

MEMORANDUM

To: Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

FROM: Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

Attn: Mr. S. McCombie

DATE: December 19, 1963

OUR FILE REF.

IN REPLY TO

SUBJECT:

FOUNDATION INVESTIGATION REPORT BY:
E. M. PETO & ASSOCIATES, LIMITED -
Big Creek Bridge at Delhi, Hwy. #3,
W.P. 138-63, District #2, London.

The above report submitted by E. M. Peto & Associates,
has been reviewed.

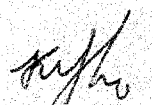
Following a subsequent discussion on several points
of the report, the Consultant submitted a letter with additional
information, which is included herein.

We believe that the factual data and recommendations
contained in this report, will suffice for your future design
work. However, should there be any queries concerning this
project, please do not hesitate to contact our Office.

KYL/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
A. Gater
H. C. Dernier
J. Roy
A. Watt

Foundations Office
Gen. Files


K. V. Lo,
SUPERVISING FOUNDATION ENGR.,
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

BA 1729

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

Our Job Number 63214

789 - 1126.

29th November 1963.

Department of Highways - Ontario,
Materials and Research Division,
Foundation Section,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. A.G. Stermac,
Principal Foundation Engineer.

Gentlemen,

W.P. 138-63, Hwy. #3,
Big Creek Bridge at Delhi,
District #2.

We have pleasure in submitting ten copies of our report
#63214 on the subsoil investigation for the above Project.

The conclusions which we have reached can be summarized
as follows:

The principal characteristic of the subsoil with regard
to the bridge foundation design is the occurrence of a dense stratum of
interlayered fine sand and silt with clay seams, the upper boundary of

which appeared to be near elevation 683, which is 17 ft below water level in the Big Creek. This deposit is recommended as suitable for the support of a pile foundation, although an alternative scheme, involving high-load piles supported on bedrock or on very dense till, is also discussed.

The fluvial deposits located above the average elevation 683 are highly pervious and generally in a loose state of packing, but contain wood fragments and some compact gravel. Tree trunks and broken concrete may be a hazard to pile driving in the uppermost layers of subsoil and are also present in the embankment fill. Timber piles, which supported previous bridges at the site, were also noted.

While we consider the report to be complete within your terms of reference, we would gladly provide additional assistance should you wish to discuss further any aspects of this investigation.

Yours very truly,

E. M. PETO ASSOCIATES LTD.



E. M. Peto, P. Eng.

RK/ap

DEPARTMENT OF HIGHWAYS - ONTARIO

SUBSOIL INVESTIGATION

W.P. 138 - 63, HWY. #3,
BIG CREEK BRIDGE AT DELHI,
DISTRICT #2.

E. M. PETO ASSOCIATES LTD.,

1287 Caledonia Road,
Toronto 19, Ontario.

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BOREHOLE LOGS

SITE PLAN and PROFILES

A. INTRODUCTION

1. Authority

The work described in this report was authorized by Mr. A. Rutka, Materials and Research Engineer, Department of Highways - Ontario, by letter dated October 21st, 1963.

2. Scope of work

The subsoil investigation was required in connection with the proposed replacement of a bridge on Highway #3 over the Big Creek in Delhi, District No. 2 under W. P. 138-63.

The proposed new bridge will probably be a three-span structure, with two outside spans of 50 ft each and a 60 ft central span. The width is to be 70 ft. A continuous span design is considered.

The centre lines of the abutments and piers will be at an angle of 65 degrees to the longitudinal axis of the bridge.

In addition to the investigation of the subsoil below the bridge foundations, some indication was required of the soil conditions within the existing and below the new embankments leading to the bridge.

B. GENERAL INFORMATION

1. Site Description

The site is situated at the crossing of the Big Creek by Highway #3, at the western limits of the Town of Delhi, Ontario. A key plan included on the appended Drawing shows the general location of the site.

A survey plan of the site is also included on the Drawing, and has been traced from Client's Plan E-4301-1. The ground contours at the site and the outlines of the existing as well as the proposed bridge are included.

The existing bridge has a span of 98.0 ft and is designed as a concrete "bowstring." The abutments partly project into the creek and are protected by timber sheeting, with a cut-off below the water surface. It is believed that the bridge is supported on timber piles and was constructed around 1929. Apart from signs of deterioration of concrete (steel reinforcement is exposed in many places), the bridge appears to be in a sound state, although numerous vertical tension cracks were observed in the concrete hand rail near the middle of the span.

B. GENERAL INFORMATION - Cont'd

The bridge is subjected to very heavy traffic; although it is located within a 30 mph speed limit, it continuously carries the heaviest trucks, descending to the structure at a speed considerably in excess of this limit.

Timber piles, which project above water surface below the bridge, are the remains of the foundations of an earlier structure.

The embankments leading to the bridge have steep slopes, which are indicated on the contours included on the site plan; near the abutments, the slope in places is steeper than 45 degrees. According to information obtained from local inhabitants, the embankments were placed over an uncleared marsh land, and were subject to considerable settlement. Unconfirmed information indicated that the road surface near the bridge was rebuilt some five years ago, when the uppermost 5 ft of fill was replaced.

It is believed that springs, originating in the elevated areas to the west of the site, cause local instability of the fill and several 18 inch C.I.P. drains are projecting from the embankments into the stream and are shown on the site plan.

B. GENERAL INFORMATION - Cont'd

At the time of the site investigation, the water level in the Big Creek was at elevation 700.9. The stream has an approximate width of 35 ft below the bridge but is wider at other sections near the crossing. From the contours of creek bottom shown on the survey plan, the depth of water reaches 12 feet.

The flow is very slow, since the water is retained by a dyke, located approximately 200 yards to the south of the site. The height of the dyke is about 12 feet, and the stream downstream of this structure is very shallow but flows at a rapid rate.

A lumber yard is located on a level area to the south of the eastern end of the bridge, where the elevation is only 2 to 4 ft above the creek level. The owner of the yard informed us that no serious flooding occurred since the construction of the dyke.

The low-lying area to the south-west of the western end of the bridge forms a marsh land, which extends for some 200 feet from the water's edge. Walls of the valley, from visual observation, consist of brown sandy material, and rise to approximately 40 ft above the creek level.

B. GENERAL INFORMATION - Cont'd

The only buried service understood to be present at the site is a water main, located approximately 25 to 30 ft south of the centre line of the new structure, and therefore within the area to be covered by the abutments and piers of the new bridge, shown on the site plan. When the sun is shining and the water surface is smooth, the water main on the bottom of the creek is visible from the bridge.

Several trees are present at the south-eastern corner of the area to be covered by the new bridge, while only small bushes grow on the slopes at the south-western corner of the site.

2. Elevations

The elevations of ground surface at the positions of the test holes and of the water surface were measured by our engineer, and referred to a temporary bench mark of elevation 709.37, set up by the Department of Highways - Ontario; the bench mark is in the form of a nail and washer in the north-west root of 1.5 ft dia. elm tree, located 94.0 ft left of Sta. 26 + 60; this bench mark is indicated on the site plan supplied by the Client. The elevations are tabulated on the enclosed drawing and are also entered on the appended borehole logs, and are Geodetic.

B. GENERAL INFORMATION - Cont'd

3. Positions and Depths of Test Holes.

The positions of the test holes were chosen and set out by our engineer.

The general stratification of the overburden down to bedrock was established by boreholes 4 and 7. Standard cone probes were driven within 3.5 to 5 ft of these boreholes and allowed an approximate correlation between results of the cone tests, and standard penetration tests performed in boreholes.

The variation of density of subsoil with depth at other locations was investigated by six additional standard cone probes, of which four were put down from a raft, moored near the extreme ends of the proposed piers. At and below a probable pile toe level, the variation of density of subsoil with depth was remarkably uniform on the basis of the borehole and probe results and no further sampling at that depth was considered necessary.

However, three further, shallow boreholes were put down to investigate the composition of the existing embankments, and of the subsoil immediately below the low-lying areas, to be covered by the new embankment fill. These holes were terminated after penetrating through the surficial, fluvial deposits containing organic matter.

B. GENERAL INFORMATION - Cont'd

The positions of all the test holes are indicated on the enclosed Drawing.

4. Field Work.

The test holes were put down between 21st October and 15th November 1963. The work was performed by our field unit #6, employing a skid-mounted diamond drilling rig. Cone penetration holes 1, 2, 5 and 6 were driven from a raft.

The field work was directly supervised by our engineer, Mr. R. Kulesza.

Standard drilling and sampling procedures were followed. Split spoon samples were generally taken at 5 ft intervals, except in the embankments and surficial fluvial deposits where the spacing between the samples was 2.5 feet. In addition two Shelby tube samples were obtained from deep-seated deposits in test hole 7.

Special precautions were taken to seal off with a cement-sand mixture an artesian ground water condition encountered near the bottom of the two deep test holes. The artesian water pressure was measured by means of test hole casing projecting above ground level and retained overnight.

A summary of the day-by-day progress of field work is presented in Table C.

B. GENERAL INFORMATION - Cont'd

5. Laboratory Work.

Natural water contents were measured on most split spoon samples. The remaining laboratory testing was confined to an oedometer consolidation test on a typical clay layer, and two Atterberg limits and three grain size distribution tests on typical samples. In view of the apparent uniformity of the subsoil conditions at and below the probable foundation level, and because of the very favourable characteristics of the subsoil with respect to a pile foundation, further laboratory testing was not considered necessary.

C. OUTLINE OF GEOLOGY

The site is located over the Norfolk sand plain. The bedrock, consisting of gray limestone belonging to the Norfolk formation of Devonian age, was reached and proved by diamond drilling in borehole 7. The elevation of bedrock surface was 614.8, which agrees with bedrock contours of Norfolk County shown in Reference 1.

A natural gas field is situated to the south and south-west, commencing within a distance of 2 miles of the site.

C. OUTLINE OF GEOLOGY - Cont'd

The bedrock is directly overlain by an extremely dense fine sand and silt till layer approximately 5 ft thick, which was probably deposited during an early Wisconsin glacial period.

Above elevation 620 to 623, the subsoil consists of an extensive, dense stratum ^{of} interlayered fine silty sand and silt, with some clay seams. The clay layers are more frequent at greater depth, and the bottom 6 to 7 feet of the deposit is primarily a silty clay with pebbles, which may be a late glacial till, although in parts the layered character may indicate a lacustrine origin.

The stratified deposit of fine sand and silt with clay seams, according to Reference 2, is a sediment of a delta associated with the glacial lakes Whittlesey and Warren.

Recent, fluvial deposits were found near the surface and contained organic matter.

D. SOIL CONDITIONS

Details of the subsoil conditions encountered in the boreholes are described on the borehole logs, which include also natural water contents and results of standard penetration tests.

Simplified subsoil profiles, in the form of sections through the test holes, are presented on the appended drawing.

The subsoil can be divided into the following broad types, in the order of occurrence,

- a) Embankment materials
- b) Shallow fluvial deposits
- Interlayered fine silty sand and silt with clay seams
- d) Silty clay with pebbles
- e) Fine sand and silt till
- f) Limestone bedrock

The above soil types will now be characterized in turn.

a) Embankment materials.

The boreholes 3 and 4, which were put down from the top of the embankments, indicated that the embankments are formed mostly of a light brown silty material, of soft consistency and containing variable admixtures of fine and medium sand and some clay. Organic matter was present mostly in the lower layers of the fill, and apparently included decayed timber logs. Softened concrete chunks were also encountered in the lower portions of the embankments.

D. SOIL CONDITIONS - Cont'd

b) Shallow fluvial deposits.

These materials were quite variable, but generally, fine or medium sand predominated and included a variable silt content and organic layers. A gravel deposit extended between the approximate elevations 684 and 693 in test hole 4, and was in a compact to dense state, in contrast to the generally loose or soft consistency at similar elevation in the remaining boreholes and probes. Fragments of decayed timber were also encountered.

c), d) Interlayered fine silty sand and silt, some clay seams.

This deposit was the main overburden at the site. Test holes 4 and 7 showed that its upper boundary, near elevation 683, was reasonably uniform, which was also indicated by the sharp increase in cone penetration resistance near this level in all cone holes.

Grain size distribution curves on typical samples of the material are included with this report. Fine sand predominates in the upper part of the deposit and is interlayered with silt and, less frequently, with seams of silty clay. However, with depth the silt and clay content increases, until near elevation 630 the material is predominately a layered clay.

D. SOIL CONDITIONS - Cont'd

Below approximate elevation 629 the material was a clay with pebbles but contained seams of fine sand and silt. Although this layer may be a clay till, the boundary between this and the overlying stratum is difficult to distinguish and is of minor consequence to the project. A consolidation test was performed on a typical layer of clay from elevation 646; the results are presented on Fig. 2. Atterberg limits of two typical samples of the stratum are given in Table A.

In an attempt to correlate the results of the two types of penetration tests, standard penetration test blows were plotted on Fig. 3 against cone blows, measured at the same levels in the adjacent cone holes. A relationship of approximately linear form is observable, and the average value of the ratio of cone blows to standard penetration test blows can be taken as 2.0 to 2.25. The scatter of the individual results is probably caused by the interlayered character of the material, with fewer blows recorded for equal penetration in clay seams.

Assuming that the minimum depth of penetration of end-bearing piles would be to elevation 677, the average cone penetration resistance was determined between elevations 677 and 660, since between these elevations the subsoil would bear the highest stresses imposed by a pile group.

D. SOIL CONDITIONS - Cont'd

Average values of cone blows per foot between
elevation 677 and 660.

<u>Cone Hole No.</u>	<u>Number of foot increments</u>	<u>Average value of cone blows per foot</u>
1	17	93
2	17	80
3	9	136
4	17	92
5	17	100
6	17	100
7	13	99
8	11	109

The average of the results in the cone holes is 101 blows per foot. From Fig. 3, assuming the ratio of cone blows to the N-value, or standard penetration test result, as 2.25 to 1, the average value of equivalent "N", based on the cone blows, between elevations 677 and 660 is 45 blows per foot.

Between the same elevations, there were four standard penetration tests in each of boreholes 4 and 7, giving average "N" values of 42 and 48 blows per foot, respectively. The average of the two test holes is 45 blows per foot, which is identical with the above - determined average N-value deduced from the relationship plotted on Fig. 3.

D. SOIL CONDITIONS - Cont'd

On the basis of the above considerations, it appears permissible to consider the density of the subsoil between the chosen elevations to be uniform, with an average N-value of 45 blows per foot. This value can be used in the calculation of bearing capacity of piles supported not higher than elevation 677.

Results not greatly differing from this average would be obtained for greater pile penetrations.

e) Fine sand and silt till.

This deposit was encountered at elevation 620.6 in test hole 7 and 623.3 in test hole 4. In test hole 7, its thickness was 5.8 ft and it was followed by the limestone bedrock.

The till, consisting of a mixture of fine sand and silt with pebbles, is in an extremely dense state of packing, as indicated by standard penetration test resistance of 100 blows for 3.5 in. of penetration.

The deposit carried ground water under artesian pressure.

D. SOIL CONDITIONS - Cont'd

f) Limestone bedrock.

The limestone bedrock was encountered in test hole 7 at the elevation 614.8 and was proved by 7.2 feet of diamond drilling. A 100% core recovery was obtained.

The limestone is dense, fine grained, with occasional cherty partings. It is considered to form an excellent support for end-bearing piles, allowing a high load per pile.

E. WATER CONDITIONS

The ground water table in the surficial layers of the subsoil is controlled by the water level in the creek, which at the time of the site investigation was at elevation 700.0. In view of the fact that the stream level is controlled by a dyke located some 200 yards downstream, only minor variations in this level are anticipated.

Some seepage of ground water is believed to occur through the western embankment from the elevated areas located to the west of the site.

E. WATER CONDITIONS - Cont'd

The interlayered fine silty sand and silt stratum is relatively pervious, and an unlined test hole would collapse.

Ground water under artesian pressure was struck in the fine sand and silt till located immediately above the limestone bedrock. In test hole 7, the equilibrium level of the artesian water head was at elevation 711.1, which is 11.1 feet above the water surface level in Big Creek.

Artesian condition was also encountered in the same stratum in test hole 4.

F. ENGINEERING CONSIDERATIONS and CONCLUSIONS

1. Bridge Foundations.
- a) Choice of type of foundation.

It is recommended that the new bridge should be supported on a pile foundation, and, in our opinion, two alternative types of piles can be regarded as suitable.

- i) Short end-bearing piles, supported in the silty sand and silt stratum.
- ii) Long end-bearing piles, supported on bedrock or hard till.

The choice between the two alternative solutions will probably be based on considerations of economics; although both types are technically feasible and are considered in greater detail in the following sections, we are inclined to consider the short pile solution as the more practical.

A footing foundation is not considered practicable due to the loose state of packing of the sandy and silty subsoil above elevation 683, while construction of footings below this level would create considerable difficulties because of the high permeability of the subsoil.

F. ENGINEERING CONSIDERATIONS and
CONCLUSIONS - Cont'd

b) Bearing capacity and settlements.

i) Short piles.

It is considered that elevation 677 is the minimum suitable depth to which piles should be driven to obtain adequate vertical and lateral support. The loose consistency of the subsoil above elevation 683, as well as the practically uniform level throughout the site at which the density rapidly increases, with depth, appear to suggest that the materials above elevation 680 were deposited in a changing stream bed; this is further indicated by the variable composition of the shallow deposits. Elevation 683 to 685 appears to be the lower limit of previous scour action.

It is considered that pile penetration to elevation 677 will ensure an adequate margin of safety against possible scouring, particularly in view of the fact that the stream level is now controlled by the dyke located about 200 yards downstream. No major flooding is believed to have occurred at the site since the construction of the dyke.

On page 13 it was shown that, for the estimate of the safe bearing capacity of a short pile foundation, the average standard penetration test N-value can be taken as 45 blows per foot; although some variation in the density was indicated by the cone probes, the departures from this average are considered to be insignificant and

F. ENGINEERING CONSIDERATIONS and
CONCLUSIONS - Cont'd

will be amply covered by the factor of safety incorporated in the pile design.

Employing the standard penetration test results, the bearing capacity of piles can be estimated on the basis of the method suggested by G. Meyerhof (Reference 3), who puts forward the following equations, applicable to cohesionless materials:

Deep penetration into supporting stratum:

$$Q_f = 4 N \cdot A_p + \frac{\bar{N} \cdot A_s}{100} \dots\dots\dots (1)$$

Small penetration into supporting stratum:

$$Q_f = \frac{4 \cdot N \cdot D}{10B} \dots\dots\dots (2)$$

where	Q_f	= failure load per pile, tons
	N	= average N-value below pile group
	\bar{N}	= average N-value along pile shaft
	A_p	= c.s. area of pile toe, sq. ft
	A_s	= total pile shaft area effective in mobilizing skin friction, sq. ft
	B	= breadth, or diameter of pile, ft
	D	= depth of penetration into supporting stratum, ft

F. ENGINEERING CONSIDERATIONS and
CONCLUSIONS - Cont'd

Equation (2) is applicable to conditions when D/B does not exceed 10, and should be adopted in the present case, should the piles be driven not deeper than to elevation 673. If the suggested pile toe elevation 677 is adopted, and with $N = 45$, as indicated above,

$$Q_f = \frac{4 \times 45 \times 6}{10 B} \text{ tons}$$

$$= \underline{108} \text{ tons, for } B = 12 \text{ in., typically.}$$

In view of the generally uniform density of subsoil below elevation 683 established at this site, we consider a factor of safety of two to be adequate; under the above conditions, the safe working load per pile would thus be 54 tons (neglecting the insignificant skin friction).

The allowable load can be calculated in a similar manner for other values of A_p , B and D . For the case of timber piles, the maximum load will probably be determined by the structural strength of the piles themselves.

Judging from the apparently high density of the subsoil and from the fact that fine sand and silt components predominate and clay is present only in the form of thin and isolated seams, the settlement of the pile - supported foundations is likely to be of insignificant magnitude, and the settlement problem is not considered to be a major consideration at this site.

F. ENGINEERING CONSIDERATIONS and
 CONCLUSIONS - Cont'd

ii) Deep piles.

A considerably higher load per pile would be permissible for the case of piles supported either on the bedrock, which in test hole 7 commenced at elevation 614.8, or on the extremely dense sandy and silty till, which in test holes 4 and 7 commenced at elevations 623.3 and 620.6 respectively.

The extremely dense consistency of the fine sand and silt till would indicate that refusal may be reached to pile driving near the surface in this stratum, for most types of piles. Also, the allowable bearing capacity in this stratum would normally be very high, and determined by the design strength of the piles themselves. If satisfactory support could be obtained on the dense till, then there would be no need to risk damaging the piles by an attempt to drive them to bedrock.

However, the consideration is complicated by the presence of ground water under artesian pressure within the fine sand and silt till. The possible upward flow of water under the artesian pressure along any gaps between the piles and the surrounding soil could feasibly cause some loosening of the presently extremely dense structure of the till immediately below the piles, with a consequent partial loss of support and some settlement. However,

F. ENGINEERING CONSIDERATIONS and
CONCLUSIONS - Cont'd

no information, to the best of our knowledge, is available in the soil mechanics literature regarding the behavior of piles under such conditions. While all risks would be eliminated by driving the piles to the surface of the bedrock, the resistance to driving through the sandy till may prove excessive.

c) Construction considerations.

i) Short piles.

With the exception of possible obstructions caused by timber logs, blocks of concrete, etc., which were identified near the bottom of the present embankments and within the uppermost fluvial deposits, the driving of piles would be quite easy down to the approximate elevation 683; below this level, the subsoil becomes dense and the resistance to driving will increase considerably. The results of the cone penetration holes, plotted on the drawing, can serve as a guide to the resistance to pile driving.

ii) Long piles.

The driving of long piles, such as intended to find support on the bedrock or in the deep-seated extremely dense till, would be governed by similar considerations as above, until the piles penetrate below elevation 683. After the piles have entered the dense

F. ENGINEERING CONSIDERATIONS and
 CONCLUSIONS - Cont'd

fine sand and sandy silt deposit, the resistance would show variations similar to those indicated by the cone penetration blows. The material becomes more clayey with depth, so that the resistance to penetration may somewhat decrease in the lower horizons.

While preboring of piles to bedrock would be easier than driving, the holes would have to be lined all the way down, and difficulty would be caused by the ground water under artesian pressure. In order to install deep-seated prebored piles under such conditions, the artesian pressure would have to be temporarily eliminated by the installation of deep pumps, placed immediately above the bedrock surface in special borings, located around the cluster of piles.

iii) Excavation and shoring.

The fluvial deposits extending to the approximate elevation 683 must be considered as pervious, and special precautions will be necessary in order to allow any excavation below water level. The work will probably have to be done within sheet piling cofferdams, driven to a short distance below elevation 683. Below this level, the permeability in the vertical direction can be assumed to be considerably lower than in the overlying deposits.

F. ENGINEERING CONSIDERATIONS and
CONCLUSIONS - Cont'd

The depths to which the cofferdam sheeting should penetrate in order to assure stability of the bottom against uplift should be estimated when the horizontal dimensions of the cofferdam and the maximum depth of excavations are known. It should be assumed that the water pressure differential between the inside and outside of the cofferdam will be dissipated only below elevation 683.

The cofferdam sheet piling will be easy to drive down to elevation 683, with the exception of possible obstructions caused by logs, broken concrete etc., which may be present within the fill and fluvial deposits. Old piles are also a hazard.

Attention is drawn to the presence of the watermain, located in the southern portion of the site.

2. Embankments.

The embankment materials were described on page 10. While presently the embankment slopes are quite steep, reaching 45 degrees in places, we consider that this gradient should be decreased, particularly in view of the proposed raising of grade by approximately 5 feet. Careful drainage measures are necessary, particularly on the western side, where it is believed springs located in the high ground to the west of the site give rise to seepage through the embankments, and cause local instability.

F. ENGINEERING CONSIDERATIONS and
 CONCLUSIONS - Cont'd

In view of the soft consistency of the embankment materials, some settlement must be expected below the weight of the new fill by which the grade is to be raised.

Much broken concrete, including large pieces, is visible at ground surface, particularly on the southern slope of the eastern embankment.

The embankments for the east-bound lanes will be placed over an area which has not been subject to previous embankment loads and where the fluvial subsoil is unconsolidated. Much organic material is present and the consistency of the upper layers of subsoil is generally very soft. It would be advisable to remove as much of the organic material as is possible, with dredging below the water level if practicable; freshly imported fill should be placed and allowed to consolidate before construction of the final road surface. The settlement is expected to develop rapidly because of the high permeability of the subsoil, although some minor long-term secondary movements may continue within any remaining organic layers.

F. ENGINEERING CONSIDERATIONS and
CONCLUSIONS - Cont'd

In view of the variable composition of the subsoil, the quantity and rate of settlement of the embankments are not subject to theoretical analysis. We would recommend that the embankment construction should be commenced well in advance of the bridge construction and carried out in stages, to allow progressive settlement of the foundation materials, without the risk of local failures. If the bulk of the fill is placed a few months before completion of the bridge, the soft subsoil will have a chance to consolidate and the subsequent settlements below the road surface will not be excessive. However, periodic maintenance of the road at the bridge approaches may be necessary for several years after completion of the Project.

Report prepared by:

R. Kulesza

R. Kulesza, P. Eng.

E. M. PETO ASSOCIATES LTD.

C. F. Freeman

C. F. Freeman, P. Eng.,
Chief Engineer.

RK/AP

Our Job Number 63214

28th November 1963.

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Job No. 63214

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2. "The Physiography of Southern Ontario",
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of Cohesionless Soils". G. G. Meyerhof.
Proc. Am. Soc. Civil Engrs., Vol. 82,
No. SM 3, 1956.

TABLE "A"

ATTERBERG LIMIT TEST RESULTS

Job Number 63214

BH/SA No.	Depth ft.	Liquid Limit %	Plastic Limit %	Plasticity Index	In-situ Water Content %
7 / 8	36	36.1	18.2	17.9	35.8
7 / 17	76	23.5	15.6	7.9	9.3

TABLE "B"
STANDARD CONE PROBE RESULTS

Job No. 63214

Cone Hole No.	1	2	3	Blows Per Foot			7	8
Elevation	700.0	700.0	717.4	717.4	700.0	700.0	701.6	706.0
Depth, Ft.								
0 - 1	Creek bottom at 6'-2"	Creek bottom at 4'-6"	Six inch counts in brackets	Creek bottom at 6'-2"	Creek bottom at 5'-0"	2		
1 - 2								
2 - 3								
3 - 4								
4 - 5								
5 - 6	1/34"	1/30"	9 (4, 5)	1/10"	8	10		
6 - 7								
7 - 8								
8 - 9								
9 - 10								
10 - 11	1	2	9	11	2	7	Borehole 8 to 14'-0"	
11 - 12	2	4	8 (5, 3)	21	6	9		
12 - 13	3	3	4 (2, 2)	25	4	8		
13 - 14	18	21	8 (4, 4)	41	4	12		
14 - 15	20	34	9 (3, 6)	37	3	22		
15 - 16	26	30	15	29	4	17	4	
16 - 17	30	44	15 (10, 5)	32	32	16	1	
17 - 18	42	51	15 (7, 8)	39	51	15	9	
18 - 19	33	72	19 (11, 8)	73	71	28	10	
19 - 20	49	52	9 (6, 3)	72	63	44	12	
20 - 21	51	87	6	56	54	36	23	
21 - 22	50	73	9 (3, 6)	45	56	56	40	
22 - 23	45	63	11 (6, 5)	69	71	56	52	
23 - 24	67	70	2	7	90	115	69	
24 - 25	86	80	1	16	120	88	57	
25 - 26	74	114	6	37	85	76	74	
26 - 27	66	91	3	31 (15, 16)	65	76	86	
27 - 28	78	77	6	23 (13, 10)	78	89	93	
28 - 29	68	74	4	22	78	76	72	
29 - 30	70	95	2	74	72	82	102	

TABLE "B" - Cont'd

Cone Hole No.	1	2	3	4	5	6	7	8
Elevation	700.0	700.0	717.4	717.4	700.0	700.0	701.6	706.0
Depth, Ft.								
29 - 30	57	75	14	50	72	69	95	120
30 - 31	86	85	44	31 (17, 14)	75	82	80	172
31 - 32	90	61	61	27 (14, 13)	103	86	73	103
32 - 33	95	61	80	15	103	79	78	82
33 - 34	135	67	77	30	175	111	81	85
34 - 35	132	75	77	82	155	133	114	85
35 - 36	115	111	102	90 (48, 42)	116	142	125	95
36 - 37	97	100	116	73 (43, 30)	88	123	164	83
37 - 38	111	79	114	76	108	132		73
38 - 39	120	54	102	105	105	147		120
39 - 40	99	56	89	96	98	106		186
40 - 41		80	110	104 (62, 42)	87			Softer in
41 - 42		54	139	75 (41, 34)				last 3 in.
42 - 43		56	180	67				
43 - 44		56	136	78				
44 - 45		75	119	75				
45 - 46		80	133	94 (55, 39)				
46 - 47		84	150	65 (31, 34)				
47 - 48		65	127	125				
48 - 49		59	129	103				
49 - 50		56		96				
50 - 51				122 (64, 58)				
51 - 52				109 (60, 49)				
52 - 53				77				
53 - 54				74				
54 - 55				106				
55 - 56				94 (51, 43)				
56 - 57				82 (34, 48)				
57 - 58				73				
58 - 59				79				
59 - 60				90				

TABLE "B" - Cont'd

Cone Hole No.	1	2	3	4	5	6	7	8
Elevation	700.0	700.0	717.4	717.4	700.0	700.0	701.6	706.0
<hr/>								
<u>Depth, Ft.</u>								
60 - 61				80(46, 34)				
61 - 62				77(36, 41)				
62 - 63				60				
63 - 64				64				
64 - 65				63				
65 - 66				81(45, 36)				
66 - 67				83(37, 46)				
67 - 68				129				
68 - 69				98				
69 - 70				100				
70 - 71				98(54, 44)				
71 - 72				113(52, 61)				

TABLE "C"

PROGRAMME OF WORK

Job No. 63214

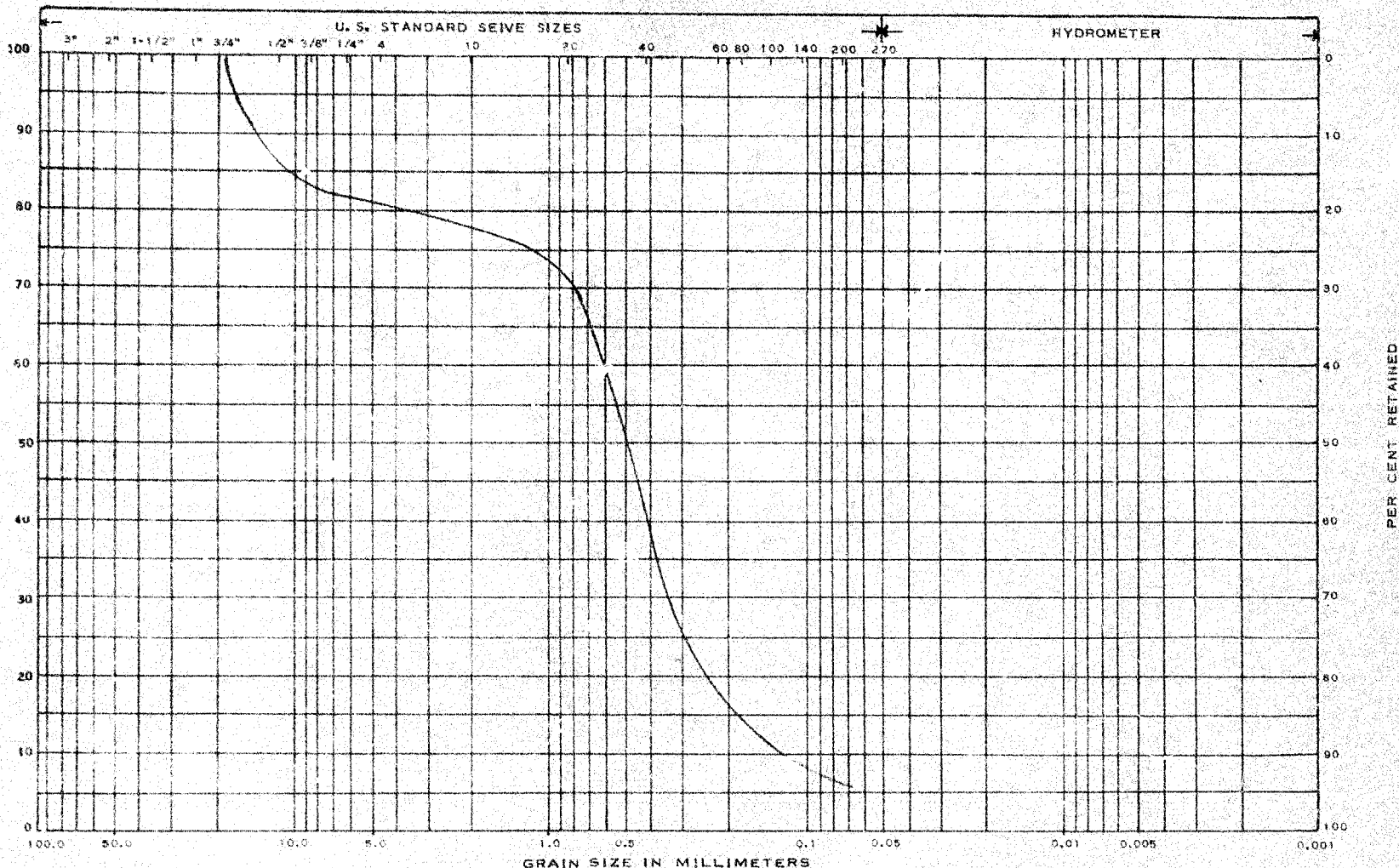
October 21, 1963	Investigation authorized by Client. Our engineer, Mr. R. Kulesza, visited Foundation Section, D.H.O. and received Site Plan and instructions. Data regarding new bridge obtained by telephone from Mr. G. Scott, Bridge Division, D.H.O.
October 30, 1963	Engineer and drilling crew arrived at site at 12 noon. Equipment unloaded. Services cleared, water main staked out. Test hole 7 put down to 26.5 ft.
October 31, 1963	Test hole 7 put down from 26.5 ft to 81.7 ft. Drizzle most of afternoon.
November 1, 1963	Rain prevented field work. Engineer visited D.H.O., and consulted with Mr. K.Y. Lo, Foundation Division, regarding available results and future programming of the field and laboratory work.
November 4, 1963	Test hole 7 chopped through very hard material to 85 ft. AX casing reamed into bedrock to 87 ft.
November 5, 1963	Test hole 7 diamond drilled from 87 ft to 93.9 ft. Rain prevented work in the afternoon.
November 6, 1963	Cementing off artesian pressure in test hole 1 and pulling casing. Standard cone probe driven to 37 ft near test hole 7. Raft assembled for work on river.
November 7, 1963	Raft aligned in position of hole 6 and standard cone probe driven to 40 ft. Raft aligned in position of hole 5 and standard cone probe driven to 41 ft.

TABLE "C" - Cont'd

November 8, 1963	No work done because of rain.
November 11, 1963	Raft aligned in position of hole 2 and standard cone probe driven to 50 ft. Raft aligned in position of hole 1 and standard cone probe driven to 40 ft.
November 12, 1963	Test hole 8 put down to 14 ft. Standard cone probe driven in hole from 14 ft to 40 ft. Test hole 3 put down to 21.5 ft. Standard cone probe driven in hole from 21.5 ft to 49 ft.
November 13, 1963	Test hole 4 put down to 61.5 ft.
November 14, 1963	Test hole 4 put down from 61.5 ft to 95 ft. Artesian water pressure cemented off between 90 and 95 ft. Casing withdrawn. Standard cone probe driven near test hole 4 to 49 ft.
November 15, 1963	Standard cone probe driven near test hole 4 from 49 ft to 72 ft. Hand auger test hole 9 put down to 3.8 ft. Elevations of holes measured. Field work completed and equipment loaded. Engineer and drilling crew departed from site at 2:00 P.M.
November 21, 1963	Engineer visited D.H.O., discussed the final results with Mr. K.Y. Lo and obtained advice on form of presentation of data.
November 29, 1963	Soils Report submitted to Client.

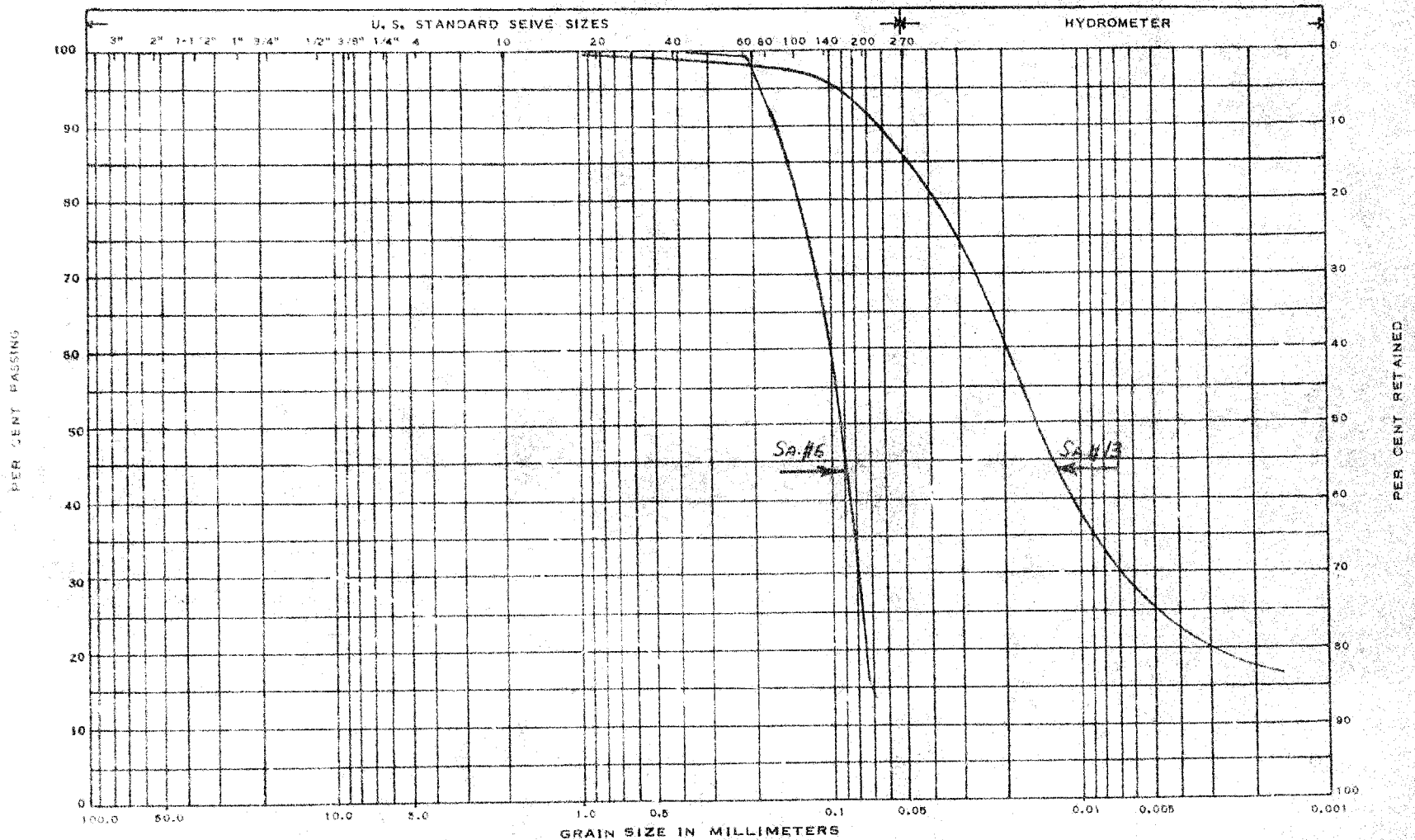
e. m. peto associates ltd.

Toronto 19, Ontario



e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

W. P. 138-63 Hwy. #3

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Big Creek Bridge at Delhi

JOB NO. 63214

HOLE NO. 7

SAMPLE NO. 6, 13

SA #6

DEPTH 25'6" ELEVATION 676

REMARKS Interlayered fine silty sand and silt, some clay seams.

SA #13

57'6"

644

GRAIN SIZE DISTRIBUTION

FIG 1b

16876024 - BIG CREEK BRIDGE AT DELHI

OEDOMETER CONSOLIDATION TEST

Borehole 7, So. #12, Depth 56 ft

Grey Clay

(Typical clay layer in dense fine silty sand and silt stratum)

Initial water content 36.1%

" bulk density 114.8 pcf

LOAD STAGE	COEFFICIENT OF CONSOLIDATION C_v
Ton/sq ft	Sq. ft./year
1/8 - 1/4	25.6
1/4 - 1/2	41.9
1/2 - 1	41.8
1 - 2	50.6
2 - 4	38.4
4 - 8	36.5

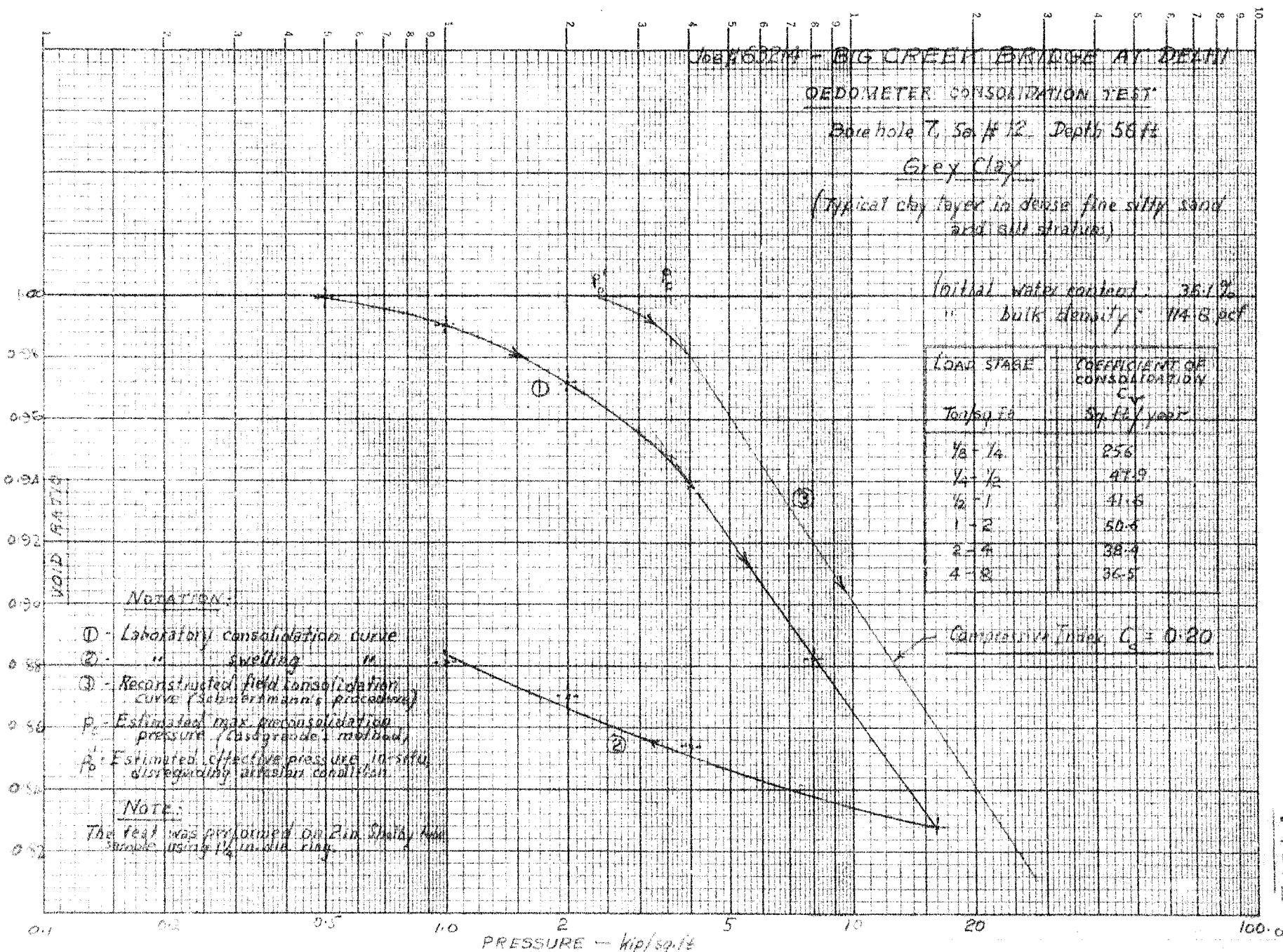
Compressive Index $C_c = 0.20$

NOTATION:

- ① - Laboratory consolidation curve
- ② - " swelling "
- ③ - Reconstructed field consolidation curve (Schobermann's procedure)
- P_c - Estimated max. preconsolidation pressure (Casagrande's method)
- P_0 - Estimated effective pressure in situ disregarding dilation condition

NOTE:

The test was performed on a 2 in. Shelby Tube sample using 14 in. o.d. ring.

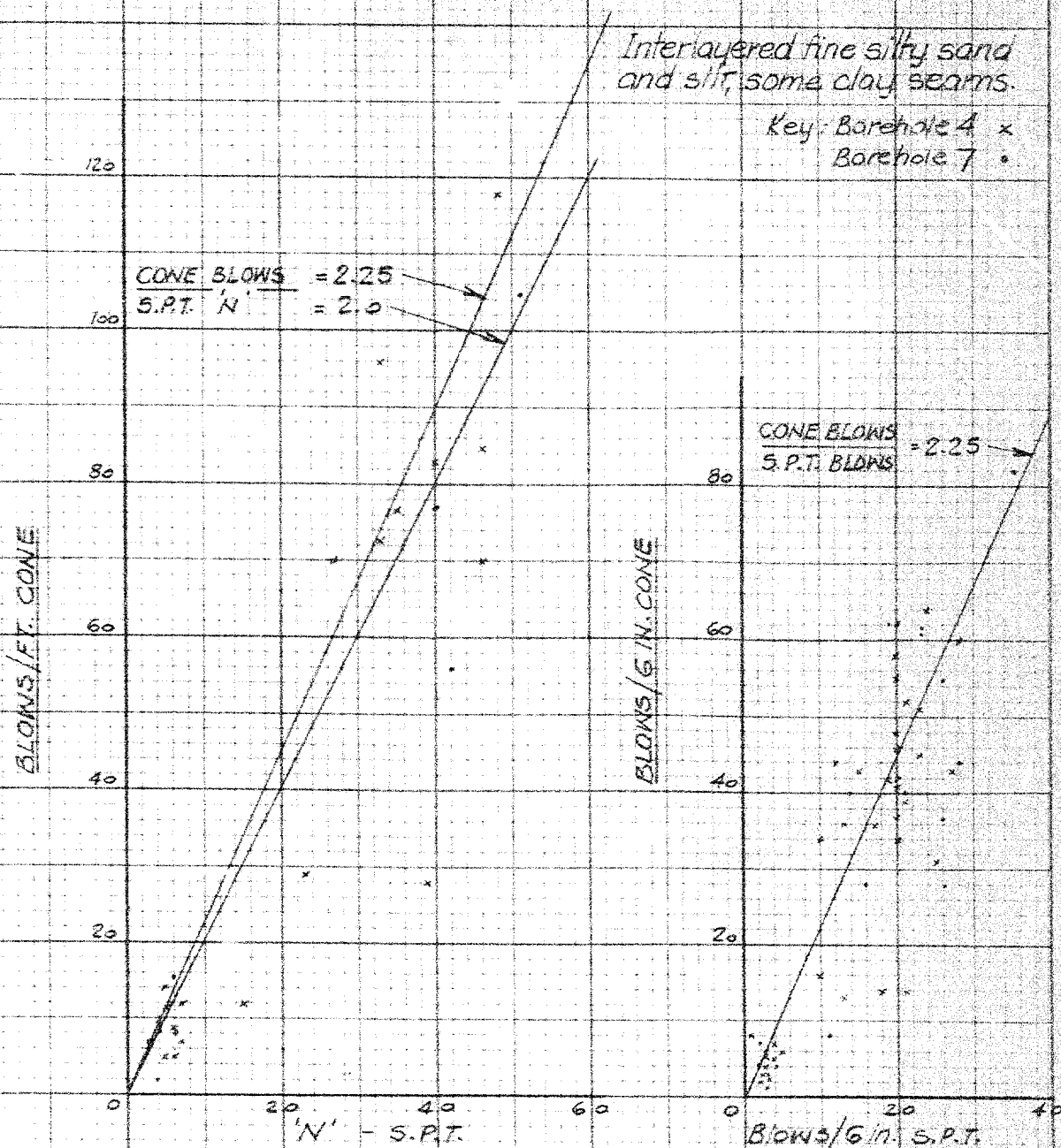


BIG CREEK BRIDGE DELHI

Job #63214
e.m.peto associates, ltd.

Interlayered fine silty sand
and silt, some clay seams.

Key: Borehole 4 x
Borehole 7 •



(a) Last 12 in. (S.P.T.) plot

(b) 6-in. increment plot

Fig. 3 RELATIONSHIP BETWEEN STD. PENETRATION TEST
RESULTS IN BORE HOLES AND CONE PROBE
RESISTANCE AT SAME LEVEL IN ADJACENT CONE HOLES.

BOREHOLE LOG

Checked By S. B.

[illegible]

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

W. P. 138-63 Hwy. #3

Borehole No. 8

Boring Date November 12, 1963

Checked By S. B.

ABBREVIATIONS

Y.T.	IN SITU VANE SHEAR TEST
M.	MOIST
W.L.	WATER LEVEL IN CASING
W.T.	GROUND WATER TABLE IN SOIL
W.T.P.L.	WETTER THAN PLASTIC LIMIT
D.T.P.L.	DRIER THAN PLASTIC LIMIT
A.P.L.	ABOUT PLASTIC LIMIT

[illegible]

BOREHOLE LOG

W. P. 138-63, Hwy. #3

Job Name Big Creek Bridge at DelhiJob No. 63214Borehole No. 9Client Dept. of Highways - OntarioCasing None (Hand-auger boring)Boring Date November 15, 1963Elevation 701.5Compiled By R. K.Checked By S. E.

SAMPLE CONDITION



UNDISTURBED



FAIR



DISTURBED



LOST

SAMPLE TYPE

A.S. AUGER SAMPLE

C.S. CASING SAMPLE

S.S. 2" STANDARD SPLIT TUBE SAMPLE

S.L. SPLIT BARREL WITH LINERS

S.T. THIN-WALLED SHELBY TUBE SAMPLE

W.S. WASH SAMPLE

R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST

M. MOIST

W.L. WATER LEVEL IN CASING

W.T. GROUND WATER TABLE IN SOIL

W.T.P.L. WETTER THAN PLASTIC LIMIT

D.T.P.L. DRIER THAN PLASTIC LIMIT

A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION

COLOUR

Density or
ConsistencyDepth
Elevation

Legend

Sample No.
and
ConditionSample
TypeNo. of
Blows
per Ft.Natural
Moisture
Content

WATER LEVEL & REMARKS

Decayed vegetation

Brown

V. soft

0'0"
0'5"

No samples taken

Silt with some fine sand & a
high organic content, sandy silt

Grey-black

Soft

2'3"
2'8"

Clayey silt, or very silty clay

Ditto

Ditto

3'10"



Obstruction at 3'10"

(probably large wood fragment)

Test hole terminated at 3'10" due to obstruction
and caving (fast inflow of water from bottom of test hole)

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario,
RU 9-1126.

Our Job Number 63214

15th December, 1963.

Department of Highways of Ontario,
Materials and Research Division,
Foundation Section,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. A. G. Stermac,
Principal Foundation Engineer

Gentlemen:

Re: W. P. 133-63, Highway #3,
Big Creek Bridge at Delhi,
District #2

Further to the discussion between Mr. K. Y. Lo and our engineers which took place in your office on 9th December, 1963, we respectfully submit the following comments, relating to the matters raised at this meeting and concerning our Soils Report No. 63214 on the site investigation for the above Project.

A. Sheet pile protection of pier and abutment excavations

According to the additional information which we have obtained from the D. H. O. Bridge Section, the inverts of pile-supported footings will probably be located at a level approximating to the present lowest position of the creek bottom, which is between elevations 688 and 690. Assuming that the creek surface during construction will be at

A. Sheet pile protection of pier and abutment excavations - Cont'd.

the recently measured elevation 700 (which is controlled by the dyke located about 200 yards downstream), the difference in water levels between the inside and the outside of the sheeting would unlikely need to exceed 15 ft., and may be less.

In order to provide data for the design of the sheeting, we have calculated the theoretical depth of penetration of the sheeting for a range of possible water heads and desired factors of safety against a bottom heave failure. The data is presented in graphical form on the enclosed Figure 4, and covers the possible range of dimensions of the excavation and configuration of the subsoil profile. The figure contains a key diagram and a list of assumptions which make the data self-explanatory. The critical consideration for the range of dimensions was found to be the stability against "heaving" of bottom of excavation, as determined by the hydraulic gradient along the inside of the sheet piles. The safety against "piping", as determined by the hydraulic exit gradient at the bottom of the excavation, was found to be less critical.

If it is required to lower the water level inside the sheeting to elevation 688, for a 12 ft. head of water, stability would theoretically be assured (F=1) with an 11 ft. penetration of sheeting below elevation 698, that is to elevation 677. For a factor of safety above unity, deeper

A. Sheet pile protection of pier and abutment excavations - Cont'd.

penetration would be necessary as indicated on Figure 4. Elevation 677 actually corresponds to the least recommended depth of penetration of the piles which are to support the bridge.

We would like to suggest that consideration be given to lowering the water level in the creek prior to construction. The dyke located downstream of the site is about 12 ft. high, so that complete diversion of water past the dyke would lower the level of water at the site to near, or even slightly below the probable level of the footings, which would, of course, greatly simplify the construction. For a project of this size, the suggestion would seem justifiable and would also facilitate construction of the embankments.

While no ill-effects are expected downstream of the dyke if the retained water is emptied at a slow rate, the possible results upstream of the site would have to be thoroughly investigated. In order not to cause any bank failures, the rate of drawdown should not exceed six inches per day.

Should it not be feasible to lower the water level by as much as by-passing the dyke would allow, a partial advantage would be gained by lowering the level by the maximum permissible height.

B. Foundation of new embankments

After reconsidering the available evidence, we still feel that sufficient information has been collected relating to this aspect of the project, and that additional field work is not justified, bearing in mind that the final depth of excavation will be of necessity decided in the field. To justify this opinion, we would like to state the evidence at greater length than was included in the report.

1. The boreholes have indicated that the subsoil, both within and underneath the embankments, is mainly cohesionless and as such offers sufficient resistance against shearing to assure stability of the enlarged sections. Clay was found only in the form of thin seams, interlayered with sand. Organic matter, apart from the organic debris forming the surface of the marshland, had mainly the form of wood fragments, which are not considered very detrimental to stability. In this respect, the occasional layers of organic silt are more critical, but these were quite thin and interlayered with sandy material.
2. The marshland to the south-west of the western abutment has been previously explored by a grid of soundings, the positions of which are shown on Client's Survey Plan E - 4301, W.P. 138-63. The probes extend to the limits of the survey shown on this Plan, and we would

B. Foundation of new embankments - Cont'd.

suggest checking whether this exploration was not extended even further west, of which we have no data. The soundings had a depth of 5 to 9 feet and the results are stated on the Survey Plan as "Loose sand and clay to hard sand". Thus, no evidence of major organic deposits is indicated.

We felt that further testing of the marshland area, other than by our shallow hand-auger hole No. 9, was not warranted as it would duplicate work that had been done already.

3. Regarding the composition and foundation of the existing embankments, we considered Boreholes 3 and 4 as sufficiently representative. Close scrutiny of material was conducted, including split-spoon samples at 2.5 ft. intervals to a level at least 5 ft. below the bottom of embankments, and the samples were described in detail on the borehole logs appended to our report. The embankment material, similarly as the foundation, was mostly silt and fine sand, of soft consistency but nevertheless of cohesionless character. Again the organic inclusions were mainly fragments of wood and random but thin layers of organic silt.

4. It is obvious that the embankments were built up over the years without much care to clear the organic debris and indiscriminately using inferior fill materials. There is no reason to believe that this

B. Foundation of new embankments - Cont'd.

pattern is much different at other sections of the embankments, although variations in the materials may occur at random. Additional testholes through the embankments are unlikely to supply much new and useful information and were not considered necessary.

5. Judging from the composition and consistency of both the embankment fill and the foundation soils, we are of the opinion that in both cases the material can be considered as cohesionless, and an average angle of shearing resistance of at least 30° can be relied upon to resist potential slip failures.

The present embankments have an average slope of 2:1, horizontal to vertical, and are even steeper in places. With these slopes, they appear stable, the only signs of erosion having been observed on the slopes facing the creek, adjacent to the abutments, but there the slope approaches 1:1 and the erosion apparently has been caused by water seepage.

As it is assumed that at least two feet of the organic topsoil forming the present marsh surface will be stripped prior to placement of the new embankment, the foundation conditions will presumably be superior to those of the present embankments. Also, the new fill will undoubtedly be of better quality. Thus, there appears no reason why the

B. Foundation of new embankments - Cont'd.

new embankment should be less stable than the existing one.

However, in view of the proposed raising of grade by about 5 ft., we would recommend that the slopes on both sides of the final embankment should be not steeper than 2:1. Also, it would be advisable to include a freely-draining granular cushion immediately above the stripped grade and also between the existing slopes and the new fill. This would reduce a build-up of pore water pressures in the new fill, and intercept seepage which we believe percolates through the present embankments from the elevated areas on top of the valley. A two-foot thickness of the free-draining filter should be sufficient. This measure would have the additional advantage of accelerating the consolidation of the subsoil beneath the embankments and facilitating the dissipation of pore water pressures set up by the weight of the new fill.

6. We considered that the depth to which the surficial, organic material should be stripped prior to placement of the new fill would be governed by the practical considerations, and we recommended that the stripping should extend at least to the level of the water table, equal to the creek level at elevation 700. Although deeper stripping would be desirable in order to limit settlement of the embankments, difficulty with excavation below the water level and compaction of the new fill may

B. Foundation of new embankments - Cont'd.

make this impractical. For this reason, we would prefer that the embankment materials should be placed gradually, rather than in one continuous operation, so that the subsoil could consolidate and any excess pore water pressures would dissipate. If the fill could be placed well in advance of completion of the bridge structure, the amount of subsequent settlement would be reduced and less maintenance of road surface would be needed after completion of the project. This we consider to be a normal good practice measure, particularly on marshy and possibly variable foundation and when an existing embankment has to be widened.

However, should it not be possible to build up the embankment gradually, and well ahead of the completion of the bridge, from the previous arguments it follows that no potential slope failure is anticipated, particularly if the recommendations of stripping at least to the water level and inclusion of free-draining blankets are followed. The main disadvantage would be the larger amount of settlement after completion of the project, necessitating more regrading of road surface. In order to prevent local slippage at the toe of the fill, we recommend that the fill should preferably be placed commencing with the toe of the embankment and working inward.

B. Foundation of new embankments - Cont'd.

7. Reconstruction of the embankment would be considerably facilitated if the water level in the creek was lowered, as suggested in section A. This would allow a greater depth of stripping of the organic and variable topsoils and a more sound embankment construction commencing with the foundation. The depth to which it would be advisable to excavate would probably vary and should be decided on site after examination of the excavated grade; the testholes have indicated, however, that only minor organic content is present below elevation 695, which may prove to be the lowest level to which stripping would be useful. Under these conditions, a 2:1 slope could be allowed.

Since a greater depth of stripping would mean lesser embankment settlement, it would be less important to attempt to construct the embankment in advance of the completion of the bridge.

After considering these views, perhaps you would wish to hold a further discussion.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,

Report Prepared by:

R. Kulesza

R. Kulesza, P. Eng.

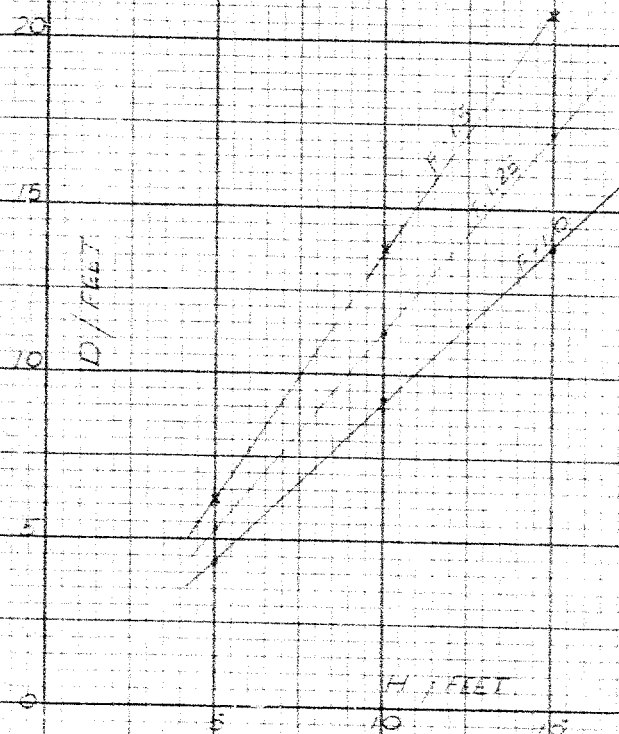
RK/vm

C. F. Freeman

C. F. Freeman, P. Eng.
Chief Engineer.

Our Job No. 63214

RELATIONSHIP BETWEEN WATER HEAD AND DEPTH OF PENETRATION OF SHEETING



SYMBOLS

- B = WIDTH OF SHEETED EXCAVATION
 H = WATER HEAD
 D = PENETRATION OF SHEETING BELOW EXCAVATION
 d = DIFFERENCE IN SOIL SURFACE LEVELS ON BOTH SIDES OF EXCAVATION
 γ = SUBMERGED DENSITY OF SOIL
 K_H = HORIZONTAL PERMEABILITY
 K_V = VERTICAL PERMEABILITY
 F = FACTOR OF SAFETY

ASSUMPTIONS

- 1) $8' \leq B \leq 20'$
- 2) $0 \leq d \leq H$
- 3) $\gamma = 50 \text{ lb/cu ft}$
- 4) $1 \leq K_H, K_V \leq 10$
- 5) HEAVING IS THE CRITICAL CRITERION (HYDRAULIC GRADIENT AT TOP OF SHEET PILING CONSIDERED)
- 6) BASED ON DATA IN "ANALYSIS OF SEEPAGE PROBLEMS" BY L. H. HARR & ROBERT C. DREW, BUREAU OF RESEARCH, U.S. ARMY

JOB # 63214

E.T. Peto Associates Ltd.

DEC. 1963

K.K.

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Materials and Research Division

October 21, 1963

G. M. Peto Associates Ltd.,
1237 Caledonia Road,
Toronto 19, Ontario.

Attention: Mr. E. M. Pata

Re: N.P. 133-63, Hwy. #3,
Big Creek Bridge at Delhi,
District #2.

Dear Sir:

Please consider this your authority to carry out a foundation investigation at the above site. Plans and profiles were provided to your representative this date.

It is understood that a qualified soils Engineer will be in charge of the field work at all times.

Ten copies of the completed foundation report with one additional copy of each subsoil profile, should be submitted to the Foundation Section prior to December 3, 1963. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawing accompanying the foundation report, showing the location of borings, the inferred subsoil conditions, etc., is to become one of the contract drawings, you are requested to prepare it in accordance with the D.H.C. standards. To enable you to do this, we are enclosing a sample drawing with all the necessary explanations, together with a linen sheet for your drawing. You are also requested to provide the D.H.C. with a Cronaflex copy of the drawing.

Charges for the work performed will be in accordance with your Schedule of Rates, dated: July 7, 1958, and invoice to be addressed to the attention of the undersigned.

RM/MAF
Encls. (2)

Yours very truly,

A. Putka
A. Putka,

MATERIALS & RESEARCH ENGINEER

cc: Messrs. L. McCombie
A. Carter
H. C. Bernier
J. Roy
B. D. Smith (2)

Mrs. E. Tate

Foundations Office

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attn: Mr. S. McCombie

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

December 19, 1963

FOUNDATION INVESTIGATION REPORT BY:
E. M. PETO & ASSOCIATES, LIMITED -
Big Creek Bridge at Delhi, Hwy. #3,
W.P. 138-63, District #2, London.

The above report submitted by E. M. Peto & Associates,
has been reviewed.

Following a subsequent discussion on several points
of the report, the Consultant submitted a letter with additional
information, which is included herein.

We believe that the factual data and recommendations
contained in this report, will suffice for your future design
work. However, should there be any queries concerning this
project, please do not hesitate to contact our Office.

KYL/ndef
attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
A. Oster
H. C. Dernier
J. Roy
A. Watt

Foundations Office
Gen. Files

Kyle
K. Y. Lo,
SUPERVISING FOUNDATION ENGR.,
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

Mr. G. Scott,
Regional Bridge Location Engr.,
Bridge Division.

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

Attention: Mr. N. Zoltay

June 4, 1964

Review of Preliminary Plan D 5440-P1 -
Big Creek Bridge - W.P. 138-63,
Hwy. #3, District #2.

We have reviewed the above-mentioned Preliminary Plan and submit the following comments:

(1) The designer seems to have followed the general recommendations contained in the foundation report.

(2) Piles should be selected on the basis of economy. It would appear that 12 $\frac{1}{4}$ " O.D. steel tubes would be the most suitable.

(3) Your attention is drawn to the recommendation that as much new fill as possible be placed prior to construction of the bridge.

(4) It might be advisable to draw the Contractor's attention to the Soil Consultant's comments on the use of sheet piling for dewatering purposes.

If you have any further queries in connection with this matter, please contact this Office.

KGS/MdeF

cc: Foundations Office
Gen. Files

K. G. Selby
K. G. Selby,
SENIOR FOUNDATION ENGR.,
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario.

DATE: June 2, 1964.

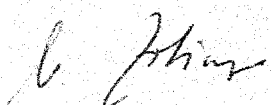
OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 138-63 Br. Site # 21-59,
Big Creek Bridge,
Highway # 3 District # 2.

We are sending to you herewith two prints of
Preliminary Plan D-5440-P1 of the above structure.

Would you please let us have your written
comments.



NZ/kd
c.c. S. McCombie
G. Scott
N.D. Smith

N. Zoltay,
for G. Scott,
Regional Bridge Location Engineer.

Bridge Division,
Downsview, Ontario,
October 15, 1965. 128-63

MEMORANDUM: To File

RE: D.H.O. Contract 65-125,
Big Creek Bridge at Delhi,
Hwy. 3, District No. 2,
Tube Pile Driving

On 22 September 1965, the meeting was arranged between Mr. W. O'Dell, District Construction Supervisor, and myself to inspect and discuss soil condition prior to driving falsework piles and placing falsework pads. Upon arrival at the bridge site Mr. W. O'Dell informed me that the steel tube piles were being driven for the east pier and that there was some difficulty in obtaining required penetration for the pile design load.

In accordance with the bridge plans the piles in east and west pier were to be driven to a depth of 15 feet for the design load of 55 tons per pile.

At the time of my arrival at the bridge site one row of piles in east pier was driven to a depth of 29-30 feet and the penetration recorded was 22 blows/foot. (1.83 blows/inch) This driving resistance was the maximum obtained during the course of driving. When the check was made with the Hilley formula it was found that for the measured rebound and penetration and for the pile load of 165 tons (using $S.S. = 3$) the driving resistance of minimum 4 blows/inch was required.

From subsequent study of borehole locations (Bridge Drawing No. D 5440-2) it was observed that the soil was consistent for a considerable depth and it was then decided to drive one pile to refusal. The pile was driven to a depth of 46 ft. before the driving became stiffer and at 47 ft. the penetration reached 55 blows/foot (4.6 blows/inch). At this point the driving ceased as the penetration was considered adequate.

Next day I reported the driving of piles to Mr. K.Y. Lo, Supervising Foundation Engineer, with the request for steps to be taken in further driving. I also pointed out to him that the contractor will have to stop driving operations if no instructions were issued. Mr. Lo reviewed the Soil Report and advised that in order to follow the recommendations of the Soil Report, the remaining piles in east pier footing should be driven to a depth of 15 feet. These piles would be left in place overnight and re-driven for about 3" next morning.

That morning 24 September 1965, Mr. K. Selby and myself were present at the bridge site when one pile was re-driven with no marked improvement in driving resistance. After further study of the Soil Report it was concluded that the material contains more clay than indicated and therefore it would be safer to drive all piles in east pier footing to practical refusal.

The District was also advised to drive one row of piles in the west pier footing to a depth of 14 feet and re-drive 3 piles after minimum 12 hour period. It was hoped that the piles would stiffen up and be left in at the depth indicated on the bridge plan, thus eliminating the need of driving longer piles.

Two piles were driven on Monday 27 September in each corner of the pier footing and re-driven next morning. Following results were recorded:

- 1) Pile in north west corner driven 1" penetration 9 blows/inch.
- 2) Pile in south west corner driven 2" penetration 5½ blows/inch.

It was noted that pile in south west corner deviated during driving, apparently caused by the presence of old timber piles.

Due to the fact that only two piles were re-driven and with variable penetration results, it was difficult to judge whether the piles could be relied upon to provide the desired load capacity if driven to 15 feet depth. If more time had been available to properly set and test one or two more piles, considerable saving might have resulted in the length of piles. However, not to delay the work, the District proceeded with driving the piles to practical refusal at a depth of between 40 to 50 feet.

KL/ag

K. Lucka,
Bridge Construction Liaison Engineer.

OVER

E. M. PETO ASSOCIATES LTD.
SOIL ENGINEERS

ATTENTION MR. A. RUTKA

1287 Caledonia Road,
Toronto 19, Ontario

789 - 1126

WORK ORDER CLIENT COPY

Date **29th October, 1963** Our Order No. **63214**
Client **Department of Highways - Ontario,** Client's Order No. **Letter**
Foundation Section, Materials & Research Div., Dated **Oct. 21/63**
Toronto, Ont.
Job Name **Crossing at Big Creek and King's Highway No. 3** Location **Delhi, Ont.**
Operator **Gus Zailis** Engineer **Site**
Reports **10** copies to **Client** Office **R. K.**
address **as above** Invoices
copies to Charges
address

File Copy

OPERATOR TO NOTE

EX HOLES , DEPTHS: **4 to minimum depth of 40 ft. ;**
4" HOLES , DEPTHS: **the number of holes and depth may vary, depending on**
site and soil conditions.
2' S.S. SAMPLES AT **2-3', 5-6', 7-8', 10-11', 12-13', 15-16' and at 5' intervals and**
as directed by engineer

ADDITIONAL SAMPLES AS FOLLOWS:

CASING (PAC) SAMPLES AT
2" BRASS LINERS AT **As directed on site by engineer**
3" BRASS LINERS AT **All S.S. samples**
MOISTURE CONTENT TESTS AT **As far as practicable**
HOLES DRY TO
CHECK WATER LEVELS: a) Daily All Holes **as directed by engineer**
b) After pulling casing c) Before pulling casing
d) At timed intervals after bailing

Note specifically all changes i.e. colour, material, density (softening or sufficient)

WORK HOURS / DAY: PUMP REQUIRED **Yes** HOSE FOOTAGE:
SPECIAL REMARKS:-

This work will be carried out under the direct supervision of the engineer,
who will decide depth of holes and details of sampling procedure.

A standard cone will be driven near each testhole, with additional cones
as and where directed by engineer.

FIELD ENGINEER TO NOTE:

1. SET OUT: Yes LEVEL: Yes TEST HOLES: Min. of 4
2. YOU SHOULD BE ON SITE AT: with crew AND REMAIN FOR: Duration of field work
3. PAY SPECIAL ATTENTION TO:

REPORT ENGINEER TO NOTE:

4. CONTACT MR. K. Y. Lo AT: 248-3282
Galvin Scott 248-3506
ON COMPLETION OF:
5. VERBAL REPORT BY: COMPLETED REPORT BY: 30 Nov. /63 latest
6. ESTIMATED FIELD TIME: IN HOURS: To be decided by engineer on site.

GENERAL INFORMATION

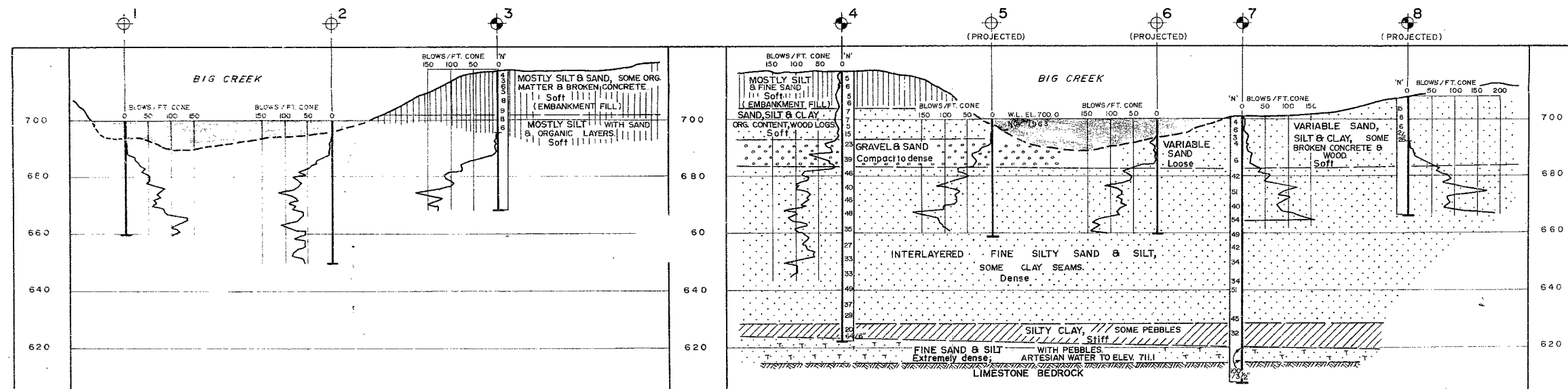
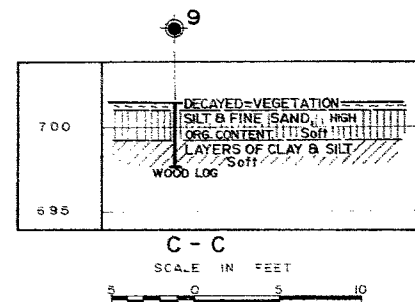
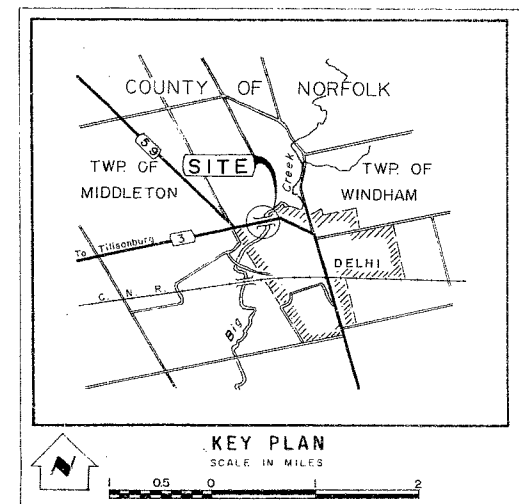
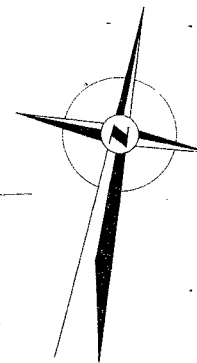
The proposed bridge may be a 3-span structure, with 50-60-50 ft. spans. If possible, a continuous span design will be adopted, but the superstructure will depend on the subsoil conditions.






The existing bridge is supported on piles, From geological information, a deep sandy stratum is anticipated.

A comprehensive but concise report is required by the Client.

@ Previous requirement as to preliminary borehole information and laboratory testing program are to be followed. "

#63-F-209-C
W.P. #138-63
HWY. #3 & BIG
CREEK BRIDGE



LEGEND			
	Bore Hole		
	Cone Penetration Hole		
	Bore & Cone Penetration Hole		
	Water Levels established at time of field investigation.		
	Auger Hole		

NO.	ELEVATION	STATION	OFFSET
1	700.0	28+30	37' Rt.
2	700.0	27+56	41' Rt.
3	717.4	27+00	42' Rt.
4	717.4	29+10	12' Lt.
5	700.0	28+58	33' Lt.
6	700.0	27+99	32' Lt.
7	701.6	27+68	15' Lt.
8	706.0	27+09	24' Lt.
9	701.5	29+34	55' Lt.

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION

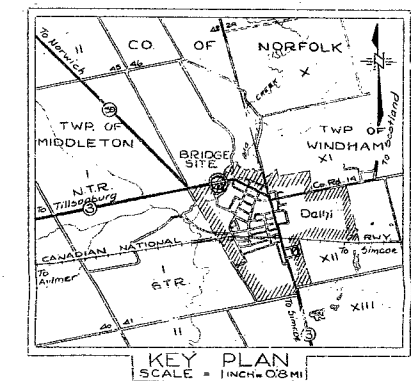
BIG CREEK BRIDGE AT DELHI

KING'S HIGHWAY NO. 3 DIST. NO. 2
CO. NORFOLK
TWP. MIDDLETON LOT 47 CON. I NTR.

SUBM'D	CHECKED	W.P. NO. 138-63	W BR DRAWING NO
DRAWN	CHECKED	JOB NO.	
DATE		SITE NO.	BRIDGE DRAWING NO
APPROVED		CONT. NO.	

[illegible]

CON. I. N. T. R.
LOT 47.



SKETCH SHOWING PROPOSED LOCATION OF BRIDGE A₂
SUBMITTED FOR FOUNDATION INVESTIGATION.
SEPTEMBER 24TH 1963.

W. P. 138-63

DATE	REVISIONS & ADDITIONS	BY	CHK'D.

DEPARTMENT OF HIGHWAYS - ONTARIO
PLANNING & DESIGN BRANCH.
SOUTH - WESTERN REGION.

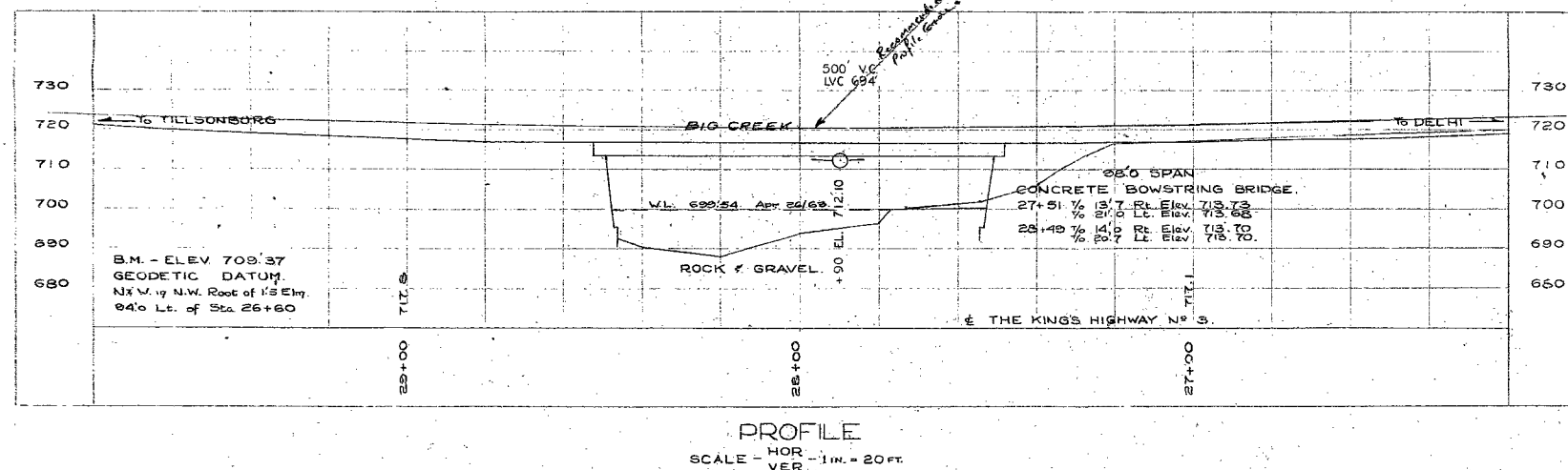
DISTRICT N^o 2.

CROSSING
AT
BIG CREEK
AND

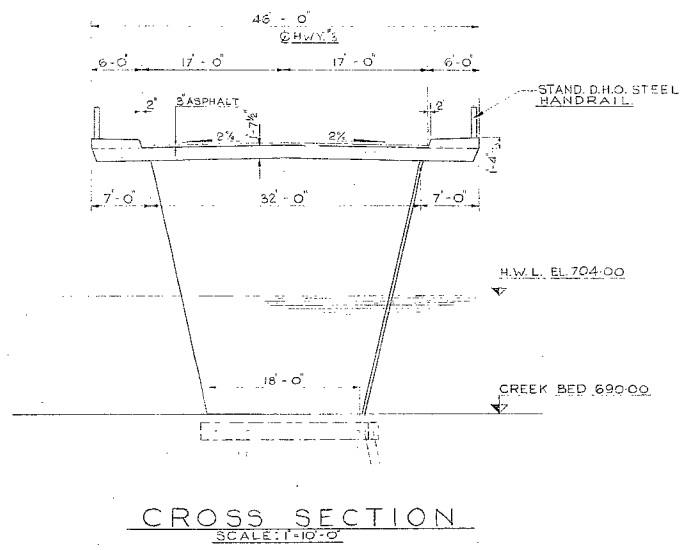
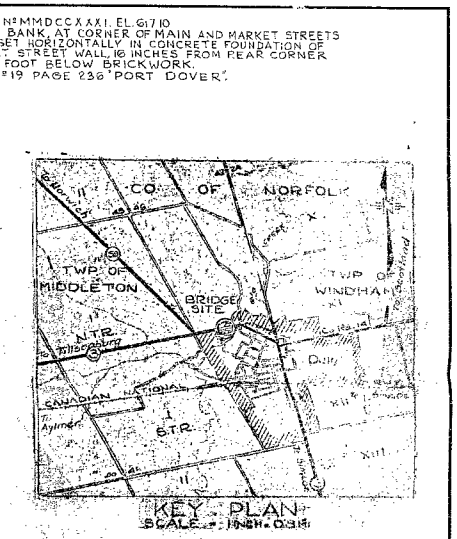
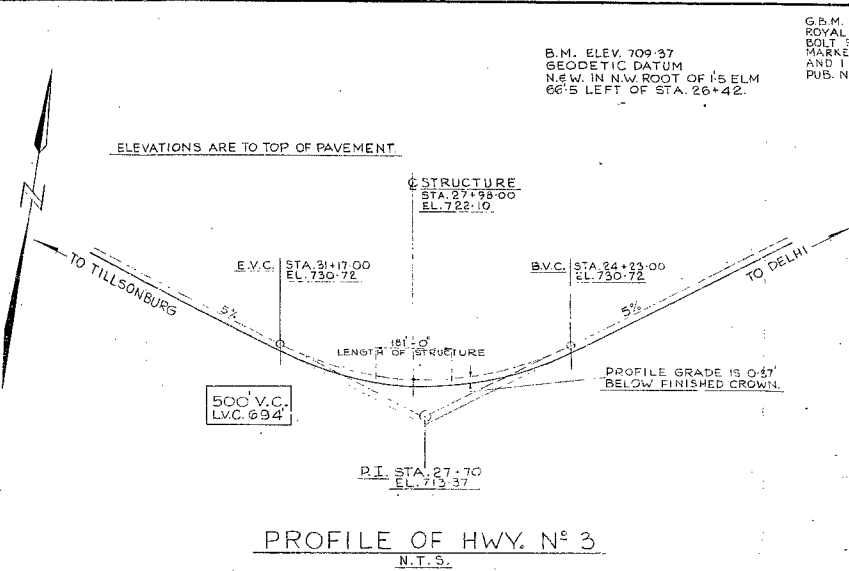
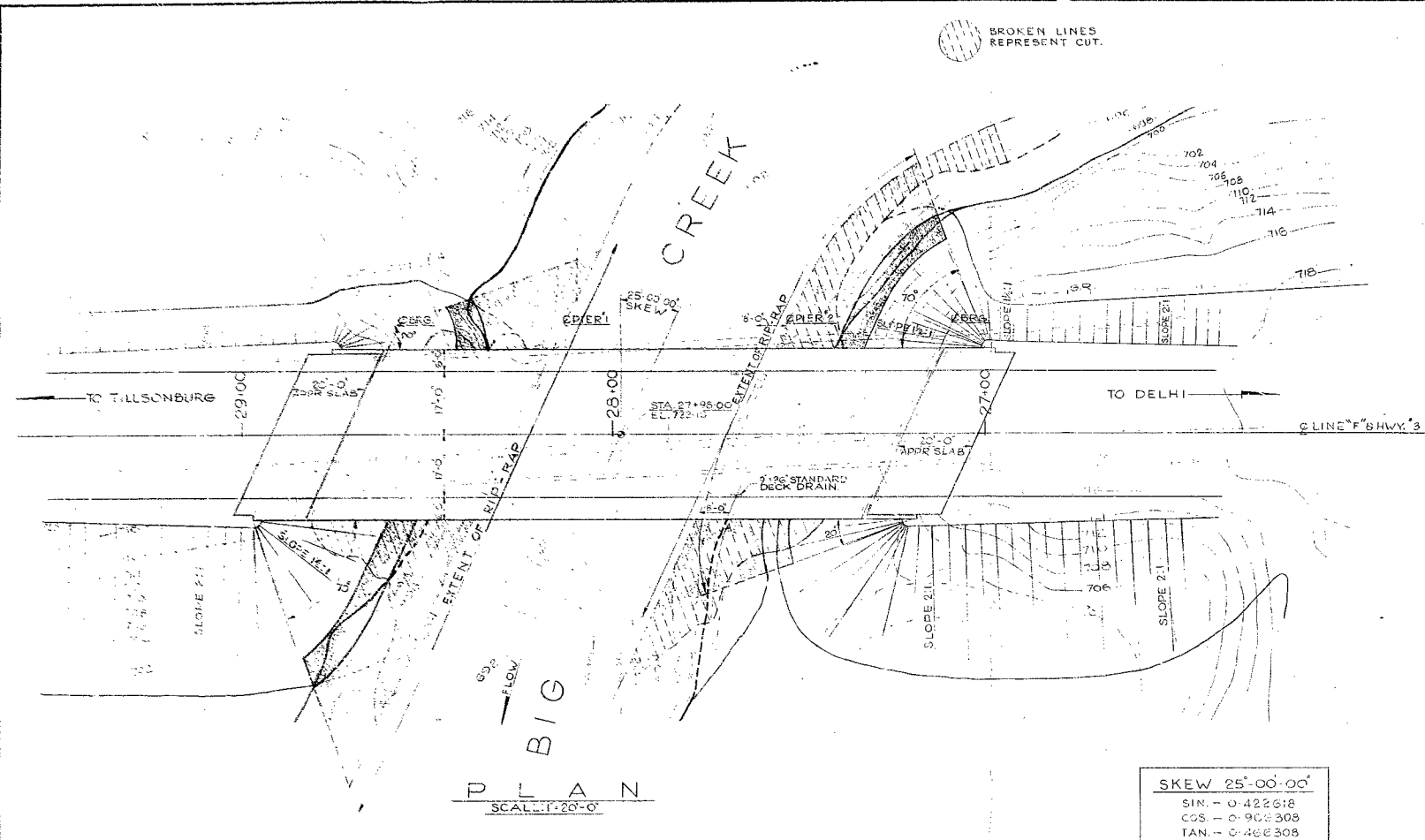
THE KING'S HIGHWAY NO 3
THE TOWN OF DELHI & CONJ.N.T.R. LOT. 47
TOWNSHIP OF MIDDLETON COUNTY OF NORFOLK

BRIDGE SITE

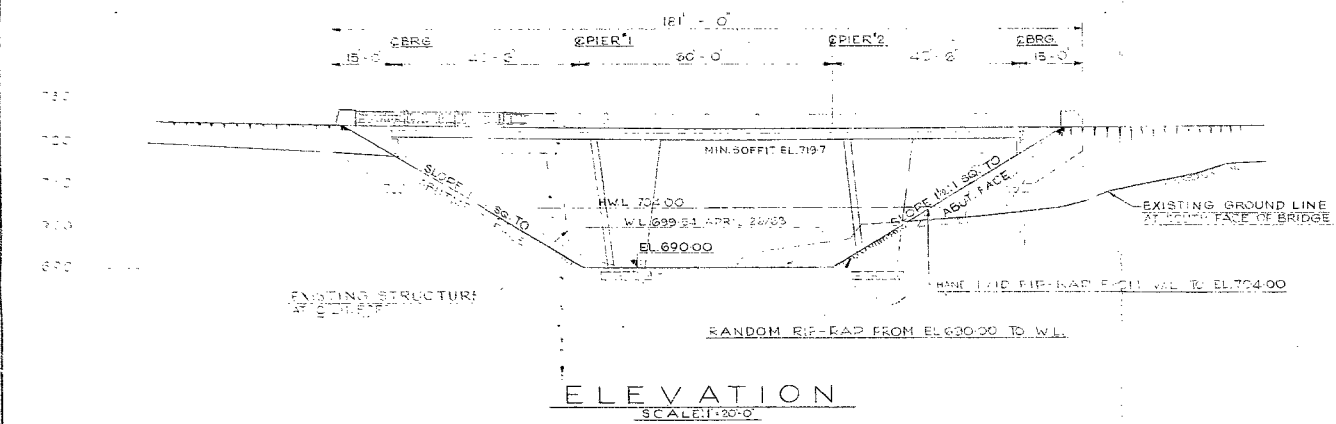
SURVEY BY		APPROVED
CHIEF OF PARTY:	K. WRIGHT	<i>John Walter</i>
SUPERVISOR:	W. SMYTH	Director of Planning & Design
DRAWN BY		SCALE AS SHOWN
DRAFTSMAN:	R. J. LAMLER	DATE OF SURVEY - MAY 1963
SUPERVISOR:	J. CAMILLERI	DATE OF PLAN - MAY 1963
CHECKED BY		MON'TOIR-63-519KMR
DRAFTSMAN:	J. SCHUR	PLAN. E-4301-1
SUPERVISOR:	J. CAMILLERI	



JOB GIVEN TO E. N. PETO & ASSOC.
OCT. 21, 1963
ASS.



- Review
- Piles should be decided on basis of economy
 - Attn should be drawn to the Contractor to the comment on use of sheet piling for dewatering if required.
 - Attn should be drawn to the Bridge office on the matter of placing fill as far ahead of bridge construction as possible.
- June 3rd 1964 Ald. Gull



REFERENCE PLANS

E 4315-1

B 123-12

C 123-12

BW 967

BA 1729



REVISIONS		DATE		BY		DESCRIPTION	

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

BIG CREEK BRIDGE
WEST LIMITS OF DELHI

KING'S HIGHWAY No. 3 DIST. No. 2

CO. NORFOLK

TWP. MIDDLETON LOT 47 CON. I.N.S. OF TALBOT

PRELIMINARY GENERAL PLAN

APPROVED _____

BRIDGE ENGINEER

DESIGN B.M.S. CHECK _____

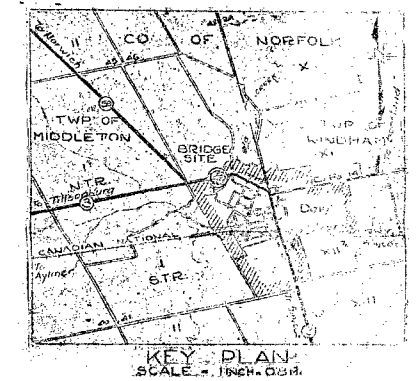
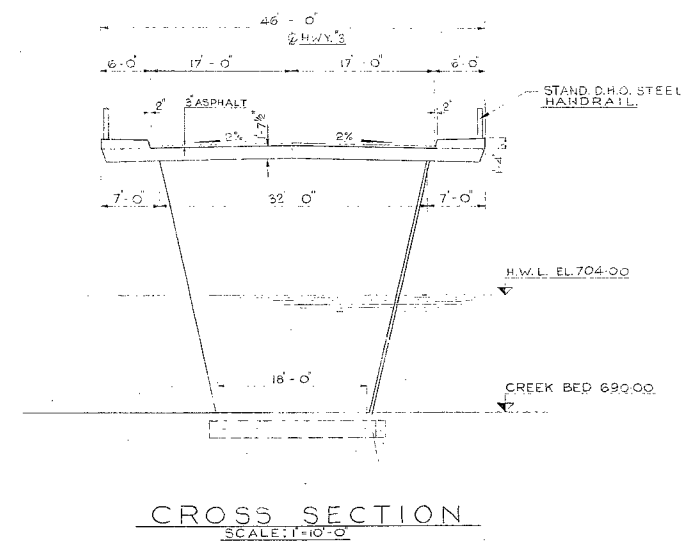
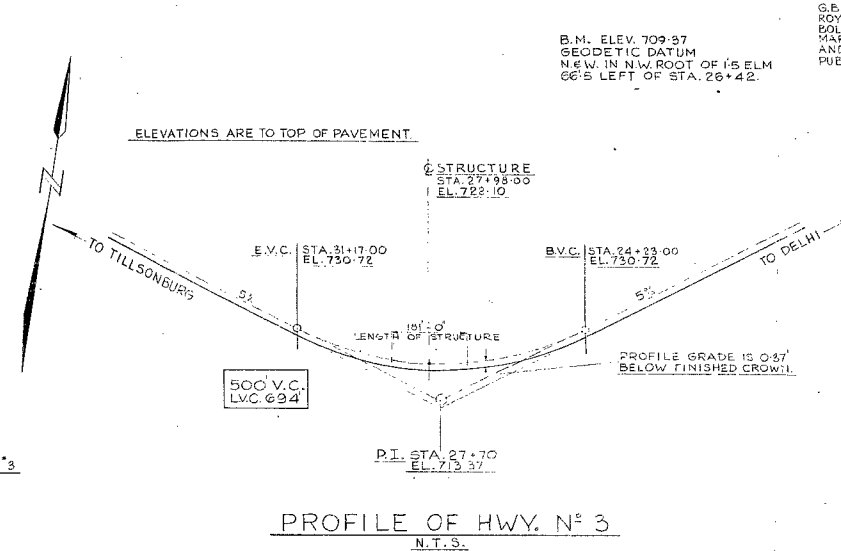
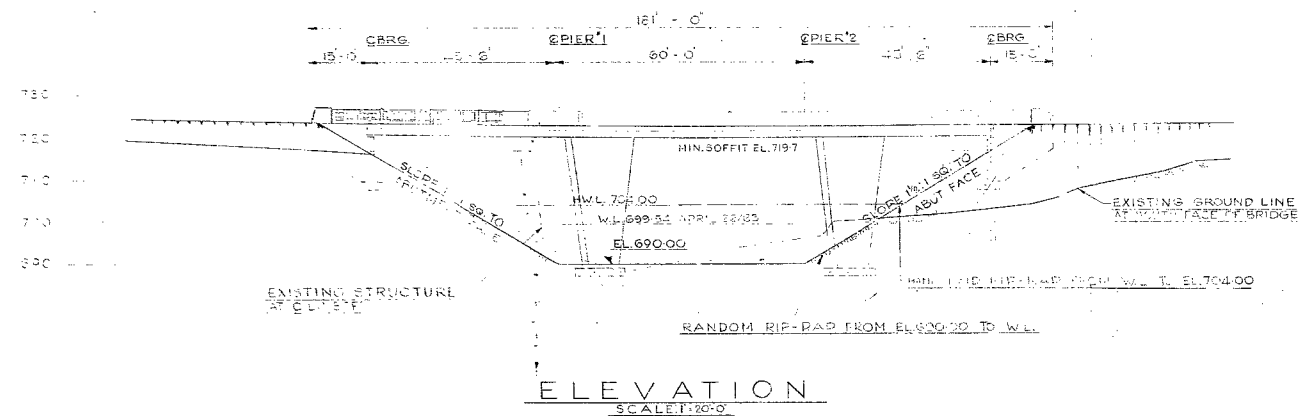
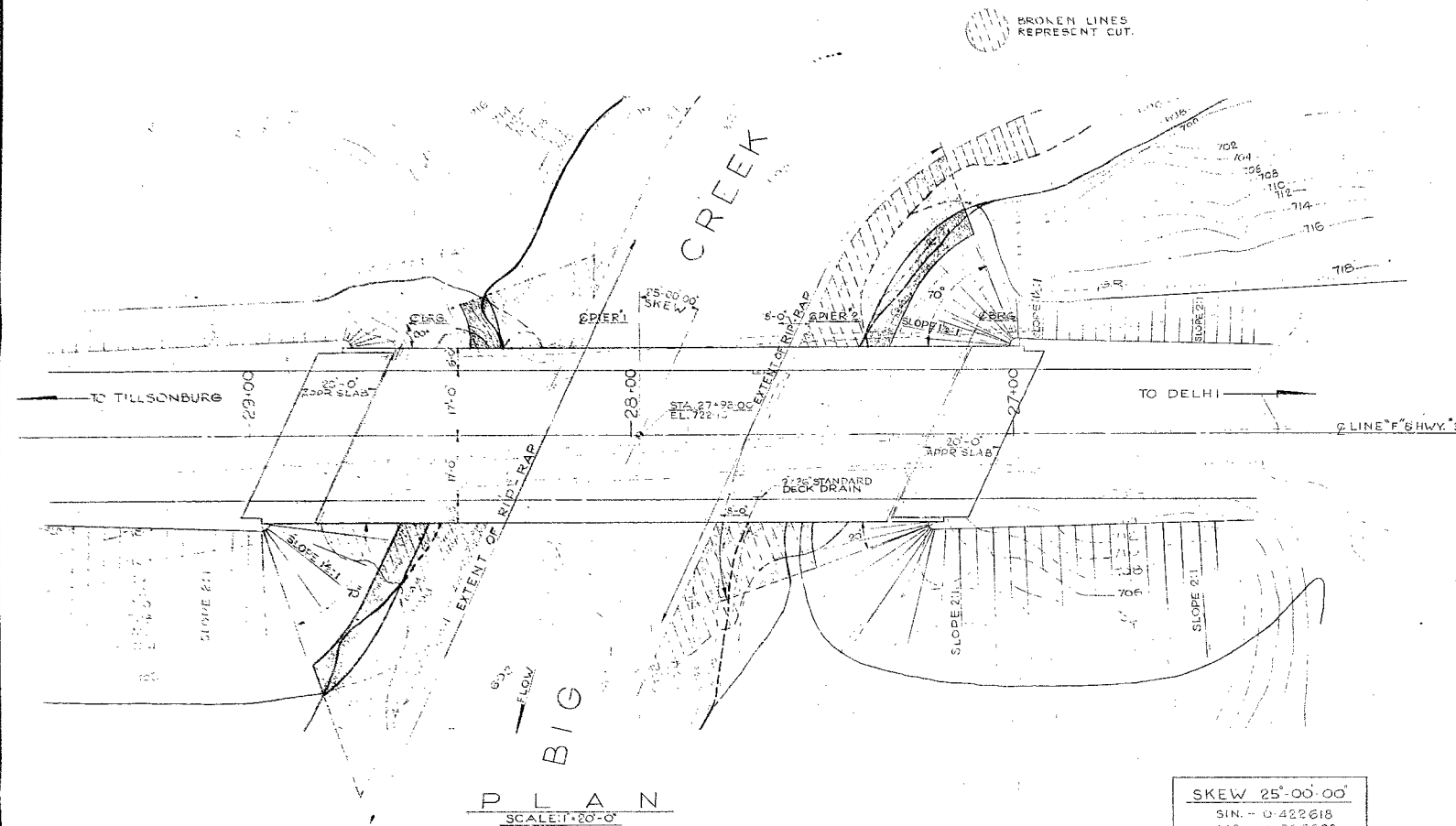
DATE MAY '64 LOADING H20/S16

SITE No. 21-59 W.P. No. 156-63

CONTRACT No. _____

DRAWING No. D5440-PI

File with Report by Peter

[illegible][illegible]

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

<h1 style="margin: 0;">BIG CREEK BRIDGE</h1> <h2 style="margin: 0;">WEST LIMITS OF DELHI</h2>			
KING'S HIGHWAY No. 3		DIST. No. 2	
CO. NORFOLK		CON. IN. & SGT. TALBOT	
TW. MIDDLETON		LOT 47	
<h3 style="margin: 0;">PRELIMINARY - GENERAL PLAN</h3>			
APPROVED		SITE No. 21-59	W.P. No. 126-63
BRIDGE ENGINEER		CONTRACT Nos.	
DESIGN <i>A & C</i>	CHECK <i>B & W</i>		
DRAWING B. M. S.	CHECK <i>B & W</i>	DRAWING No. D5440 - PI	
DATE MAY 24	LOADING 1220/516		

SOME DEFECTS IN NEGATIVE DUE

TO CONDITION OF ORIGINAL DOCUMENTS