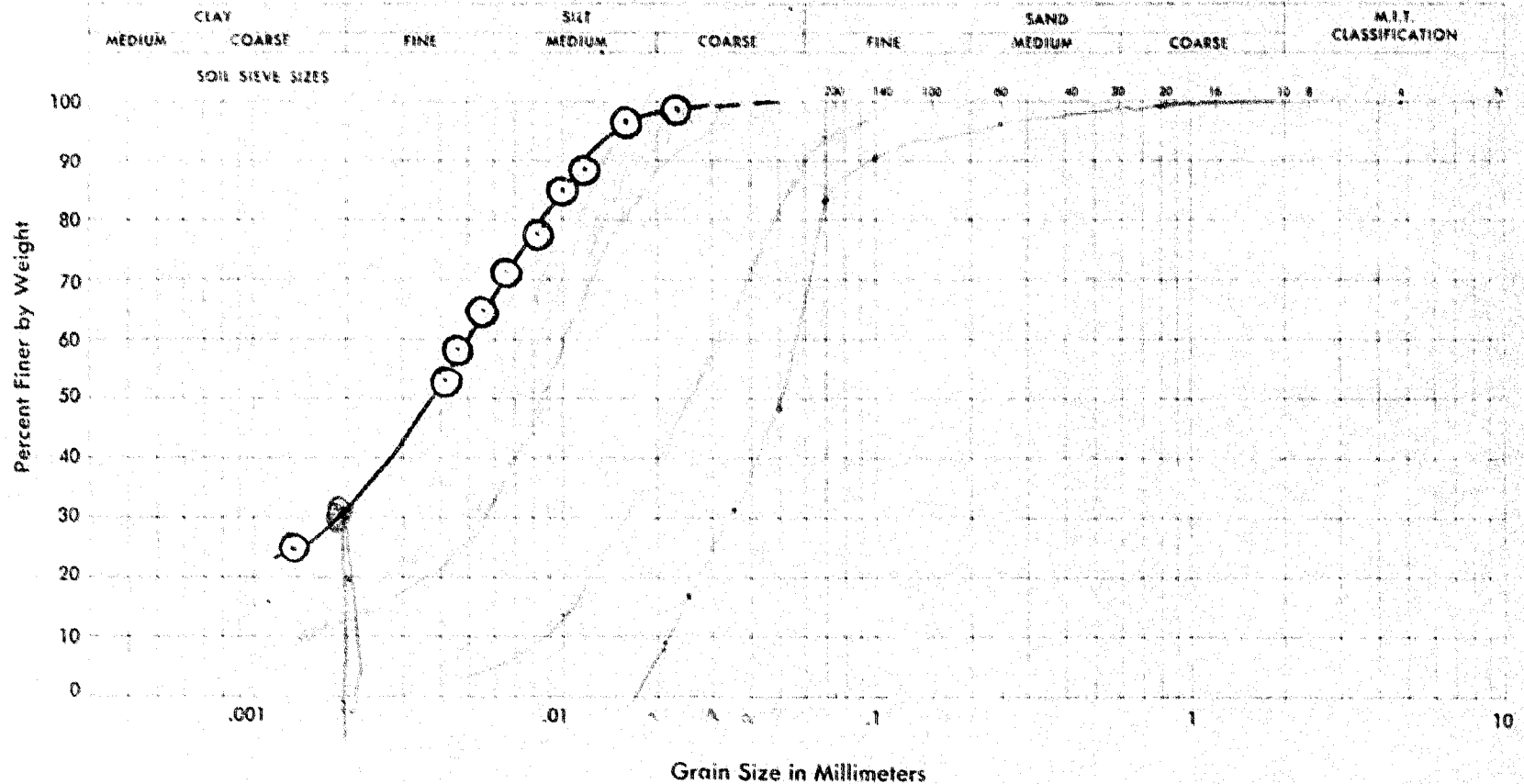
1" Split tube

2" Dia. Tube

~~Static Cone Test~~ Static Cone Test

# RACEY MacCALLUM AND ASSOCIATES LTD.

## GRAIN SIZE DISTRIBUTION



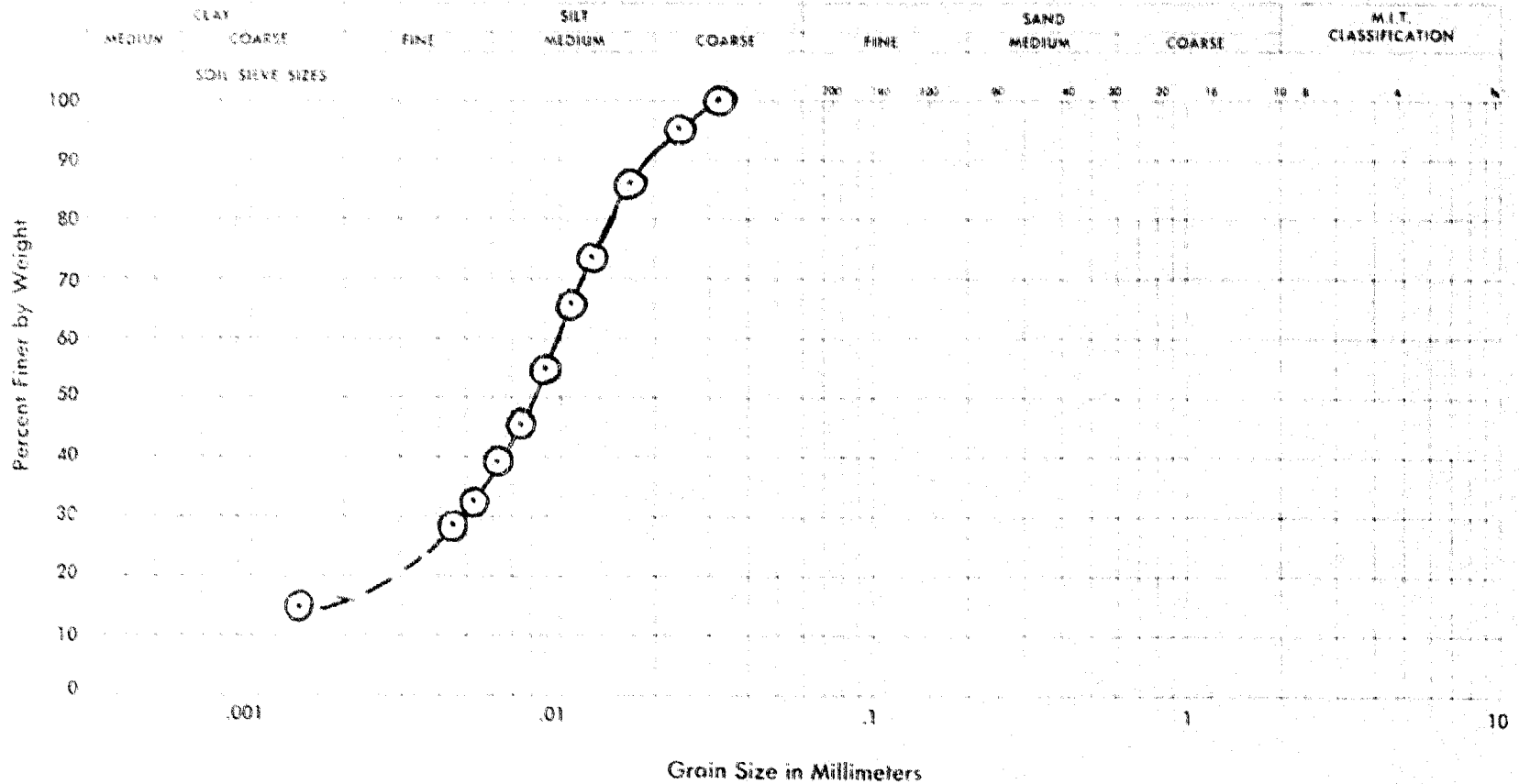
Project HUNTER RIVER BRIDGE, BOLTON, ONTARIO.

Legend BH #1, Sample No. 4.

Order No. S-500/T-2335

# RACEY MacCALLUM AND ASSOCIATES LTD.

## GRAIN SIZE DISTRIBUTION



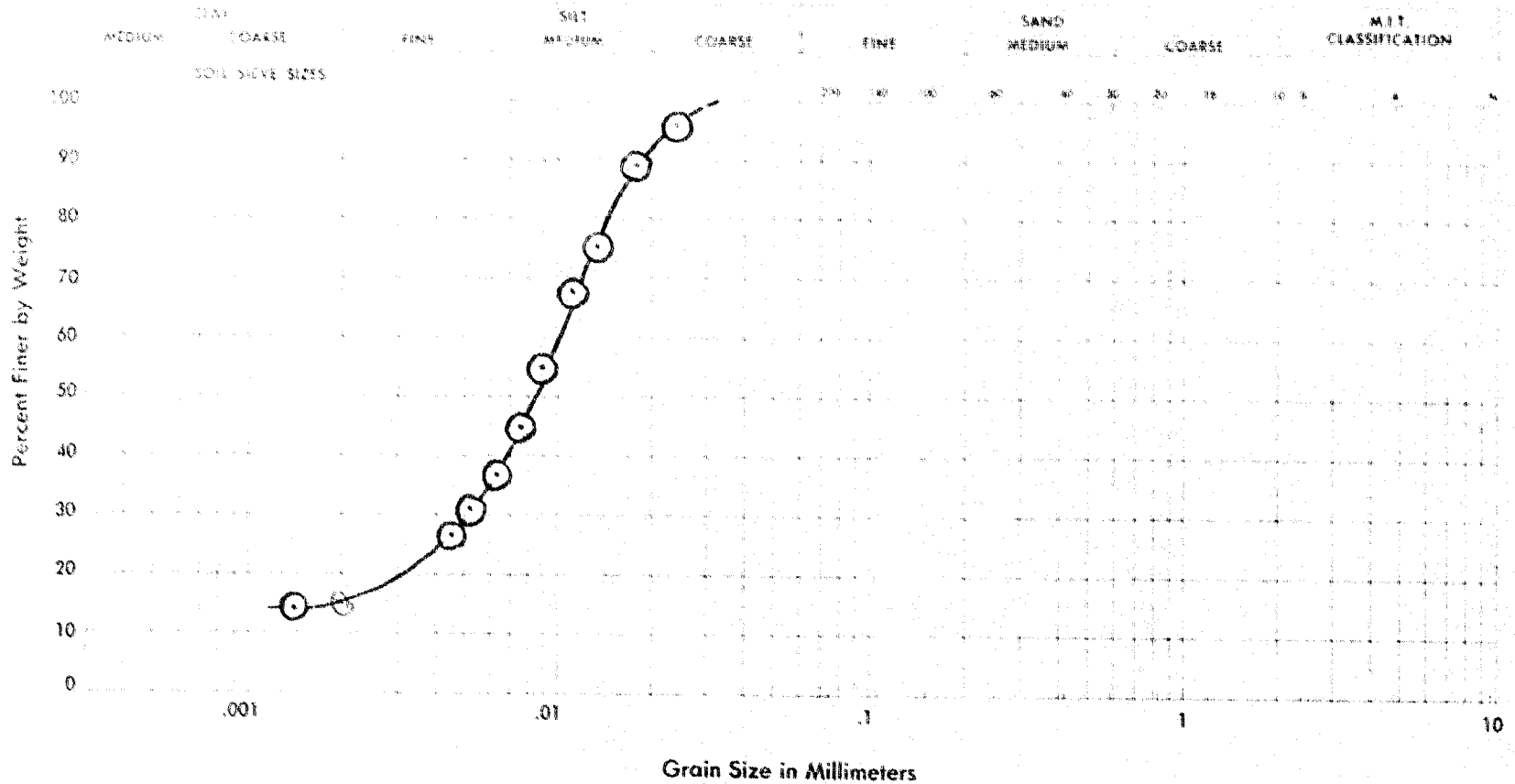
Project HUMBER RIVER BRIDGE, BOLTON, ONTARIO,

Order No. S-500/T-2335

Legend #2, Sample No. 4.

# RACEY MacCALLUM AND ASSOCIATES LTD.

## GRAIN SIZE DISTRIBUTION



Project HUNTER RIVER BRIDGE, BOLTON, ONTARIO.

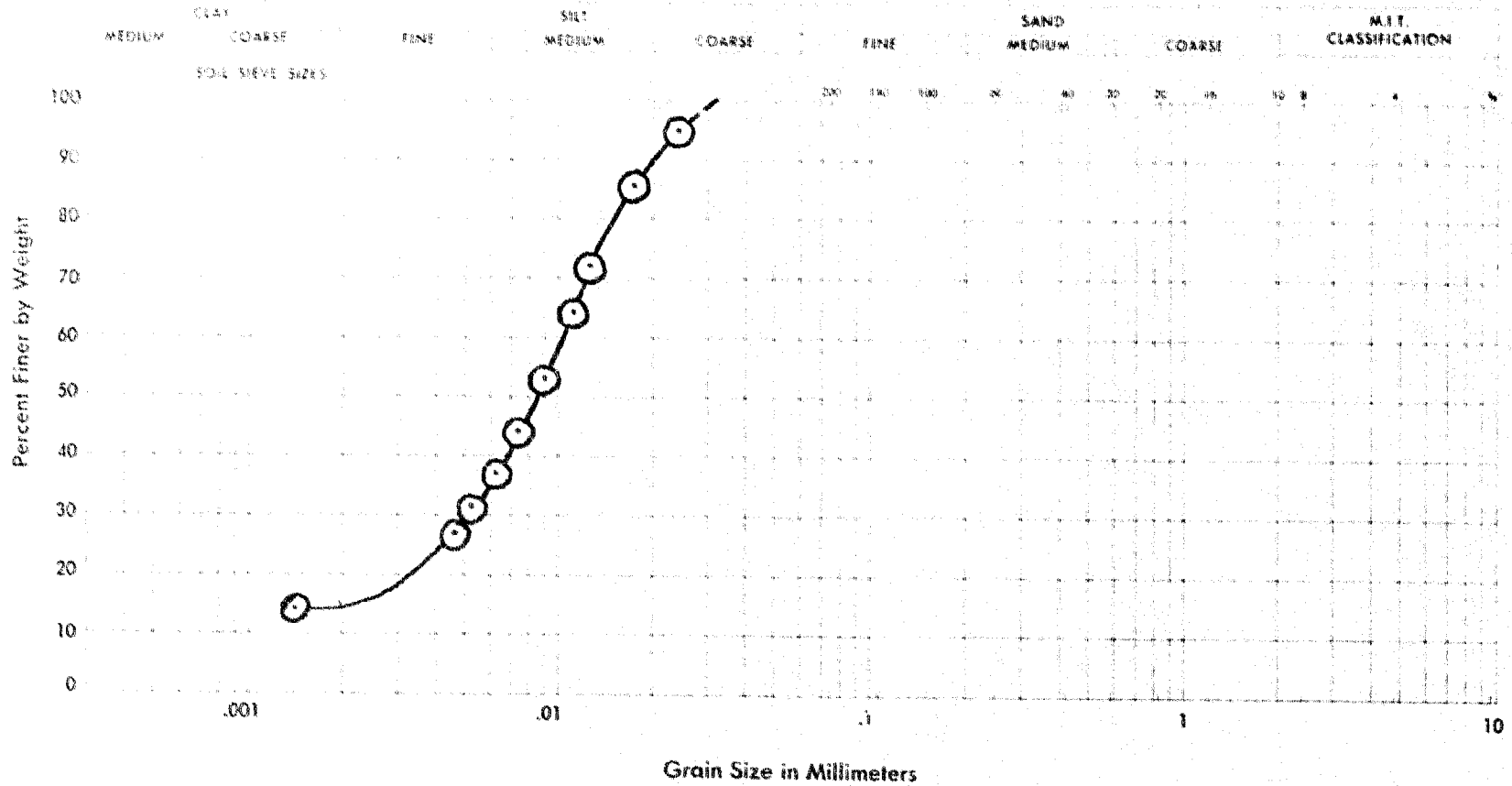
Order No. S-500/T-2335

Legend FH #2, Sample No. 6.



# RACEY MacCALLUM AND ASSOCIATES LTD.

## GRAIN SIZE DISTRIBUTION



Project HUNTER RIVER BRIDGE, BOLTON, ONTARIO.

Legend SI #4, Sample No. 5.

Order No. S-500/T-2335

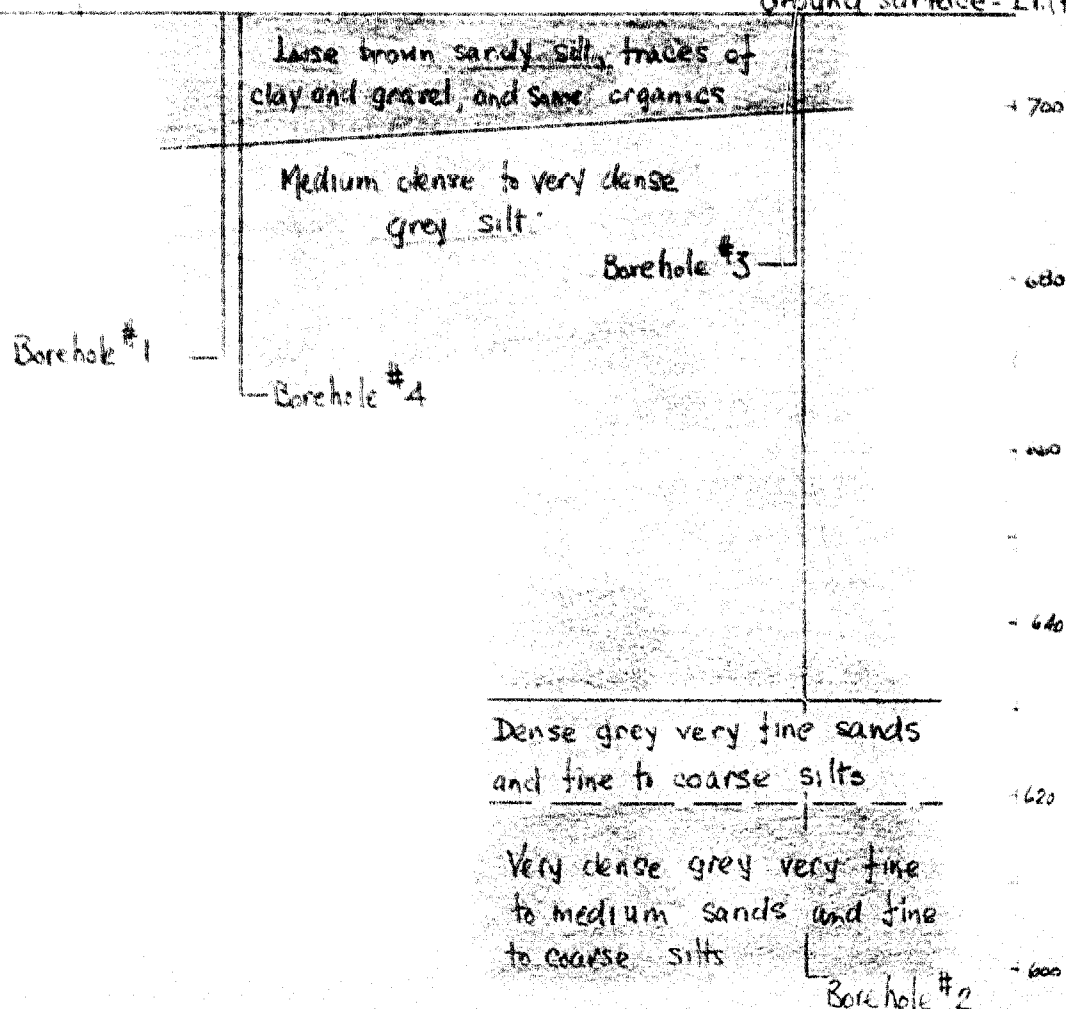


# SECTION THROUGH STRATA ON & HWY No 50 at HUMBER RIVER BRIDGE

Order No. S-500/T-2335  
Prep By J.P.

Existing bridge

Ground surface - El. (ft)



Scale : 1" = 20'

Location: See enclosure No. 1

*Memo to* Mr. A. M. Toye, *Date* December 30, 1960.  
Bridge Engineer. *Subject* \_\_\_\_\_  
*From* Materials & Research Section.

Attention: Mr. F. I. Hewson.

Re: Humber River Bridge at Bolton,  
Highway No. 50, District No. 6,  
W.P. 35-60.

Pile Loading Tests:

Static load tests on two Class 'A' untreated timber piles driven in the area of the North abutment of the above proposed structure have been completed. The load-settlement and time-settlement curves obtained from the tests are attached to this memo. Our comments pertaining to the proposed pile design are as follows:-

- (1) The maximum safe permissible load per pile is considered to be 35 tons. Piles loaded to this capacity should be of the size and type tested and should be driven to a tip elevation 660.0', or below.
- (2) The settlement of the pile group proposed should not exceed 1 1/2 inches with the individual piles carrying the maximum permissible load of 35 tons.
- (3) The test piles were driven by means of a hammer with a rated energy of 12 ft. kips per blow. The resistance to driving recorded for the last foot of penetration of the test piles was 4 blows/inch. A required minimum driving resistance of 4 blows/inch should be specified in the piling contract in addition to the maximum tip elevation noted in Item (1) above.

cont'd. /2 ...

Dewatering Problem:

In conjunction with the analyses of test pile results, the Foundation Section have carried out further analyses of the soil type at and below the proposed footing founding elevation. This has led us to conclude that the recommendation contained in the Soils report prepared by Racey, MacCallum and Associates, pertaining to the effectiveness of dewatering excavations by well points, is not based upon fact and, in reality, is not a practicable engineering solution. The particle size distribution of the soil is such that the dewatering could only be carried out using electro-osmosis, or stabilized by freezing.

This finding has resulted in a detailed review of the proposed footing design and the following problems appear to us to make this design impracticable:-

(1) The elevation of the underside of the proposed pile cap is in the order of 6 to 8 feet below the proposed bed of the River Diversion and some 12 to 13 feet below normal river level - (which can be assumed to be ground water level). Due to the fine-grained soil type, a stable footing excavation in the dry to the depth required, cannot be guaranteed. Theoretically, "piping", or a "quicking" condition can be predicted. The conditions contributing to piping will be further augmented by the driving of displacement type piles into the soil below the footing elevation.

(2) The following alternative construction procedures have been considered: (a) making the footing excavation without pumping the water out as excavation proceeds and then driving the piles through the flooded excavation to cut-off elevation; or (b) driving the piles from ground elevation to cut-off elevation by means of a follower, and then making the excavation in the wet. In both methods, tremie concrete would be necessary and even then, a stable excavation bottom cannot be guaranteed. No simple remedial or corrective measure can be adopted should this condition occur.

In view of the foregoing comments, it is our opinion that the footing design should be revised. The obvious alternative appears to be to drive interlocking steel sheet piling around the footing perimeter and then to remove the material from within the enclosure to footing elevation. Footings bearing directly upon the silty subsoil, can be designed using an allowable contact pressure of 3 tons/sq.ft. if the above procedure is followed. Sheet piling should penetrate to a depth of 8 to 10 feet below the footing founding elevation. The sheet piling should be left in place.

If, during excavation, quicking should develop, the enclosure can be flooded and excavation carried out under water. A tremie seal would then be required prior to dewatering.

The foregoing comments have been discussed with Mr. T. Gregg of Messrs. Morrison, Hershfield, Millman & Huggins, and include suggestions put forward by him. A preliminary comparison of cost of timber piles vs. steel sheet piling indicate that the sheet piling and simple spread footing design is slightly less expensive.

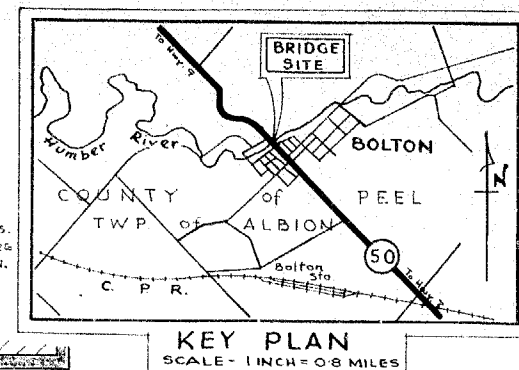
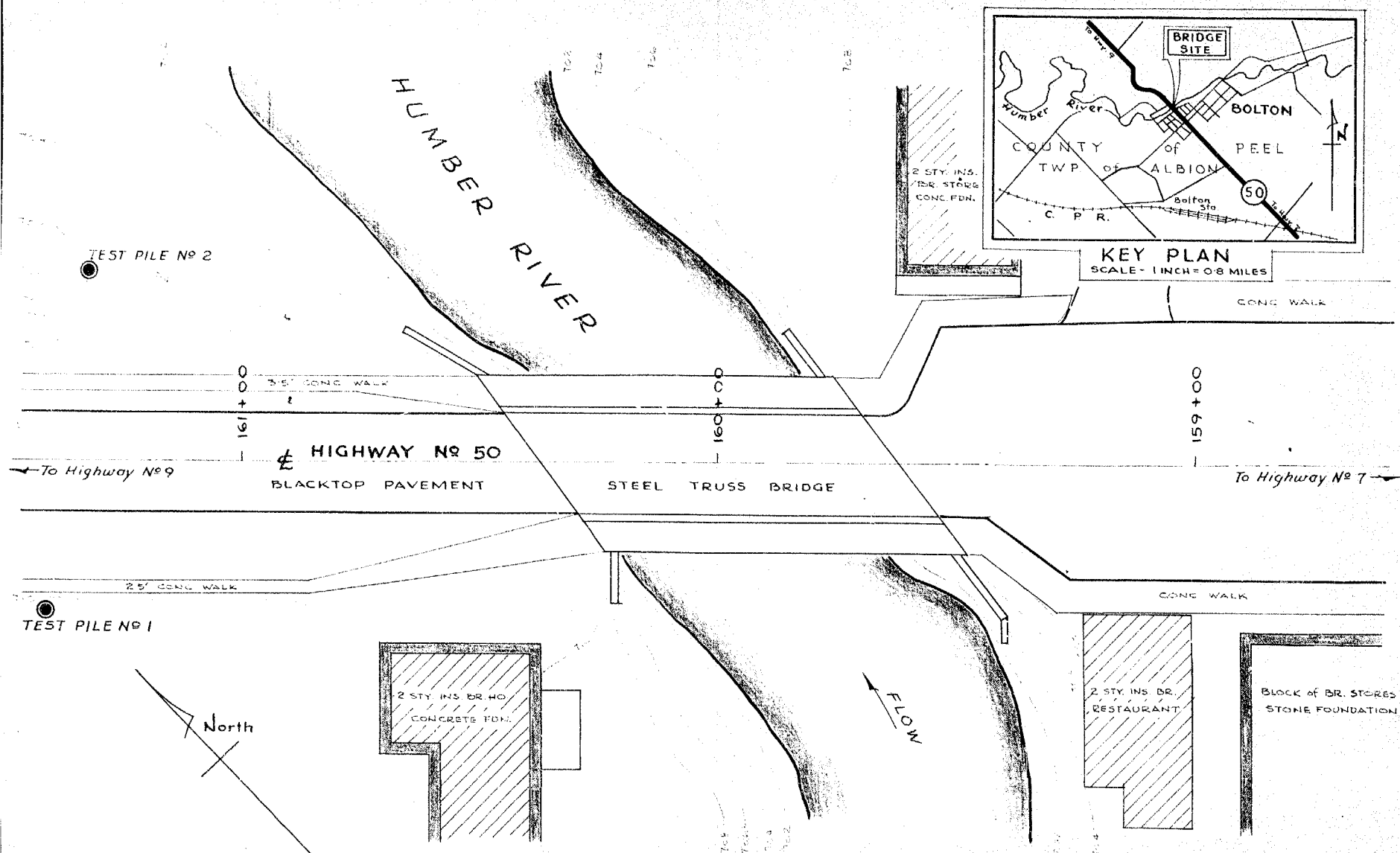
We would appreciate if you could give consideration to the above alternative design and authorize Mr. Gregg to complete his design on this basis.

*L. G. Soderman*

L. G. Soderman,  
PRINCIPAL FOUNDATION ENGINEER

IGS/MdeF  
Attach.

cc: Foundations Office  
Gen. Files.

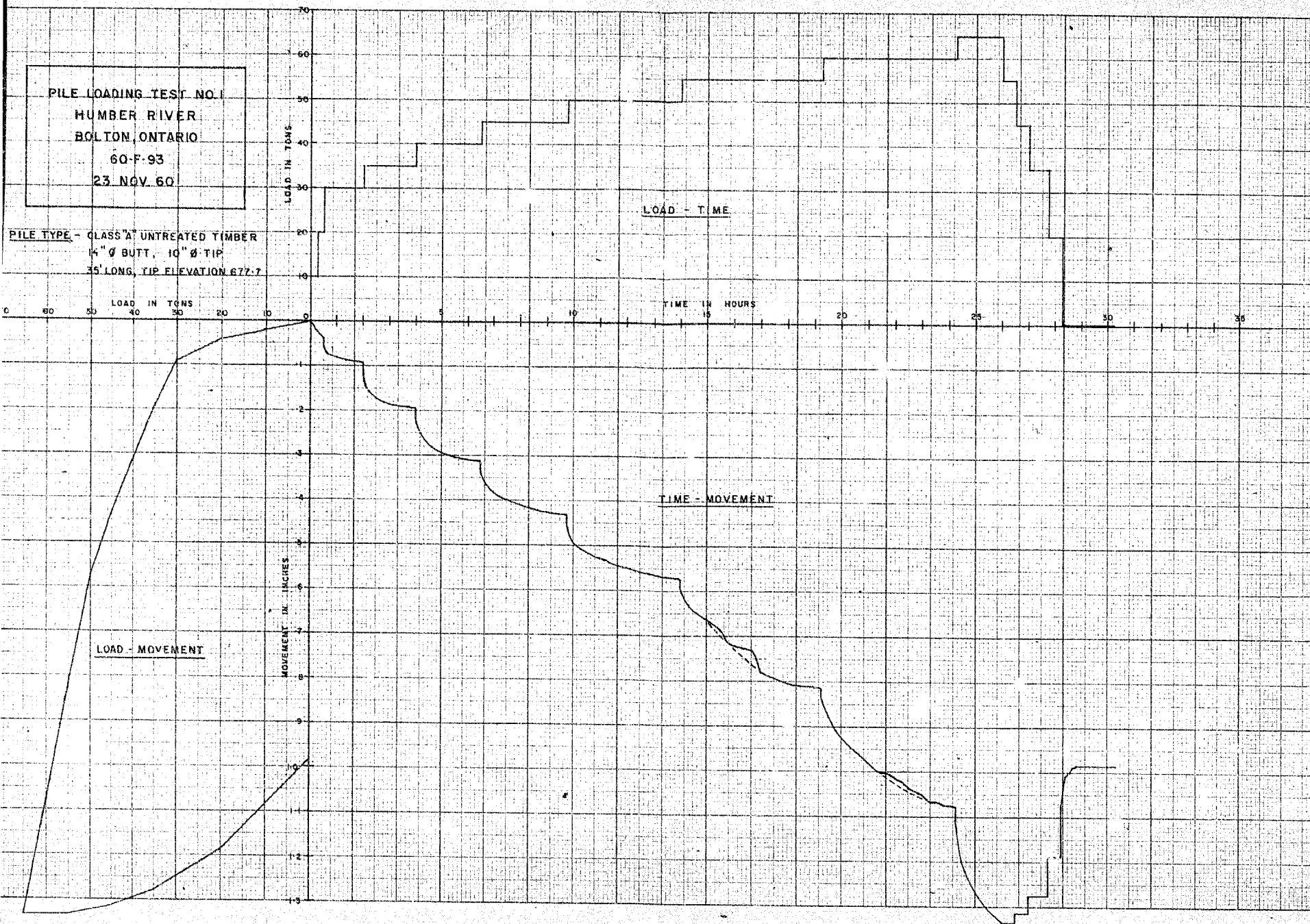


ORIGINATED L. SODERMAN	DEPARTMENT OF HIGHWAYS - ONTARIO	SCALE 1 inch = 20 feet
DRAWN H. D. REED	<b>MATERIALS &amp; RESEARCH SECTION</b>	W. P. NO. 35-60
CHECKED L. G. S.	<b>PILE TEST LOCATIONS</b>	JOB NO. 60-F-93
APPROVED L. G. S.	HUMBER RIVER - BOLTON, ONTARIO	DWG. NO. <b>60-F-93A</b>
DATE 28 Dec. 1960		

Ref. Plan - E 3775-1 (MAR 6-85)

PILE LOADING TEST NO. 1  
HUMBER RIVER  
BOLTON, ONTARIO  
60-F-93  
23 NOV 60

PILE TYPE - CLASS "A" UNTREATED TIMBER  
14" Ø BUTT, 10" Ø TIP  
35' LONG, TIP ELEVATION 677.7



PILE LOADING TEST NO. 2

HUMBER RIVER

BOLTON, ONTARIO

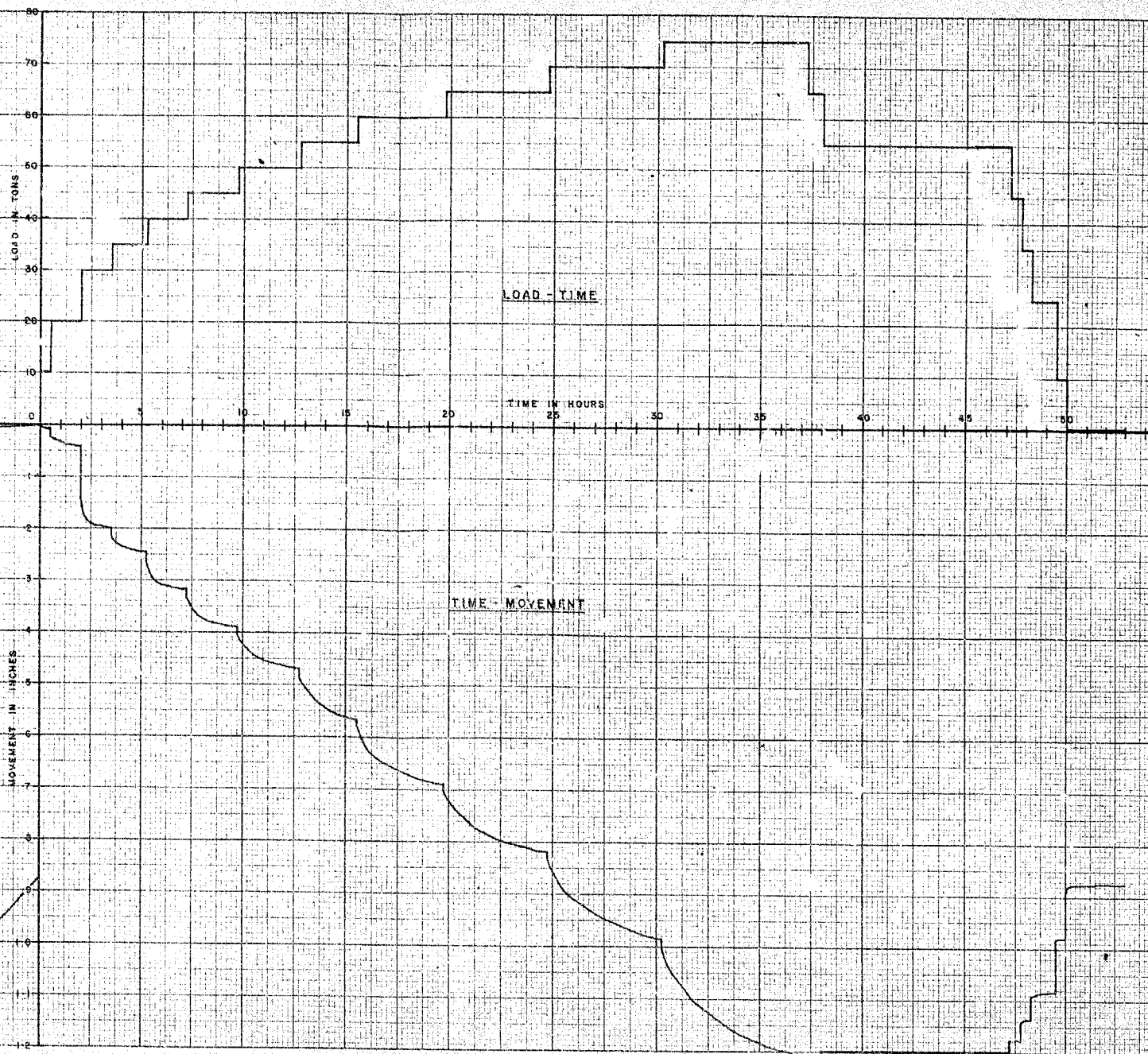
60-F-93

26 NOV 60

PILE TYPE - CLASS "A" UNTREATED TIMBER

15 3/4" Ø BUIT, 10 3/4" Ø TIP

35' LONG, TIP ELEVATION 673.1







ONTARIO  
DEPARTMENT OF HIGHWAYS

*Memo to* Mr. A. M. Toye, *Date* January 16, 1962.  
Bridge Engineer. *Subject* \_\_\_\_\_  
*From* Materials and Research Division,  
(Foundation Section)

Attention: Mr. S. McCombie.

Re: Humber River Bridge at Bolton,  
Hwy. No. 50, District No. 6,  
W.P. 35-60.

The subsoil conditions for the above-mentioned structure were first investigated in June and July, 1960, by the soil consultant, Messrs. Racey, MacCallum & Associates, and a report containing factual information with recommendations for the footings was submitted. There were subsequent discussions between the Bridge Design Consultant and the representatives of this Section pertaining to the design and execution of the foundation work.

To clarify the question of piled foundations, two pile loading tests were carried out by this Section and the results reported and discussed. As a final result of all the above-mentioned investigations and discussions, spread footings with a safe bearing load of 3.0 T/sq.ft., placed 6 ft. below the new diverted creek bottom, were decided upon. Because of the proximity of free water and the required depth of footing excavations, danger of piping within the excavation was considered as possible, and interlocking steel sheet piling driven to about ten feet below the proposed excavation bottom was incorporated in the design.

After the design was completed and submitted for review, it was brought to our attention that the proposed and above-mentioned sheet piling would add about \$50,000 to the cost of the structure. In the light of this fact, the Foundation Section has carried out some additional field and laboratory work in order to explore the possibility of eliminating this costly construction procedure.

cont'd. /2 ...



At the site of the proposed structure on Hwy. 50, an additional borehole was drilled at the South-west corner of the proposed South abutment. Continuous sampling was carried out from a depth of 17.0' to 29.0'. The samples were carefully identified and classified. In the laboratory, Atterberg limits and comparative permeability tests were carried out.

The investigation has shown that from 14.0 ft. downward, the subsoil is stratified, consisting of layers of clayey silt, silt, and silty clay. The stratification is shown on the accompanying sketch.

Comparative permeability tests have shown different coefficients of permeability for the silt material - ( $K = 3.5 \times 10^{-4}$  in/sec) and the clayey silt material - ( $K = 1.26 \times 10^{-7}$  in/sec). These values cannot be taken as absolute, but rather as indications of permeability differences.

Two boreholes were also drilled, one at each of the two bridges located on King Street's crossings of the Humber River. These structures were built on spread footings, supposedly some 6 ft. below river bottom and during construction, neither dewatering difficulties nor piping were encountered. If the soil conditions at these sites were found to be comparable with those at the site of the proposed structure on Hwy. 50, the extrapolation of experience would be justified. However, the soil conditions were not found to be comparable, the subsoil here, being clayey silt of very to extremely high density, 'N' values being in a range of 40 to 160 blows/ft.

The above-mentioned facts and newly-gathered evidence point to the choice of sheet piling as incorporated in the design as a good and safe engineering solution. However, the stratified character of the subsoil, the difference in permeability of different layers are valid evidence that no homogeneity exists and therefore, deviations from ideal conditions for piping exist. It is practically impossible to incorporate these factors into any kind of computation or analyses, and only field trial sections could provide a reliable and true answer.

It is therefore our recommendation that work at the bridge site be started without steel sheeting and this be used only if field evidence proves it necessary. It is recommended that first the new river bed at the bridge site be excavated to its final elevation. If this is done while the water in the river is controlled and kept as low as possible (which, we understand, is possible), dewatering of the excavation is not expected to present any difficulty. After this stage is completed, excavation with simple dewatering of one of the wing walls should be commenced.

Here, evidence will be gathered if sheeting will or will not be necessary. In case that this proposal is accepted, it is suggested that the Foundation Section be advised of the commencement of construction in order that assistance, guidance and advice could be given to the District's supervisory personnel.

We believe that the above information will enable you to arrive at the final conclusions and also make the final decisions. However, if there are any additional questions that you would like to discuss, please feel free to call on our Office.

A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.  
Per:

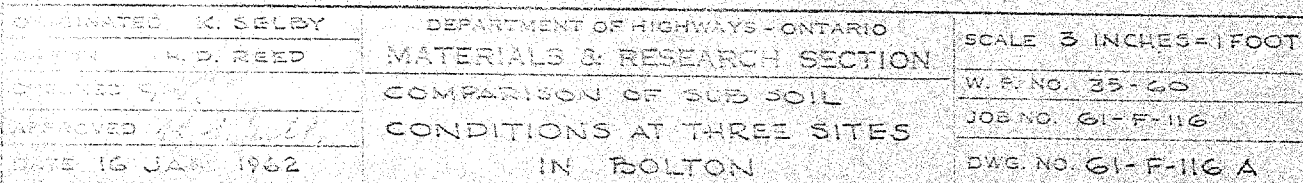


(K. G. Selby,  
SR. PROJECT FOUNDATION ENGR.)

KGS/MdeF  
Attach.

cc: Foundations Office  
Gen. Files.

KING ST. BRIDGE No 2



# H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER  
V. MILLIGAN

2444 BLOOR ST. W.  
TORONTO 9  
RO. 7-9201

February 8, 1962

Morrison, Hershfield, Millman & Huggins Ltd.,  
96 Bloor Street West,  
TORONTO 5, Ontario.

Attention: Mr. J. T. Gregg, P. Eng.,

RE: HUMBER RIVER BRIDGE AT BOLTON  
HIGHWAY NO. 50, DISTRICT NO. 6  
WP35-60.

Dear Sirs:

Further to the meeting between your Mr. Gregg and the writer on February 2, 1962, we have reviewed the data supplied to us concerning the soil conditions at the site of a proposed bridge on Highway No. 50 in Bolton, Ontario. The data supplied included copies of correspondence between your firm and the Department of Highways, Ontario, drawings Nos. D4724-1 and D4724-2, of the proposed bridge foundations and detailed memoranda concerning the soil conditions at the site and the soil conditions at each of two bridges located on King Street, Bolton.

The proposed bridge foundations are to be spread footings founded in a stratum of stratified silt, clayey silt and clay, some 11 to 12 feet below normal river water level. Because of the depth below water level to foundation grade excavation in the dry cannot be guaranteed and a "piping" condition could well occur. It was therefore proposed by the Foundation Section of the Department of Highways, Ontario, that interlocking steel sheet piling be driven around the footing perimeter and the excavation carried out within a strutted cofferdam. The sheet piling would be driven to a penetration of about 8 to 10 feet below the footing elevation and would be left in place as scour protection. If during construction "boiling or piping" should develop, the cofferdam would be flooded and the excavation then carried out under water. The base of the excavation would be covered with tremie concrete seal prior to final dewatering. All the existing evidence

concerning the soil conditions at this site indicates that this proposed method of construction, as incorporated in the design, is a sound engineering solution.

Now, it has been suggested by the Department of Highways, Ontario in a letter by Mr. A. Selby dated January 16th, 1962 and in a memorandum by Mr. S. McCombie dated January 29th, 1962, that the proposed sheet piling procedure would add approximately \$50,000.00 to the cost of the structure. Whether this sum is correct or not, I do not know. It has been proposed, in the two letters referred to above, that the sheeting should not be used at the start of the construction and should be used only as experience in the dewatering of the excavation for one of the wing walls proves it to be necessary.

On February 5th, 1962 in a discussion of this recommended construction procedure with Mr. A. Sternac and Mr. A. Selby of the D.H.O., Foundation Section, they pointed out that this method of construction may be unsafe but they considered it to be justified on a trial basis in view of the fact that the two King Street bridge foundations close to the site had been constructed without the use of sheeting. With Mr. Selby, I examined laboratory analyses and soil samples from the sites to which they referred. The results of D.H.O. borings are summarized on drawing No. 61-F-116A dated January 16th, 1962. It is significant to note that the soil conditions at the sites of the bridges on King Street are not comparable with soil conditions for the proposed Highway #50 structure. The King Street structures are founded in a relatively uniform clayey silt of high relative density. Typical classification tests for this material indicate that the liquid limit of the clay silt is of the order of 25 to 31, the plastic limit approximately 16 to 17 and the water content generally well below the liquid limit by some 2 percent.

On the other hand, the highly stratified sub-soil conditions at the proposed bridge are anything but uniform. Individual layers of silt and clayey silt range in thickness generally from 2 to 6 inches and are often quite erratic; individual clay layers are relatively thin and of the order of 1 inch or less in thickness. Six Atterberg limit tests only were carried out on typical clayey silt layers from samples at foundation grade and some 10 feet below it. These tests are listed below:

<u>L<sub>L</sub></u>	<u>P<sub>L</sub></u>	<u>W</u>
21.5	18.2	22.9
21.0	19.8	22.6
22.5	18.2	23.2
22.2	19.2	23.2
22.0	20.1	22.8
22.7	19.0	23.6



It may be noted that in every instance the water content of these layers is above the liquid limit. Consequently, while the presence of the thin clay layers (which were not tested in the laboratory) will tend to cut down the possibility of piping or boiling, the presence of the clayey silt layers, (probably of low shearing strength as we may infer from the Atterberg limit tests) raises the question of instability at the base of the excavation during construction. This question of instability is of some importance when we consider that the north abutment of the proposed bridge is located close to the relatively steep slope at Queen and Hickman Streets and the south abutment will be within some 10 feet of the existing Humber River.

Thus, should instability develop, it is possible that sheeting can not then be used and the bridge may have to be founded on friction piles. We understand that in this case, the cost of the structure would be increased by approximately \$50,000.00. Instability could have more severe results than simply a change in the foundation type but in view of the stratified character of the sub-soil it is difficult to make an accurate analysis of what these effects would be.

This possibility of instability was discussed with Mr. Stermac and Mr. Selby at our meeting and it was agreed that to make an initial trial excavation at one wing wall without sheeting posed a risk which could not at the present time be accurately assessed. To have this risk, provided that construction was carefully controlled may be possibly justified should a saving of \$50,000.00 result, but to have this risk for a much lesser possible saving, say \$25,000.00, was not justified.

This then represents the substance of my review of this problem. Whether the exact sums of money referred to above are \$50,000.00 or \$25,000.00 I leave to you to estimate.

Yours faithfully,

H. J. GOLDBER & ASSOCIATES LTD.

*V. Hilligan*

V. Hilligan, P. Eng.

VM/jb  
6207

#60-F-270C

W.P. #35-60

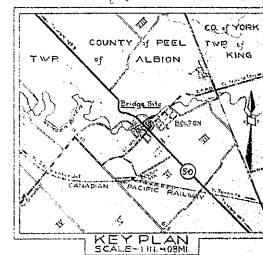
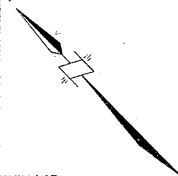
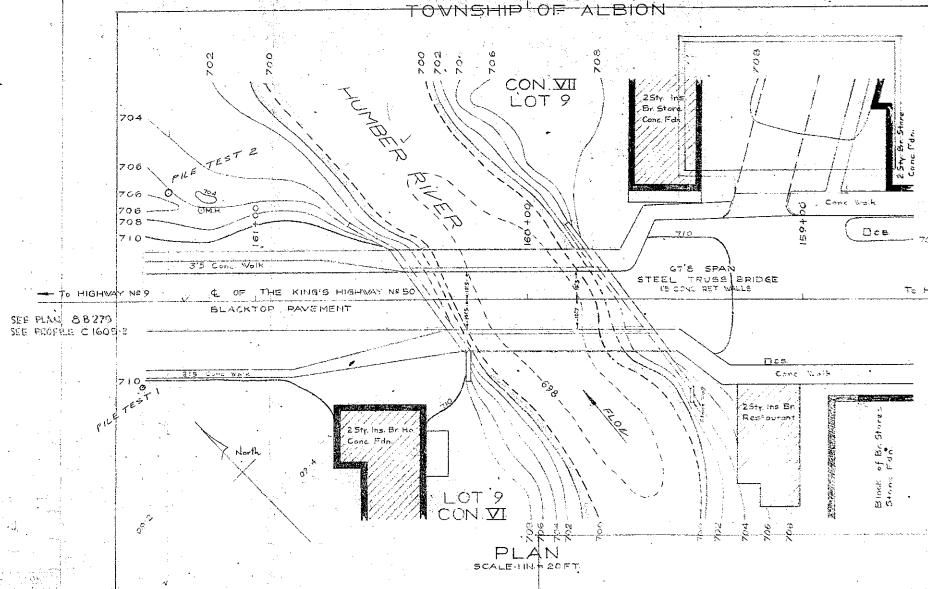
Hwy #50

HUMBER R.

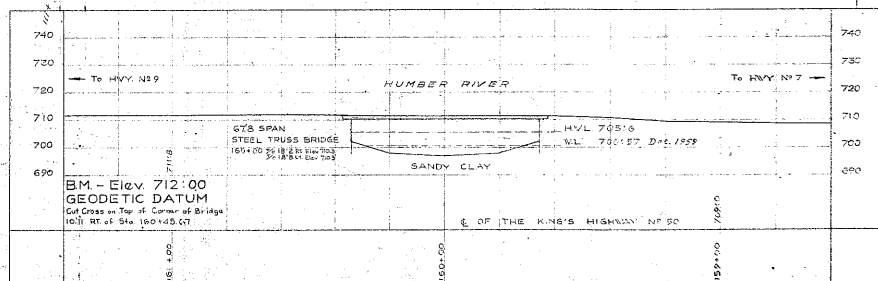
BRIDGE

AT BOLTON

COUNTY OF PEEL  
TOWNSHIP OF ALBION



GEODETIC B.M. # 260, Elev. 858.151  
BOLTON: Very small concrete box culvert under C.P.R. about 1/4 mile, south of station, 250' east of west side of road, 1/2 mile east of road, distance between east and west side of road is about 1/2 mile. This is the only bridge in the area.



W.P. 35-60

DEPARTMENT OF HIGHWAYS ONTARIO  
PLANNING & DESIGN BRANCH

DISTRICT NO. 6

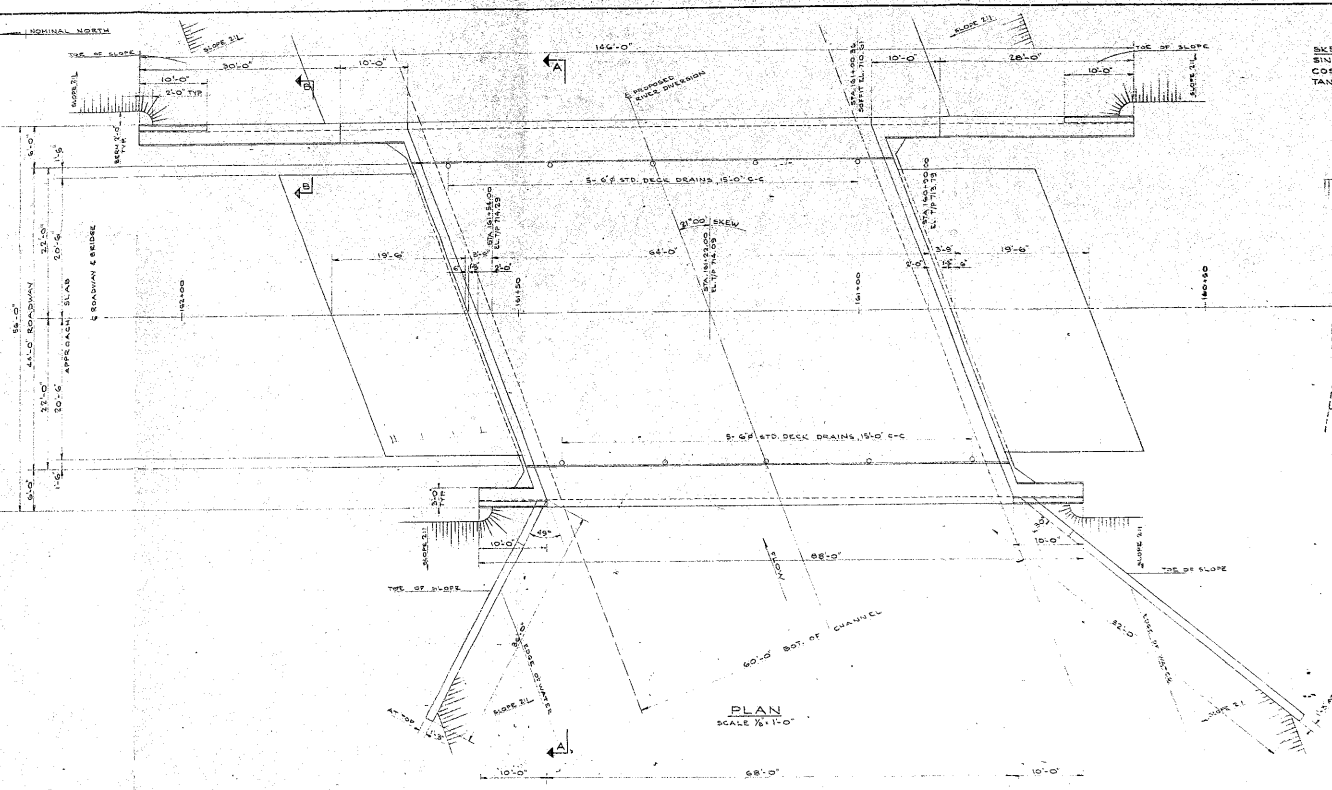
CROSSING  
AT  
HUMBER RIVER  
AND  
THE KING'S HIGHWAY NO. 50  
VILLAGE OF BOLTON

LOT 9 TOWNSHIP OF ALBION CONS. VI/22 COUNTY OF PEEL

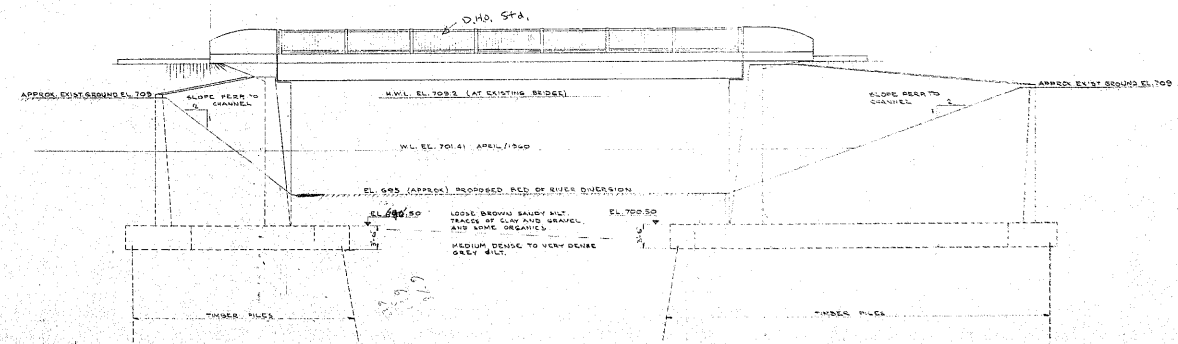
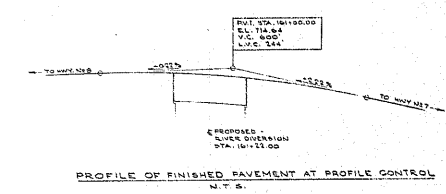
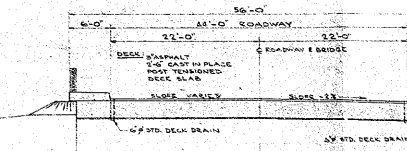
BRIDGE SITE

SURVEY BY CHIEF OF PARTY: J. NELSEN SUPERVISOR: D. SCOTT	APPROVED: <i>[Signature]</i> DATE: FEB. 1960
DRAWN BY DRAFTSMAN: F. BEAL & FRANK SUPERVISOR: J. CASS	SCALE: AS SHOWN DATE OF SURVEY: DEC. 1959 DATE OF PLAN: FEB. 1960
CHECKED BY DRAFTSMAN: H. LEWIS SUPERVISOR: H. FLEASANCE	PLAN: E 2775-1





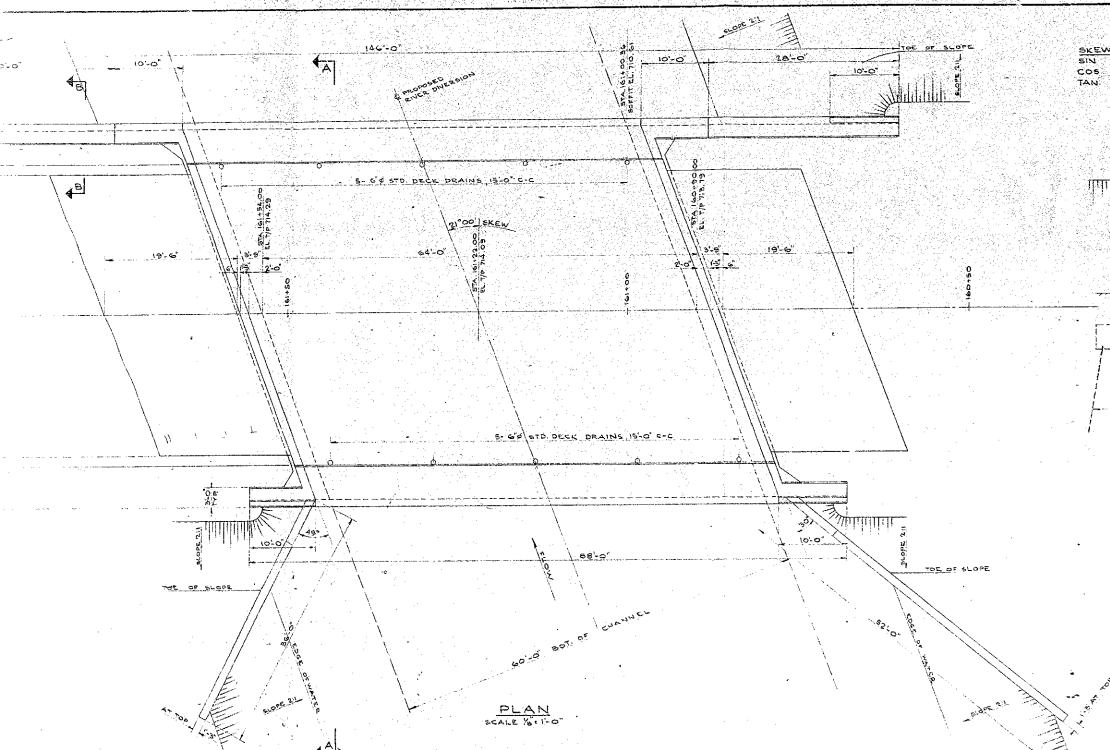
NEW ANGLE 31°00'  
 SIN. 0.51837  
 COS. 0.95338  
 TAN. 0.53586



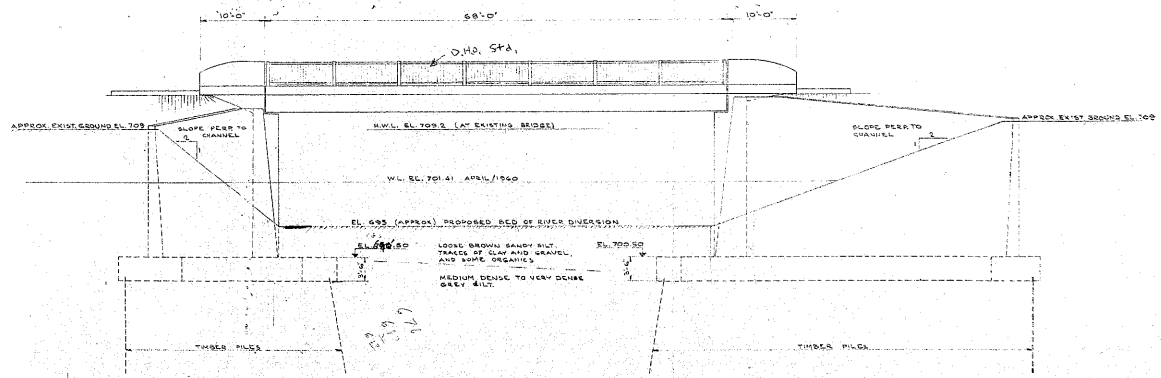
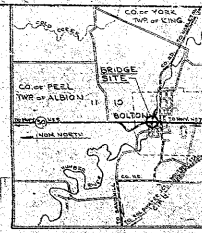
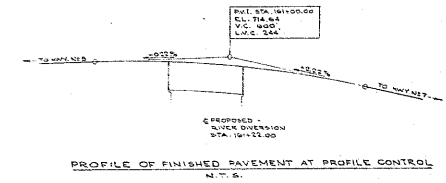
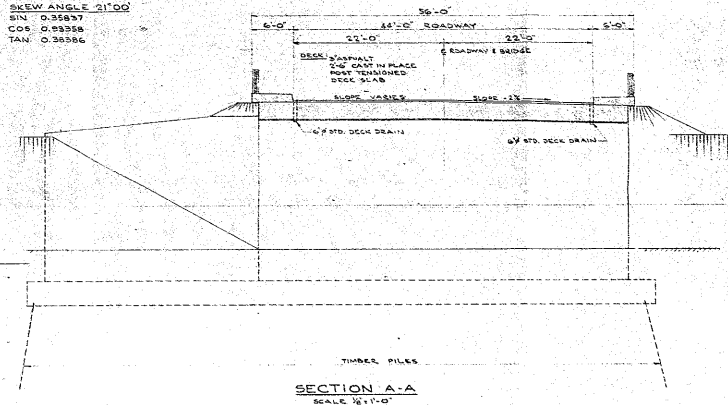
SECTION B-B  
 SCALE 1/8"=1'-0"

ELEVATION  
 SCALE 1/8"=1'-0"

DATE	BY	DESCRIPTION	REVISION	NO.	DATE



SKW. ANGLE 21'00"  
 SIN 0.35837  
 COS 0.93358  
 TAN 0.38386



As Supd to R.D.D.  
 etc.

OFFICE	
REV. DWS.	TITLE
MORRISON, HERSHFIELD, MILLMAN & HUGGINS, LTD. CONSULTING ENGINEERS	
DEPARTMENT OF HIGHWAYS - ONTARIO BRIDGE OFFICE - TORONTO	
HUMBER RIVER BRIDGE AT BOLTON	
THE KING'S HIGHWAY No. 50	DIST. No. 6
CO. PEEL	LOT 3
TWP. ALBION	CON. VI & VII
PROPOSED GENERAL ARRANGEMENT	
APPROVED	
OCT 24 1960	
DESIGN ENGINEER	
BRIDGE ENGINEER	
DRAWING	
DATE OCTOBER 27 1960	