

Mr. K. L. Kleinsteinber,  
Municipal Bridge Liaison Engr.,  
Bridge Division.

Attn: Mr. G.C.E. Burkhardt.

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

October 26, 1962.

Soils Report by E. M. Peto Assoc., Ltd.,  
Township Bridge #18, County of Lambton,  
Bridge Office Ref. #BA 1512.

We are in receipt of a copy of the letter dated October 18, 1962, addressed to Mr. J. Van Deus, County Engineer, Sarnia from E. M. Peto and Associates, Ltd., in reply to our comments contained in the memo of October 4, 1962, to Mr. K. L. Kleinsteinber, Municipal Bridge Liaison Engr. regarding the above-mentioned foundation report.

In his letter, E. M. Peto and Associates have listed reasons for the work he performed. The arguments presented, however, did not bring forward any new or significant point, and therefore, we would consider our comments in the memo of October 4, 1962, justified.

We would like to draw your attention to some aspects of E. M. Peto and Associates' letter.

On page 1, the following statement is made: "The clay in question contained pebbles and for this reason it was considered that the Standard penetration blows could have been influenced to the extent of giving favourable results."

On page 2, the following conclusion is drawn to prove the justification for performing unconfined compression tests on split-spoon samples:

"(c) The results do in fact confirm and support the strength suggested by the Standard penetration test, which was in fact designed for sands and not clays."

We find it impossible to reconcile these two statements.

With regard to settlement computation, we would like to emphasize again, that because of the preconsolidated nature of the subsoil, settlements will be small, certainly within allowable limits, and therefore an analysis is unnecessary.

Mr. K. L. Kleinsteinber,  
Municipal Bridge Liaison Engr.,  
Bridge Division.

October 26, 1962.

One of the reasons put forward by the consultant for suggesting the settlement analysis is the fact that the existing concrete abutments show considerable cracking and settlement. We wonder whether the proposed settlement analysis would have provided an explanation for this.

Another reason given by the consultant in his letter is that the new bridge is to be a rigid frame structure where differential settlements are an important consideration.

Differential settlements can only be a result of -

- a) different load intensities,
- b) different or heterogeneous soil conditions.

The load intensities are, as we know, the same. It suffices to glance at the drawing of the report showing the soil section through P.E. #1 and #2 to realize that one can hardly encounter more homogeneous subsoil conditions.

At least, but not least, it should be noted that for a proper settlement analysis, additional field work would be required to provide undisturbed samples. All the consultant has now, are samples from the split-spoon sampler used in the Standard Penetration tests. The use of such samples for oedometer testing is inconceivable.

The consultant's statement that it is extremely difficult to satisfy a technical client, is certainly unjustified. Never have we been critical of any work that was reasonable and warranted. However, we find it our duty to comment upon any foundation work that the Department pays for, which we feel is not satisfactory.

As far as expenses are concerned, we think that this is a matter of principle, not of amount. If expenses are incurred because of some tests or work that is unnecessary, then they are unwarranted irrespective of the sum to be spent.

KYL/WdeF  
cc: Foundations Office  
Gen. Files.

K. Y. Lo,  
SUPERVISING FOUNDATION ENGR.  
For:

A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.

**e. m. peto associates ltd.**

YOUR REFERENCE :

OUR REFERENCE :

**62157**

**1287 caledonia road  
TORONTO 18, ONTARIO.  
HUsell 8-1128**

**October 18th, 1962.**

**Mr. O. van Deurs,  
County Engineer,  
County Buildings,  
Sarnia, Ontario.**

**Dear Sir,**

**Re: Township Bridge #18  
County of Lambton  
D. H. Bridge Office Ref, #BA1512**

Thank you for your letter dated October 17th, 1962, enclosing a copy of a memorandum to Mr. Kleinstelber from the Foundation Section of the D. H. O.

We feel that the criticisms made by the Foundation Section are somewhat harsh and for this reason alone warrant at least an appreciation of the reasons for the work that was done, and the recommendations made in this report.

The clay in question contained pebbles and for this reason it was considered that the standard penetration blows could have been influenced to the extent of giving favourable results.

Our description based on visual examination and manipulating the samples is firm to stiff.

The unconfined tests were carried out as corroborative evidence of the strength of the clay suggested by the penetration blows.

**DEFECTS IS NEGATIVE DUE TO  
CONDITION OF IMPRINT DOCUMENT**

Whilst the comments in regard to the use of split spoon samples are in general technically correct, we feel that there is every justification in this case for making these tests because:

- (a) The soil is of low sensitivity
- (b) The samples were in our opinion fair samples and are described as such.
- (c) The results do in fact confirm and support the strength suggested by the standard penetration test, which was in fact designed for sands and not clays.

The suggestion that the settlement should be analysed was made for the following reasons:

- (a) The new bridge is to be a rigid frame structure where differential settlements are an important consideration.
- (b) The existing concrete abutments do show considerable cracking and settlement.
- (c) The existing span is approximately 55 feet, whereas the new span is proposed as 90 feet. Therefore any preloading effect derived from the existing abutments will only be of minor importance at the new abutment locations.
- (d) In view of the foregoing considerations we offered to carry out a settlement analysis in order to give the Engineer every confidence in the soil conditions reported and on which his design would be based. We did at the same time indicate that differential settlement was unlikely to be a problem.

- (e) At the time of the report we were not aware of the loadings or the footing sizes proposed by the Engineer and this was a further reason for offering to review the conditions when the design was finalized.

Contrary to popular belief we do not make an additional charge for this service where a report has already been submitted, unless a previous arrangement has been made for further work subsequent to the preparation of the report.

May we point out that it is extremely difficult to satisfy a technical Client who could have been equally critical if we had based our findings on the results of the standard penetration test only without any corroborative evidence.

Lastly we feel that the comment in regard to unwarranted expenses is extremely harsh bearing in mind the cost of the tests referred to which amount to some \$45.00, a very small sum in comparison to the cost of the structure.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

*C. F. Freeman*

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

CFF/ap

cc: J. A. Monteith & Associates  
D.H.O. Bridge Division  
D.H.O. Foundation Section.

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

Mr. K. L. Kleinsteinber,  
Municipal Bridge Liaison Engr.,  
Bridge Division.

Attn: Mr. G.C.E. Burkhardt.

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

October 4, 1962.

Re: County of Lambton,  
Brooke-Warwick Townline,  
Lot 20, Con. VI, Warwick Twp.,  
Structure Site #15-114,  
Bridge Office Ref. #BA 1512.

The report on the soils investigation at the above site, submitted by the Consultant, R. M. Peto Associates, has been reviewed.

The subsoils at the site are favourable and do not present any problem for the proposed structure. However, we find that the Consultant has done some unnecessary and irrelevant work, as shown in the report. The Standard Penetration resistance exceeds 10 blows at Bl. 90 ft., increasing to 30 blows at Bl. 60 ft. The clay stratum is therefore, stiff to very stiff. This statement is substantiated by the fact that the natural moisture content is close to the plastic limit. An allowable pressure of 2 T.S.F. may easily be obtained from such soil.

Ten unconfined compression tests were performed by the Consultant on split-spoon samples. These tests are not only unnecessary, as explained before, but also misleading, since the split-spoon samples must be too disturbed for reliable shear strength determinations.

The suggestion contained in the "Conclusions and Recommendations" that settlements should be analyzed, is incomprehensible. The geology of the area, the liquidity of the clay and the high penetration resistance all show that the clay is overconsolidated. In addition, the site is also preloaded by the existing bridge. Therefore, settlement should be within tolerable limits for such a structure.

It is our opinion that the Consultants should be informed of these comments so that unwarranted expenses may be avoided in any future work.

KYL/mdeF

cc: Foundations office  
Gen. Files.

K. Y. Lo,  
SUPERVISING FOUNDATION ENGR.  
For:

A. G. Stermac,  
PRINCIPAL FOUNDATION ENGR.

BA1512

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,  
Toronto 19, Ontario.

Our Job Number 62137

RUSSELL 9 - 1126.

August 20th, 1962.

The County of Lambton,  
c/o J. A. Monteith & Associates Ltd.,  
P. O. Box 579,  
Petrolia, Ontario.

STRUCTURE SITE No. 15-114

Attention: Mr. G. Ingram.

Gentlemen,

Soil Site Investigation  
Township Bridge #18  
Warwick/Brook Township Line.

We have pleasure in submitting three copies of our Report Number 62137 on the above site investigation. One additional copy has been forwarded directly to Mr. O. Van Deurs, Lambton County Engineer.

Foundations of the new bridge, in the form of spread footings, can be placed at the proposed elevation 61, where they will rest on a stiff clay till stratum. The allowable net bearing capacity at this depth is 2.0 ton/sq. ft.

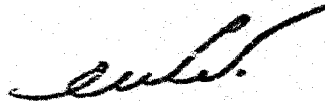
PAGE TWO

Theoretical settlement was not analysed, as it will depend on the size of the footings and on the applied load (dead and live separately), which data was not available at this stage. We will gladly carry out an estimate of the settlement after receiving the necessary information.

We would also be very pleased to answer any queries, which you may wish to raise in connection with this report.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

A handwritten signature in dark ink, appearing to read 'E. M. Peto', with a stylized flourish at the end.

E. M. Peto, P. Eng.

RK/ap



THE COUNTY OF LAMBTON  
C/O J. A. MONTEITH & ASSOCIATES LTD.

SOILS REPORT

PROPOSED RECONSTRUCTION OF  
TOWNSHIP BRIDGE NO. 18  
COUNTY OF LAMBTON, ONTARIO.

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,  
Toronto 19, Ontario.

## TABLE OF CONTENTS

	<u>Page Number</u>
A. INTRODUCTION	1
B. GENERAL INFORMATION	2
C. SITE and GEOLOGY	4
D. SOIL CONDITIONS	5
E. WATER CONDITIONS	10
F. CONCLUSIONS and RECOMMENDATIONS	11

APPENDIX "A" STANDARD PROCEDURE

APPENDIX "B" LABORATORY TEST RESULTS

BOREHOLE LOGS

SITE PLAN and PROFILE

## A. INTRODUCTION

The work described in this report was authorized verbally by Mr. G. Ingram of Messrs. J. A. Monteith and Associates Ltd., Consulting Engineers, on behalf of the County of Lambton.

A new bridge is to replace an existing structure, which carries a minor road over Brown Creek on the Warwick-Brook Township Line. A site investigation was required for the design of the new bridge foundations.

The existing bridge is approximately 55 ft long and 16.5 ft wide. It is supported by concrete abutments, which show considerable cracking and settlement.

The new bridge is expected to be of rigid frame construction, with a possible span of 90 ft between abutments. Taking the elevation of centre of deck of existing bridge as 100.0, the Consulting Engineers expected to place the footings of the new bridge near the elevation 81. The low water level in the stream is at the elevation 89, while a high water level, corresponding to a flood condition occurring once in twenty years, is 87.5. This is slightly above the road level on the east side of the existing bridge.

## B. GENERAL INFORMATION

1. Two test holes were carried out on the site. The test holes were put down from the shoulder of the existing road on the embankment a short distance on either side of the present bridge. This followed the instructions of Mr. G. Ingram, issued to our drilling foreman, who chose the actual location of the test holes. The test holes were subsequently surveyed by our engineer.

2. Grade elevations at the positions of the test holes were measured by our engineer and referred to a temporary bench mark of assumed elevation 100.0, taken as the middle of the existing bridge deck at mid span.

The elevations are entered on the enclosed borehole logs, and are also shown on the enclosed site plan and subsoil profile.

3. The field work was performed by our drilling crew unit No. 6, between August 9th and 11th, 1962. Our standard drilling and sampling procedures were followed, as outlined in the enclosed Appendix "A".

**B. GENERAL INFORMATION .. Cont'd**

4. Both test holes were terminated at a depth of 41.5 ft below the existing grade at the top of the present embankment. In view of the uniform stratification and satisfactory consistency of the subsoil, this depth was sufficient to solve the engineering problems connected with the new bridge foundation design.

Details of the soil conditions found in the test holes are described on the enclosed borehole logs, which contain also results of standard penetration tests and natural water content measurements.

A simplified subsoil profile, in the form of a section through the test holes is included on the enclosed drawing. The section is based on a road profile provided by the Consulting Engineers.

5. Laboratory testing of subsoil samples consisted of measurements of cohesive strength by unconfined compression test and determination of Atterberg limits and grain size distribution of the clay till stratum. The results of the tests are included in Appendix "B". In addition, the apparent cohesion, moisture content and Atterberg limits and standard penetration resistance are plotted against elevation on the appended drawing.

### C. SITE and GEOLOGY

The site of the proposed new Township Bridge No. 18 is located on a minor road on the Warwick-Brook Township line, where it crosses the Brown Creek. The site is roughly one half mile to the east of Highway 79, and approximately one mile south of Watford along this Highway.

The terrain at the bridge site is gently undulating. On the west side, the road level rises by some 25 ft within 500 ft of the bridge, while on the east side the ground is almost flat. The stream meanders at the site, running roughly parallel to the existing embankment for 150 ft on the south-west side of the bridge. The present embankments are up to 10 ft high.

Geologically, the site overlies a clayey till stratum, which forms the main overburden over a shale bedrock in the area. Both test holes were terminated in the clay till stratum, and the shale bedrock was not reached. Its depth at the site is unknown.

#### D. SOIL CONDITIONS

Details of the soil conditions encountered in the test holes are described on the enclosed borehole logs, while a simplified subsoil profile is included on the appended drawing.

The subsoil can be subdivided into the following general soil types, in the order of occurrence:

		Average Elevation	
		<u>Top</u>	<u>Bottom</u>
(a)	Crushed stone fill	Existing grade	98.5 and 96.6
(b)	Silty clay fill	98.5 & 96.6	88
(c)	Organic silt	88.9	87.6 (B.H. #2 only)
(d)	Layers of silt and clay with sand and pebbles	88	84
(e)	Silty clay till	84	Unknown.

The engineering characteristics of each of the above soil types will now be briefly described .

D. SOIL CONDITIONS - Cont'd

a) Crushed stone fill

Both test holes were put down from the edge of the travelled road, the surface of which consists of crushed stone mixed with sand, silt and clay fill. This type of fill was 8 inches thick in test hole 1, and 2 ft 3 in. thick in test hole 2.

This material can be reused in the reconstructed embankment.

b) Silty clay fill

The existing embankments were found to be built up of a silty clay fill, which extended to a depth of 11.3 ft below the existing grade in test hole 1, and to approximately 10 ft in test hole 2 (average elevation 89). The fill was of dark brown colour and was generally in a soft to firm state of compaction. Standard penetration test results varied from 5 to 11 blows per foot, showing a tendency to increase with depth, while the average of the results increased from 7 blows per foot near the top to 10 blows per foot near the bottom of the fill. The results are plotted against elevation on the drawing.

The moisture contents were variable, ranging from 12.6% to 21.1%, as shown on the borehole logs and drawing.

The fill consisted essentially of silty clay with some sand and pebbles, and contained some plant roots and traces of other organic matter.



D. SOIL CONDITIONS - Cont'd

It is considered that this fill material can be reused in the reconstructed new embankments.

c) Organic silt

A thin layer of organic silt was encountered between the depth of 10 ft and 11 ft 3 in. in test hole 2, immediately below the brown silty clay fill. The elevation of this layer corresponds to the creek level and can be assumed to indicate the former level of creek bottom before the embankments were constructed. The material was soft, with a moisture content of 27.2%. It contained some sand and clay. It was not reported in test hole #1.

d) Layers of silt and clay with sand and pebbles.

Variable layers of silty clay, silt and sand with some pebbles were encountered between the depths of 11.3 ft and 14.7 ft in test hole 1 and 11.3 ft and 15.4 ft in test hole 2. This material was found to rest on top of the silty clay till stratum, and was probably water-deposited in a drainage channel, eroded in the surface of the clay till stratum. Due to the variability of these layers, the borehole logs should be consulted for individual description of the samples. However, the consistency of the material was generally firm or firm to stiff. Standard penetration test results were in the range of 10 to 14 blows per foot, while the moisture content ranged from 19.8% to 22.0%.

**D. SOIL CONDITIONS - Cont'd**

Layers of silt, reported in this stratum near elevation 85, produced some water seepage, particularly in test hole 1.

**e) Silty clay till**

This material forms the main overburden over bedrock in the area. It commenced at a depth of 14.8 ft in test hole 1 and 15.4 ft in test hole 2, corresponding to elevations 84.4 and 83.5 respectively.

The deposit is of grey colour, and consists of a mixture of silt and clay with some sand and pebbles. Its typical till form indicates a glacial deposition. A grain size distribution curve, determined on a sample of the material, is included in Appendix "B".

The variation of consistency of the clay till with depth is illustrated graphically on the enclosed drawing, on which the standard penetration test results (N-values), water content and Atterberg limit distribution, and the undrained shear strength (cohesion) are plotted against elevation.

The curves indicate that the material is firm in its upper portions, where the average standard penetration test results are between 14 and 16 blows per foot. The density increases progressively with depth, and the rate of increase of density is greater below the elevation 70, below which the material has a stiff consistency.

D. SOIL CONDITIONS - Cont'd

The results of shear strength and moisture content measurements, also plotted on the drawing, show a certain scatter, which is normally associated with a till material. However, a tendency for an increase of strength and decrease of water content with depth is clearly observable on the graphs. The average moisture content falls from about 17.5% at the elevation 73, where it reaches a peak, to 15% at the elevation 55. The material is also drier (average moisture content 16%) in the upper portions of the stratum, where it can be considered to form a stiffer crust. However, the crust is not very pronounced.

Two Atterberg Limit tests were performed on samples of the clay till and the results are included in Appendix "B" and also plotted on the drawing. The natural moisture content is observed to be near the Plastic Limit of the material, which corresponds to a stiff consistency.

The results of unconfined compression tests are tabulated in Appendix B, where the wet and dry densities and void ratios are also included. The cohesion values (also plotted on the drawing) indicate that the average undrained shear strength increases from about 2100 lb/sq.ft at elevation 80 to more than 3000 lb/sq.ft below elevation 60.

On the basis of the above relationships, we consider that the shear strength, for the purpose of calculation of bearing capacity of foundations located at or below elevation 81, can be taken as 2000 lb/sq.ft.

## E. WATER CONDITIONS

In test hole 1, the seepage caused the hole to collapse at this depth, but was cut off when the casing penetrated into the clay till stratum to a depth of 15 ft below top of borehole.

In test hole 2, a water-bearing silt layer was reached at a comparable depth, but the quantity of seepage was so small that the test hole could be continued, practically dry, without driving casing below a depth of 10 ft.

Apart from the above cases, no pervious, water-bearing seams were encountered in the two test holes. The ground water table at the site can be assumed to correspond to the average level of water in the creek.

## F. CONCLUSIONS and RECOMMENDATIONS

1. The test holes have indicated that foundations of the new bridge, in the form of spread footings, can be placed at the proposed elevation 81.0. At this depth, the footings will rest near the top of the firm to stiff grey silty clay till stratum, the undrained shear strength of which is not less than 2000 lb/sq. ft within the depth which will be affected by stresses from the footings.
2. The allowable net bearing capacity under the footings, with a factor of safety of three, can be taken as 2.0 ton/sq. ft. To the above figure, a component equal to the least weight of final overburden above the footing can be added. However, where the footings are to be adjacent to the stream, it would appear safer to neglect the additional overburden effect, as a precaution against possible removal of the overburden by scouring.
3. The above recommendations regarding the foundation level and allowable bearing capacity are based on the soil conditions as established in the two test holes. Local variations are possible at the site, particularly on the southern side of the western abutment, where the stream runs parallel to the existing embankment and a short distance from its toe. In this area, the till could possibly be eroded by the stream to a greater depth than indicated by the nearest test hole. A decision as to whether elevation 81 is suitable in this area for

F. CONCLUSIONS and RECOMMENDATIONS - Con'd

the support of footings should be made following field inspection of the excavated grade. This applies also to other areas where similar conditions may obtain.

4. The settlement of the bridge was not analysed, as it will depend directly on the dimensions of footings and on the applied pressures, which were not determined at this stage. However, in view of the proposed rigid frame type of the structure, in which bending moments induced by differential settlements may be of importance, we consider that the settlement should be analysed, and we will gladly carry out the calculations after receiving the above, necessary data. It would appear, however, that the total settlement will be within tolerable limits, while no indications were obtained in the two test holes of any conditions that may potentially lead to differential settlement between the two abutments.

Consolidation tests were not performed on soil samples from this site, but we consider that sufficiently accurate estimate of settlement can be obtained using relationships based on our amassed test results from other sites in the area.

**F. CONCLUSIONS and RECOMMENDATIONS - Cont'd**

5. Apart from a thin layer of silt near elevation 85, producing minor seepage, no free water-bearing, permeable seams were encountered in either of the test holes, which could carry water from the creek into an excavation. It therefore appears probable that once the flow from the creek is cut off or diverted, the excavation for the footings can be conducted dry or with only minor water seepage.

Every precaution should be taken to ensure that the excavated formation grade below the footings is kept free of water, as contact with water would cause the clay to swell, resulting in a loss of strength, and leading to additional subsequent settlements. It is considered preferable to construct the footings immediately after excavation of the grade. Should, however, a delay between completion of excavations and construction of the abutments be unavoidable, the excavated grade should be protected by an impervious seal, e. g. in the form of a thin layer of lean concrete. Alternatively, the last six to twelve inches of the clay should remain unexcavated until the last possible moment before construction of the footings.

The clay has sufficient cohesive strength to allow the excavations to stand open in a vertical cut during a reasonable period of construction.

## F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

Should any pervious seams be disclosed by the excavation, water seepage from such seams should be blocked by sheeting, or led away from drains placed below the pervious layer. Should the quantity of seepage be considerable, the water should be pumped from auger holes, put down a short distance outside the excavation and opposite the pervious layer. The alternative solutions, like a water tight sheet pile cut off or an interceptor trench placed outside the excavation, would probably be uneconomical for this project. However, occurrence of serious water conditions is not anticipated.

6. Most of the excavated material, including the fill recovered from the existing embankments, can be reused in the new bridge approach embankments. However, due to the high silt content of the fill, due to which it is classified as susceptible to frost heave, a well compacted blanket of granular material, at least six to twelve inches thick, should be included under all pavements in order to ensure against damaging effects of frost heave.



F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

7. Minor settlement of the new embankments can take place if the grade is raised in areas where the existing fill overlies the soft organic silt layer, as encountered between the depth of 10 ft and 11.3 ft in test hole #2. The thickness of this organic layer, which probably formed the topsoil of the original creek bed before deposition of the embankment fill, at various points cannot be determined without additional test holes. However, it is not likely to be considerable and the settlement of the embankment will probably be only minor.

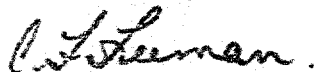
Where the new embankments are to be placed over the existing creek floor, the soft organic silt should preferably be removed before placement of the fill. The depth of the material to be scraped off should be determined by visual inspection.

Report prepared by:



R. Kulcsza, P. Eng.

E. M. PETO ASSOCIATES LTD.



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

APPENDIX    "A"  
STANDARD    PROCEDURE

The field investigation work is carried out by means of a skid mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by augers, tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals, with either a 2 inch or 3 inch O. D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

The standard penetration test results are recorded when sampling with the regular 2 inch O. D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch Cone probe test is made by driving the drill rods into the ground with a 2 inch dia. x 60° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using Modified Acker vane test equipment.

Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing as required.

The test holes are bailed (or pumped out) during the work as necessary, at the end of the day, and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when Artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.

Borehole logs are prepared giving details of the soil description and condition as recorded in the field. These logs form the basis of the soil profile, which indicates the general stratigraphy assumed to exist between the boreholes as represented by the borehole logs.

The boreholes are normally set out by the Field Engineer, who also records the ground elevations referred to a temporary bench mark or known reference point. If the client has been responsible for setting out the boreholes and recording their ground elevations this is stated in the preamble to the report.

A plan is drawn up from drawings supplied by the Client or his representatives, showing the locations of the boreholes and the T. B. M. where applicable.

Normally, the standard penetration blows and the natural moisture contents are plotted against elevation as a graph, and these graphs form part of the appendices, together with laboratory test result details, ground water readings and other soil characteristics which can be best illustrated in graphical form.

APPENDIX "E"

LABORATORY TEST RESULTS

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

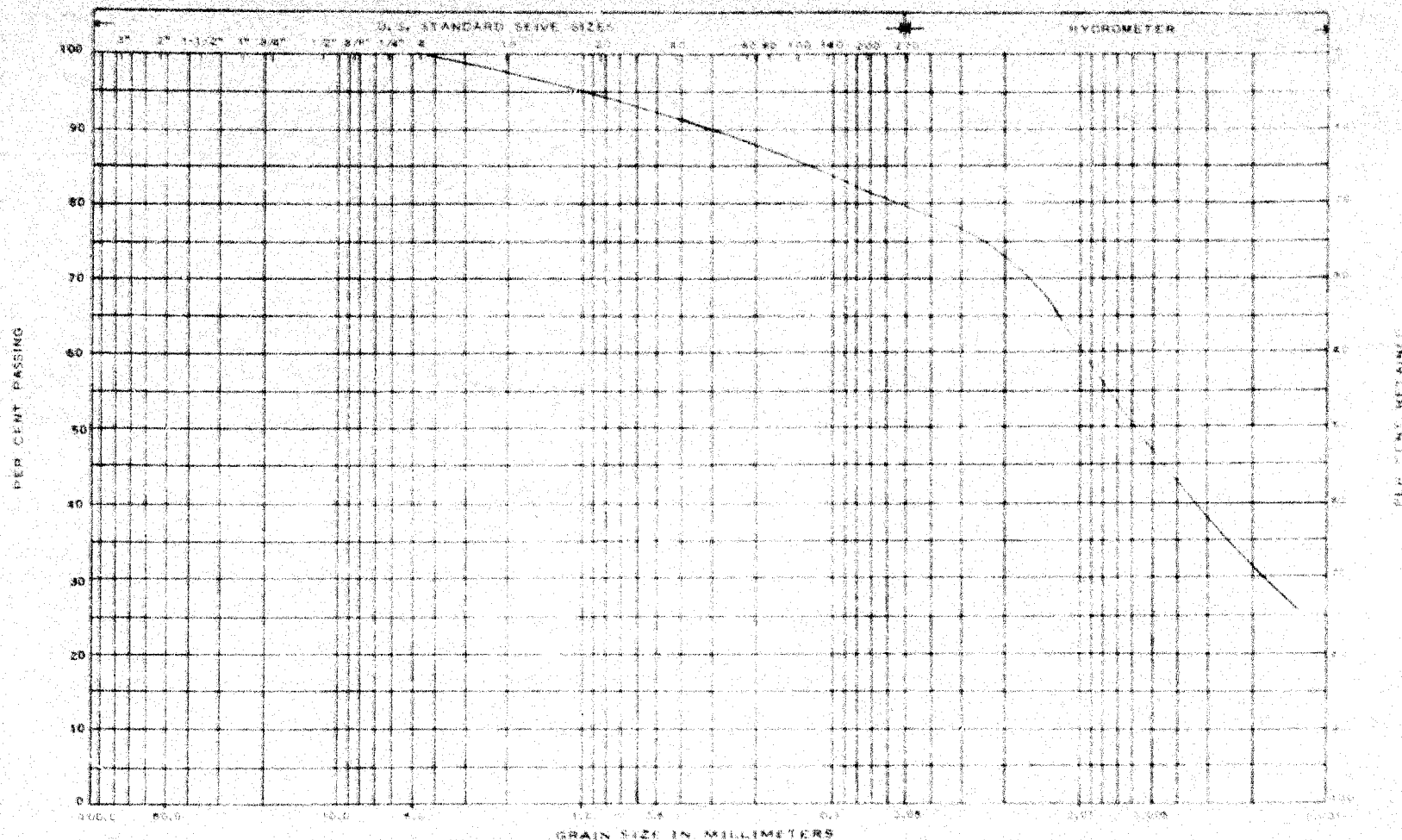
Job No. 62137

Atterberg Limit Test Results

B. E. /Sa. No.	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Natural Water Content
	Ft.	%	%		%
1/10	31	32.5	17.5	15.0	18.3
2/7	21	29.7	15.2	14.5	15.4

e. m. peto associates ltd.

Toronto 18, Ontario



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

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Township Bridge #18 JOB NO. 02137 HOLE NO. 2 SAMPLE NO. 10  
 DEPTH 31' ELEVATION \_\_\_\_\_ REMARKS Clay Till

GRAIN SIZE DISTRIBUTION

## E. M. PETO ASSOCIATES LTD.

## UNCONFINED COMPRESSION TEST DATA SHEET

Job No. 62137

Corehole Number	Sample Number	Depth feet	Wat. M. C.	Wet Density p.c.f.	Dry Density p.c.f.	Voids Ratio, %	1/c Shear Strength p.s.f.
1	8	20'-21'6"	17.6	137.0	116.7	0.43	3270
1	9	25'-26'6"	18.3	134.8	113.8	0.48	3320
1	10	30'-31'6"	18.3	134.8	113.8	0.48	2410
2	6	15'-16'6"	15.5	141.0	122.1	0.28	2470
2	7	20'-21'6"	17.7	139.0	118.2	0.42	1960
2	11	35'-36'6"	16.4	139.7	120.0	0.40	2580
2	12	40'-41'6"	19.7	139.0	119.2	0.41	3380
2	9	27'-27'6"	18.7	134.4	113.3	0.48	2010
2	9	27'6"-28'	21.0	129.0	106.5	0.58	470

SOIL DESCRIPTION	COLOR	Depth Feet	Depth Meters	Sample No. and Location	Sample Type	No. of Blows per ft.	WATER LEVELS & REMARKS
Crushed stone (road surface)	Grey		0'0"	1	C.S.		
Silt and clay fill	Brown	Soft		2	S.S.	5	21.1
As above with some pebbles and plant roots	Ditto	Firm		3	S.S.	10	20.3
Silt with some clay, sand & pebbles (fill)	Ditto		5'3"	4	S.S.	11	18.3
Laminated silty clay with some pebbles	Brown with Grey	Ditto	11'3"	5	S.S.	10	22.0 Virgin soil
Silty clay with sandy silt seams and pebbles	Dark grey with brown	Firm to stiff	13'4" 13'8" 14'8"	6	S.S.	14	19.8
Silt layer							Water seepage in silt layer
V. Silty clay with pebbles	Light grey	Firm to stiff		7	S.S.	13	17.4 Near P.L.
			20'0"				
Silty clay with pebbles	Grey	Firm		8	S.S.	14	17.4
			23'0"				
Ditto	Ditto	Firm to stiff		9	S.S.	18	17.8
			30'0"				
Ditto	Ditto	Ditto		10	S.S.	26	18.3
			35'0"				
Ditto	Ditto	Ditto		11	S.S.	27	13.7
			40'0"				
Ditto	Ditto	Stiff	41'6"	12	S.S.	33	12.9
							Testhole terminated at 41'6"



e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

Job Name Township Bridge #18

Job No. 62137

Borehole No. 2

Client: The County of Lambton

Casing ... 43

Boring Date August 10&11/62

Elevation ..... 98.9

Compiled By: B. K.

Checked By H. K.

SAMPLE CONDITION



UNDISTURBED



FAIR



DISTURBED



LOS F

SAMPLE TYPE

### A. 5. ALGER SAMPLE

C.S. CASING SAMPLE

### 2.5. 2" STANDARD SPLIT TUBE SAMPLE

S.L. SPLIT BARREL WITH LINERS

S.T. THIN-WALLED SHELL BY TUBE SAMPLE

W.S. WASH SAMPLE 3

S.F. - ROCK CORE

### ABBREVIATIONS

### V.T. IN SITU VANE SHEAR TEST

M                      MOIST

W.L. WATER LEVEL IN CASING

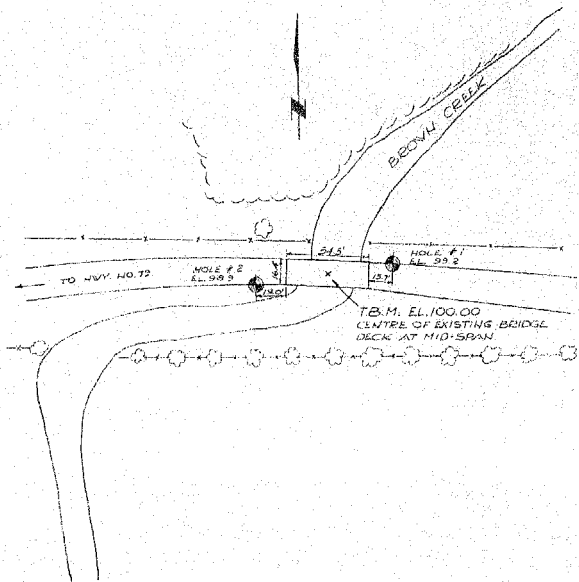
GROUND WATER TABLE IN SOIL

B.T.P.L. BETTER THAN PLASTIC LIMIT

S.I.P.L. DESER THAN PLASTIC LIMIT

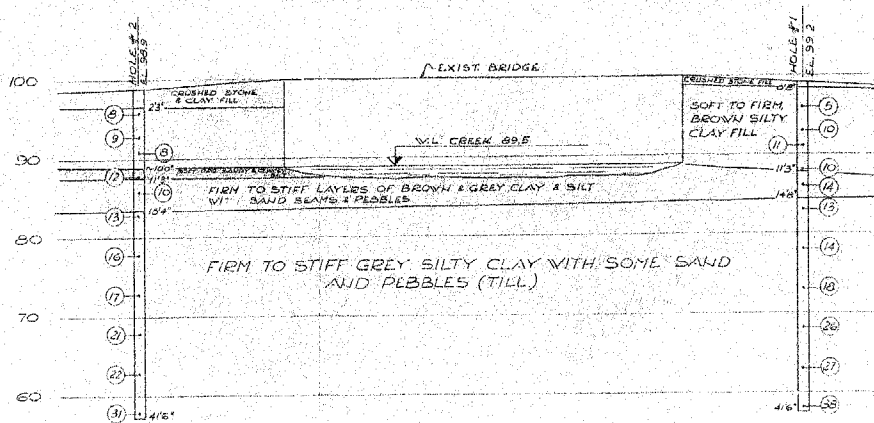
A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOR	Consistency or C. Coefficient	Depth Feet	Sample No.	Sample No.	Sample No.	Sample No.	Sample No.	Sample No.	WATER LEVELS & REMARKS
Mixed fill with crushed stone	Brown & grey		0'0"							
Silty clay fill, some pebbles and roots	Brown	Firm	2'3"	1		S.S.	8	15.7		
Silty clay with some pebbles (fill)	Brown	Firm	5'0"	2		S.S.	9	17.8		
Ditto	Ditto	Ditto		3		S.S.	8	17.6		
Seam of sandy and clayey silt with organic matter	Dark grey to black	Soft	10'0"	4		S.S.	12	27.2		Former creek bottom
Clay	Grey with some brown pockets	Firm	11'3"	5		S.S.	10	19.9		
Silty clay with some sand and pebbles (till)	Grey	Ditto	12'4"	6		S.S.	13	16.9		W.T.P.L.
Ditto	Ditto	Firm to stiff	20'0"	7		S.S.	16	15.4		W.T.P.L.
Ditto	Ditto	Ditto	25'0"	8		S.S.	17	17.5		
Ditto	Ditto	Ditto		9		2" S.L. Pushed				
Ditto	Ditto	Ditto	30'0"	10		S.S.	21	14.7		W.T.P.L.
Ditto	Ditto	Ditto	35'0"	11		S.S.	22	17.3		
Ditto	Ditto	Ditto	40'0"	12		S.S.	31	17.7		
			41'0"							Test hole terminated at 41'0"



SITE PLAN

SCALE: 50' TO 1"



SOIL SECTION THROUGH HOLES 2 & 1

PROFILE SCALE: HOR. & VERT. 10' TO 1"

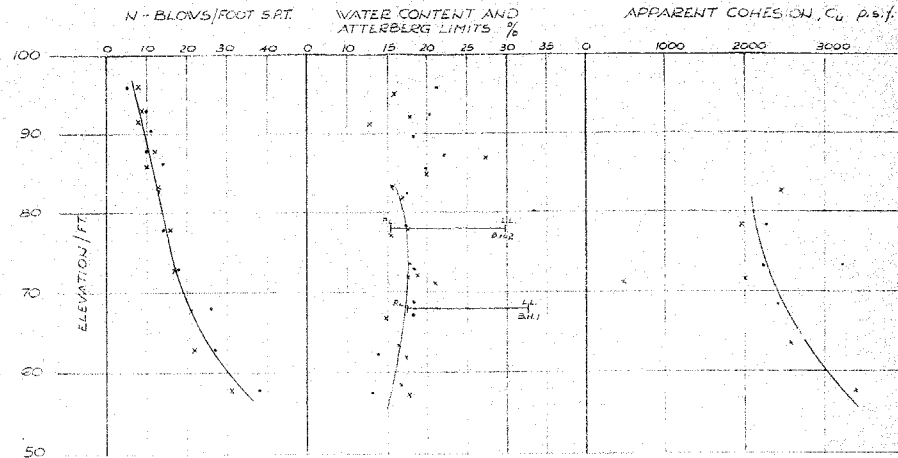
# LEGEND

- ⊕ BOREHOLE
- ⊖ BLOWS/FOOT S.P.T.

## NOTE:

SEE BOREHOLE LOGS FOR COMPLETE SOIL DATA.

NOTE: The actual soil stratification has been verified from data obtained at the borehole locations only. The inferred contacts shown are based on geological evidence and these may vary from those shown between borings.



SYMBOLS: ⊕ HOLE #1  
⊖ HOLE #2

## NOTE:

$C_u$  IS ONE HALF OF UNCONFINED COMPRESSION STRENGTH.



THE COUNTY OF LAMBERTON  
56 J. A. MONTEITH & ASSOCIATES LTD.

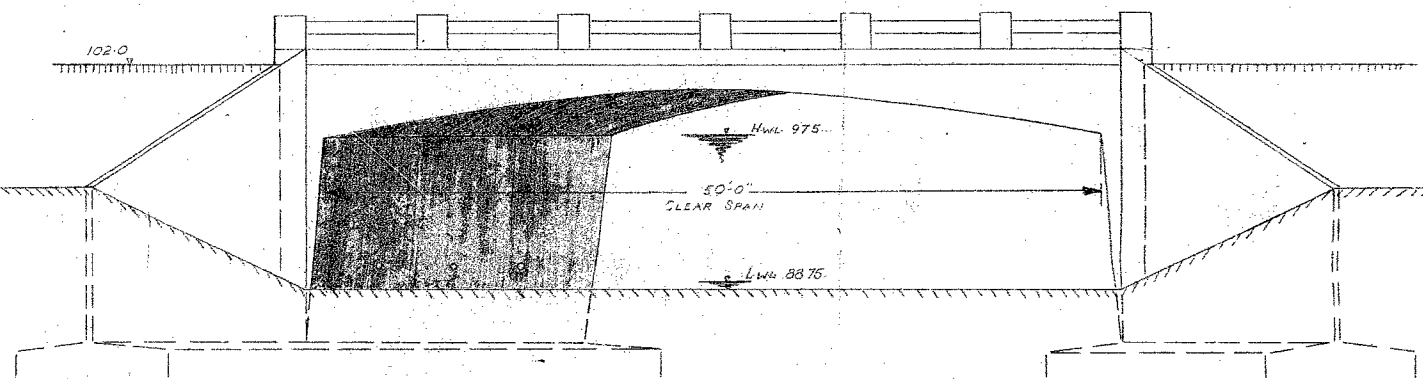
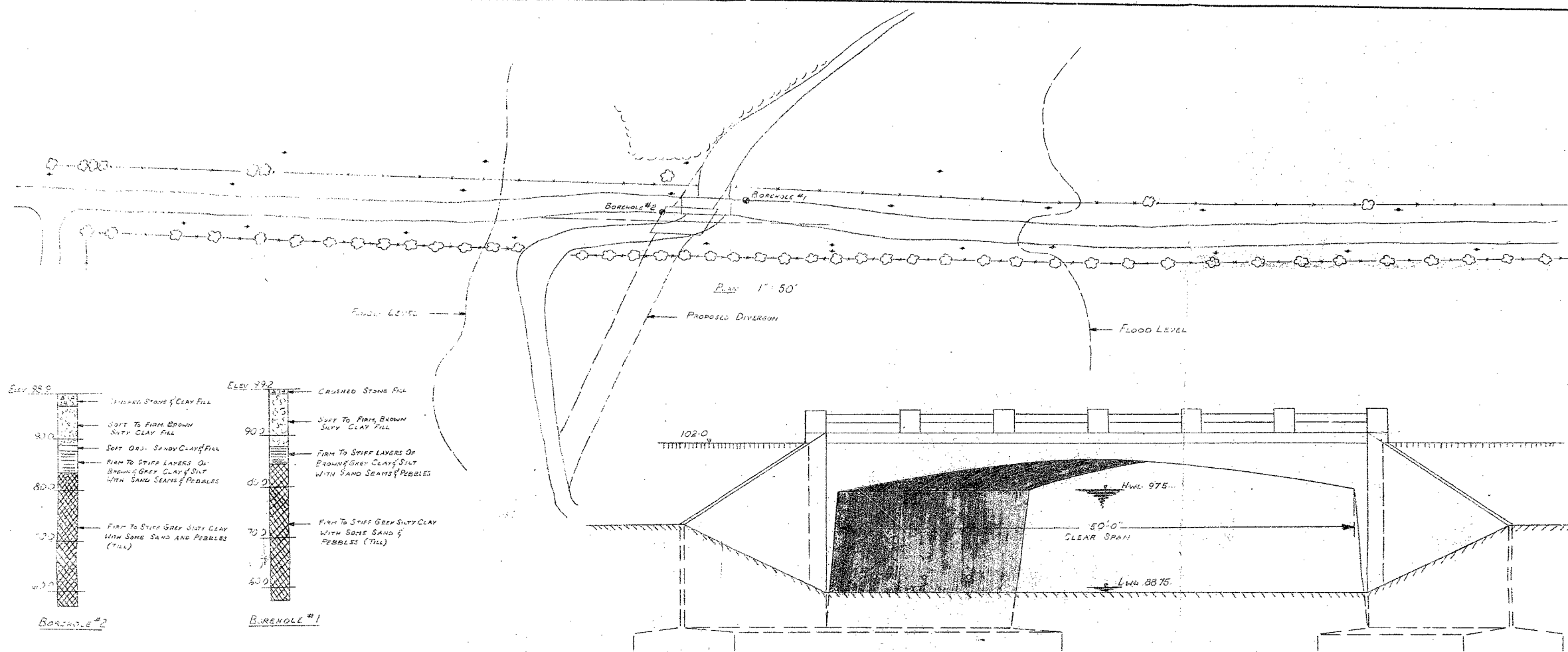
TOWNSHIP BRIDGE #18

PREPARED BY  
e. m. peto associates ltd.

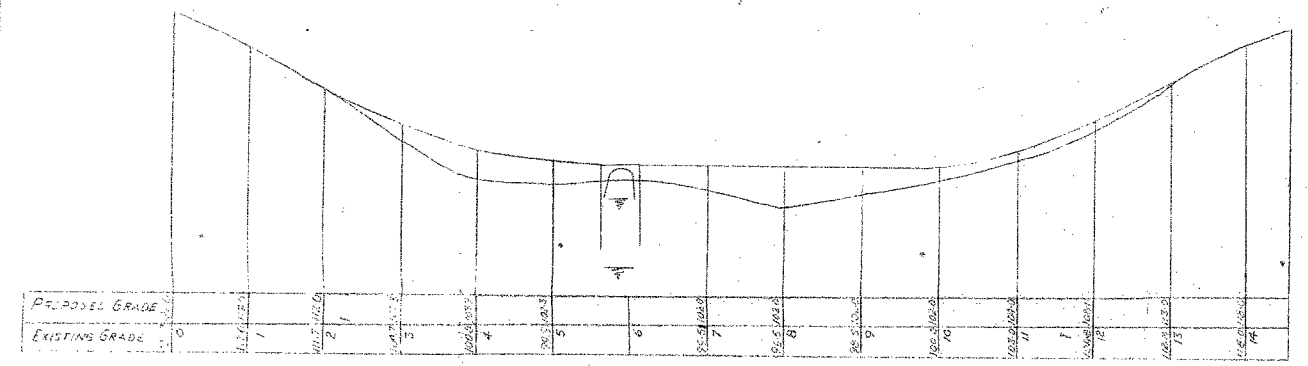
JOB NO. 62137 | AUG. 1962 | K.K.

#  
62-F-281M  
TWP BRIDGE 18  
BROOKE-WARWICK

TOWNLINE  
WARWICK  
TWP



ELEVATION OF PROPOSED STRUCTURE  
SCALE 1" = 5'



PROFILE  
VERTICAL 1" = 10'  
HORIZONTAL 1" = 100'

PLAN	GEOLOGY	HYDROLOGY	STRUCTURAL	REVISIONS	COUNTY OF LAMBTON
	SOILS REPORT BY F.T. PETO ASSOC. TORONTO SEE BORE-HOLE LOGS ABOVE SOIL ABOVE STIFF GREY SILTY CLAY NOT SUITABLE FOR BEARING OR SCOUR ALLOWABLE BEARING PRESSURE AT FOOTING ELEVATION 4 KILOPS/FT. <sup>2</sup> DESIGN BEARINGS 3 KILOPS/FT. <sup>2</sup>	H.W.L. FROM LOCAL KNOWLEDGE 97.5 L.W.L. 88.75 CATCHMENT AREA 14 SQ. MILES OF ROLLING LAND ESTIMATED FLOOD = 1,400 C.F.S. NET WATERWAY EXISTING 40 YEAR OLD BRIDGE IS 410 SQ. FT. PROPOSED BRIDGE 440 SQ. FT. BRIDGE ONE MILE DOWNSTREAM 60 FT. SPAN	50 FT. SPAN REINFORCED CONCRETE RIGID FRAME 30" SKEL. 43.9' EFFECTIVE SPAN A.D.T. 50 NO SIDEWALKS CU YDS. OF 3000 P.S.I. CONCRETE TONS OF REINFORCING STEEL CU YDS. OF FILL 100' EACH SIDE OF BRIDGE CU YDS. OF FILL IN REMAINDER OF CONTRACT DESIGN: C. J. JONES		STRUCTURE SITE No. 15-114 TOWNSHIP BRIDGE #13 CONTRACT # SITE DETAILS J.A. MONTEITH ASSOCIATES LTD. CONSULTING ENGINEERS PETROLIA ONT. DATE SEPT. 62 DRAWN RZ DWG. NO. 8-1965-1