

#63-F-244M

COUNTY RD. #12

BONNIE DOON

BRIDGE

(COUNTY BRIDGE
C-12-1)

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.

December 23, 1963

County of Lambton,
Bridge over Creek on County Road #12,
Twp. of Plympton,
Con. F.L.H., Lot 24, 25,
Structure Site No. 15-39.
Your File Ref. No. BA 1726

We have reviewed the above-mentioned report and
herewith submit our comments for your consideration:

It appears that a piled foundation is best suited
for the subsoil conditions if a structure rather than a flexible
pipe is used for the river crossing.

We feel that the most economical solution could be
obtained by driving H-piles to practical refusal that will occur
either in the hard silty clay stratum with grits and sand, or in
the underlying black shale bedrock. If 12 BP 53 H-piles are
used, a safe load of 60 tons can be attributed to the individual
pile.

The Consultant explains on pages 14 and 15, the method
and provides figures for the computation of the bearing capacity
of caissons. It appears to us that the addition of a frictional
component to the bearing capacity is not justified since the
ground will be loaded and will have to consolidate under the
additional load. In such a case, it seems that the taking into
account of a negative rather than a positive friction is more
justified.

Should there be any other queries, please feel free
to call on our Office.

AGS/MdeF
cc: Foundations Office
Gen. Files

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Materials & Research Section,
DOWNSVIEW, Ontario.

FROM: G.C.E. Burkhardt

DATE: December 20, 1963.

OUR FILE REF. BA1726

IN REPLY TO

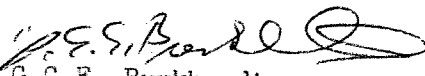
SUBJECT: County of Lambton
Bridge over Creek on County Road # 12
Twp. of Plympton
Con. F.L.H. Lot 24, 25
Structure Site No. 15-39

Attached please find one copy of the Foundation Report by E.M. Peto and Associates Ltd., for your comments.

The proposed structure is a three span (each 65 feet c. to c.) simply supported bridge.

We would appreciate it very much, if you could give your special attention to the allowable pile loads recommended in the foundation report.

GOEB/kd
c.c. J. Walter


G.C.E. Burkhardt
for K.L. Kleinsteinber
Municipal Bridge Liaison Eng.

~~SECRET~~
E. M. PETO ASSOCIATES LIMITED

Our Job No. 63177

1207 Caledonia Road,
Toronto 19, Ontario,
789-1126.

29th October, 1963.

The County of Lambton,
c/o James D. Nisbet, Consulting Engineer,
206 Water Street,
Sarnia, Ontario.

C3-244M

Gentlemen:

Re: Subsoil Investigation,
Ponnie Doon Bridge,
County Bridge C-12-1.

We have pleasure in submitting four copies of our report No. 63177 on the above site investigation; two additional copies have been forwarded directly to Mr. O van Deurs, Lambton County Engineer.

The character of the subsoil at the site is such that a variety of culvert or bridge types could be adopted, several of which are discussed in the report; the final choice will probably be made on the basis of considerations of hydraulics as well as economics. A flexible structure, capable of safely withstanding a maximum anticipated settlement of up to four inches, could be founded on shallow footings, while piles would be recommended in the case of a structure sensitive to settlement.

No apparent danger of embankment failure due to a weak foundation has been detected for the case of embankment extending not

STRUCTURE SITE No. 15-39

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

appreciably higher than to elevation 615. However, measures are recommended for the stabilization of the existing valley slopes in the immediate vicinity of the embankments; the present slopes show signs of recent slip failures, probably caused by undercutting by the stream.

While we consider the report to be comprehensive 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,

F. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

RK/vm

COUNTY OF LAMBTON,
c/o JAMES D. NISBET, CONS. ENG.

SUB-SOIL INVESTIGATION REPORT

of

BONNIE DOON BRIDGE,
COUNTY BRIDGE C - 12 - 1

F. M. PETO ASSOCIATES LIMITED,
1287 Caledonia Road,
TORONTO 19, Ontario.

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BOREHOLE LOGS

SITE PLAN

PROFILES

A. INTRODUCTION

The work described in this report was authorized verbally by Mr. James D. Nisbet, Consulting Engineer, on 16th September, 1963.

In connection with the proposed realignment of the County Road No. 12, a new crossing is to be constructed over a small creek. The location of the site is indicated on the key plan included on the appended drawing. The bridge is named Bonnie Doon Bridge, and is referred to as the County Bridge C 12-1.

The type of the new structure has not been decided as yet; reinforced concrete arch culvert or a multi-plate arch culvert of Armco type construction is considered, although a bridge-type structure may be adopted instead. The span is to be 50 ft. approximately, with a height above stream bed of about 25 to 30 ft.; accurate data on the new grade alignment was not available at the time of preparation of this report. Should a culvert design be adopted, the structure would be approximately 160 ft. long.

This Company was retained to carry out a subsoil investigation connected with the Project.

B. GENERAL INFORMATION

1. Four testholes were put down at the site, in the positions chosen by the Consulting Engineer, who kindly supplied a site sketch, on which the site plan included on the appended drawing has been based. The Geodetic ground elevations at the positions of the testholes, also provided by the consulting engineer, are included on the borehole logs and subsoil profiles.
2. The field work was performed by our drilling rig unit No. 3 between 17th and 24th September, 1963. Our standard drilling and sampling procedures were followed.
3. Laboratory testing of soil samples consisted of the following tests:
 - Water content determinations
 - Atterberg Limit tests
 - Unconfined compression tests, with volumetric analysisResults of the tests are included in the Appendix.

C. SITE DESCRIPTION

The site is located on County Road No. 12 in Concession No. XI, Plympton Township, within 1/4 mile of Lake Huron and approximately 1.7 mile north of Camlachie. A Key plan is included on the appended drawing.

The centre line of the proposed new road alignment is located approximately 500 ft. upstream of the existing crossing. The creek, which was only a few feet wide at the time of the field work, flows towards the north-west. The stream cuts into the western slope of the valley and evidence of slope failures was noted in the relatively steep valley walls.

D. OUTLINE OF GEOLOGY

The bedrock commences near elevation 541 and consists of a black, fissile shale, which is a part of the late Devonian or early Mississippian Kettle Point formation.

During the Pleistocene period, the bedrock was overlain by deposits of clay and clayey till, which are shown on the appended subsoil profiles. The lowest layer, immediately overlying the bedrock, consists

D. OUTLINE OF GEOLOGY - Cont'd.

of clayey sand and boulders and is possibly an Illinoian or early Wisconsin till. The overlying hard silty clay till was probably laid down during the first half of the Wisconsin glaciation, as suggested by its very high degree of preconsolidation.

The overlying deposit, referred to as the "stiff sandy clay with pebbles", probably originated during the later stages of the Wisconsin ice age. Overlying this stratum was a deposit of layered clay, having a thickness of 12 to 20 ft. This clay was probably deposited during the warmer Cary Fort Huron interval, when the glaciers retreated to the Georgian Bay area and the glacial Lake Arkona covered the Lake Huron and Erie Basin. The ice sheet advanced once more over this area and another till layer was deposited, referred to on the subsoil profiles as the "stiff silty clay, some pebbles", from which the present topography at the site has been carved. After the ice retreated, lakes covered the area from time to time, but probably because of the frequent water level changes, little sediment has remained, except the surficial sand layer, which was probably deposited in the glacial Lake Nipissing.

In two of the testholes, natural gas under high pressure was encountered in the lowest, sandy and gravelly till deposit. It is probable that the gas penetrated through fissures in the bedrock from the Silurian

D. OUTLINE OF GEOLOGY - Cont'd.

Guelph formation, which is estimated to be at a depth of 400 to 450 feet.

E. SUBSOIL CONDITIONS

A geological appreciation of the site was given in the previous chapter, while details of the subsoil conditions encountered in the testholes are entered on the borehole logs, which also contain the results of standard penetration tests and water content measurements.

Simplified subsoil profiles, in the form of sections through the testholes, are presented on the drawing and include the inferred levels of contacts between the various strata.

The main geotechnical properties of the subsoil are illustrated on Figure 1, which includes the distribution with depth of the standard penetration resistance (N-values), water content, Atterberg limits and undrained shear strength. In addition, the undrained shear strength was plotted against the water content on Figure 2.

The following additional comments regarding the subsoil conditions are offered:

1. The stiff silty clay with some pebbles, which is present

F. SUBSOIL CONDITIONS - Cont'd.

above the average elevation 571 and forms the walls of the valley, possessed a desiccated, mottled brown and grey crust, which was 4 to 6 ft. thick in testholes 1, 3 and 4 but was absent in testhole 2.

Below the valley floor, the average undrained shear strength of this deposit can be taken as 1,100 lb./sq. ft. The material is not considered to be very compressible, and provides adequate support for a flexible culvert-type structure, which can be regarded as capable of safely withstanding some settlement. Rigid frame types of structures sensitive to settlements should preferably be supported at a greater depth.

Where the silty clay is recovered in grade alteration excavations, it can be used as fill in embankment construction, but it must be considered as susceptible to frost heave, and a suitable granular cushion is necessary immediately below pavements.

2. The firm, sensitive, layered clay located between the average elevations 571 and 556 is the weakest deposit at the site. While the average undrained shear strength of the stratum is 1,000 to 1,200 lb/sq. ft. the strength in the top 5 ft. is limited to between 600 and 800 lb/sq. ft.

F. SUBSOIL CONDITIONS - Cont'd.

The clay is moderately compressible, and the value of the co-efficient of volume change, m_v , can be estimated to be of the order of 0.012 to 0.015 sq. ft./ton. However, consolidation tests were not performed on samples from this site, since the thickness of the compressible strata is not large, and while types of structures sensitive to settlement would in any event have to be supported on the hard, deep-seated deposits, the settlement of flexible structures supported at a shall depth, will be relatively uniform and in all probability limited to a tolerable amount.

3. The stratum of hard silty clay, commencing near the elevation 550, has an undrained shear strength in excess of 7,500 lb/sq. ft. and provides an excellent support for piles or caissons. The subsoil below this level can be considered as incompressible.

4. Randon pockets of natural gas under high pressure must be expected in the sandy till immediately above the bedrock, but the testholes indicated that these are localized. Ground water under artesian pressure was not encountered at this site.

F. SUBSOIL CONDITIONS- Cont'd.

5. The shale bedrock was proved by diamond drilling in two of the testholes to commence at elevation 541.1. Refusal at very similar elevation in the two remaining testholes indicated that the level of the bedrock surface is very uniform at the site.

Examination of rock cores indicated that the bedrock is in a very sound condition and forms an excellent medium for the support of end-bearing piles.

F. GROUND WATER AND NATURAL GAS

A summary of the ground water observations is included on the borehole logs. No free ground water was reported until the hard clay till was reached near elevation 550. Below this level, the ground water observations were obscured by the use of wash water to advance the testholes, but no seepage seams with significant yield were present.

Natural gas was struck immediately above the bedrock in testholes 1 and 4, where the rock was covered by a deposit of relatively pervious, sandy and gravelly till. In testhole 4, the gas was struck at elevation 545 and blew out of the hole to an estimated height of 25 to 30 ft. above ground level, carrying the water which was present in the testhole,

F. GROUND WATER AND NATURAL GAS - Cont'd.

as well as earth and stones. The drilling rig could not be approached by the drillers for two hours. The gas was still coming out of the hole, although at greatly reduced pressure, when the crew completed the field work and left the site the following day.

In testhole 3 natural gas was struck in a thin layer of the sandy till near elevation 542, but at considerably lesser pressure than in testhole 4. When set alight, it was reported to burn with a red flame. In testholes 1 and 2 natural gas was not encountered, although these holes penetrated to similar depth and were performed first.

G. CONCLUSIONS AND RECOMMENDATIONS

1. Culvert Foundations

According to the information supplied by the consulting engineers, the following alternative types of structure are being considered at this site:

- (a) Corrugated steel culvert
- (b) Reinforced concrete arch culvert
- (c) Bridge-type structure

The foundation considerations of each of the above alternatives will be discussed in turn.

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

(a) Corrugated Steel Culvert

The subsoil conditions are considered to favour this type of culvert. A closed, multiplate structure can be supported on a granular cushion, placed directly on the surficial layers of subsoil which consists of silty sand followed by stiff silty clay. The allowable bearing capacity of the subsoil at a shallow depth is estimated as 1.25 ton/sq. ft, with a factor of safety of about three.

However, all soft mud or organic topsoil should be removed and the compacted granular cushion below the culvert should only be placed after firm and undisturbed subsoil has been exposed. Any local soft pockets should preferably be excavated and replaced with additional thickness of granular fill. Also, any boulders present in the creek bed should be removed.

A thickness of compacted granular cushion below the culvert of one to two feet should be sufficient, if the actual subsoil conditions along the line of the culvert correspond to those indicated by the testholes. The width of the cushion should be equal to at least one and a half times the width of the culvert in order to provide a good base for the proper compaction of the backfill on the sides of the culvert.

G CONCLUSIONS AND RECOMMENDATIONS Cont'd.

The preparation of base and the compaction of the fill outside the culvert should be performed in accordance with the usual requirements of good practice, the main points of which can be summarized as follows:

Very firm support must be provided along the lower quarter of the circumference of the culvert. The bedding should be carefully shaped to accurately fit the shape of the structure. Best results for bedding corrugated metal structures are obtained by preparing a flat surface and carefully tamping the fill under the haunches.

It is advisable to seal the pervious granular fill under the pipe at the ends of the structure against the inflow of water. This can be done by bedding the ends in well-tamped clay or by including a head wall or end section.

The corrugated metal structure, being flexible, will bend under the embankment loads and deflect out horizontally, thus building up side support. In order to allow sufficient mobilization of lateral resistance, the back fill around the pipe should consist of good quality.

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

drainable material, free of rocks and organic matter. However, clay fill is acceptable, if available at a consistency which will allow good compaction. The fill under and around the structure should be placed in layers not exceeding six inches at a time, or as necessary to ensure compaction to 100% of Standard Proctor value, which should be the compaction criterium to be aimed at. The height of fill on both sides of the culvert should be built up simultaneously.

The fill above the crown of the culvert should be of equally good quality and standard of compaction as on the sides of the structure.

If a multi-plate type of structure consisting of two roof sections with central support is considered, the side walls and the central support would have to be founded on reinforced concrete footings, unless a common mat foundation proves economic. Provided that the foundations are not placed lower than at elevation 575, the safe bearing capacity is estimated as 1.5 ton/sq. ft. Below this level, the safe bearing pressure decreases with depth, to a value of 1.0 ton/sq. ft. at and below elevation 573. Considerably higher bearing pressure is permissible only below elevation 555. The minimum depth of cover will be governed by requirements for protection of the footings against frost action and potential scour.

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

It is assumed that the types of structures discussed above can safely withstand a settlement which has not been analysed in detail but is believed not to exceed four inches.

(b) Reinforced concrete arch culvert

According to the information supplied by the consulting engineers, if a reinforced concrete arch culvert is adopted, it would have a span of 50 ft, with a rise in arch of 20 ft. above the stream bed.

The subsoil conditions are considered to be less favourable for the support of this type of structure on spread footings placed at a shallow depth, on account of the presumably greater sensitivity to the anticipated settlements of the order of up to four inches. Although the settlements could be expected to be relatively uniform, nevertheless, we have doubts whether it would be prudent to support a concrete arch culvert with a span as large as 50 ft. on spread footings under the present conditions. If the anticipated settlement is considered within tolerable limits, we would recommend the adoption of somewhat more conservative footing pressure than for the case of a corrugated metal

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

culvert. At elevation 575, the safe bearing pressure for this type of construction is considered to be 1.25 ton/sq. ft., reducing with depth to 1.0 ton/sq. ft. at elevation 573. The above are the net bearing capacity values; an overburden component, equal to the least weight of overburden above the footing (taking into account the maximum anticipated scour depth) can be added.

In our opinion, a considerably more sound design would provide for supporting the reinforced concrete arch culvert on a pile foundation. A suitable medium for the support of piles is the hard silty clay till, or the very dense gravelly and sandy till, and the piles could penetrate to any depth between elevation 550 and the surface of bedrock at elevation 541.

Various types of piles could be considered, both prebored, cast-in-situ, and driven displacement piles. The site may be regarded as attractive for using caisson-type foundation, on account of the high end-bearing capacity of the lower horizons of subsoil. The allowable end bearing pressure is estimated as 12.0 ton/sq. ft. of the cross sectional area of the pile toe between elevation 550 and the surface of bedrock at

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

elevation 541, while up to 20.0 ton/sq. ft. is considered safe for caissons resting on the bedrock surface; these figures include a factor of safety of at least two. In addition to the end-bearing value, a frictional component of bearing capacity can be included in the design, equal to the product of the adhesive strength of clay on pile and the surface area of the pile shaft below elevation 580. The average adhesion can be taken as 0.2 ton/sq. ft.; this value includes a factor of safety of about two.

Although natural gas may be released in the caisson or pile pre-borings, it is expected that it would dissipate after some hours. The testholes did not encounter significant ground water and lining may prove unnecessary, but all risks would probably be eliminated if the pre-bored piles would not penetrate below elevation 550.

Displacement piles would encounter considerable resistance in the hard tills below elevation 557, and refusal may be reached above the bedrock with some types of piles and driving equipment. Steel H piles would probably be most appropriate should it be desired to reach the bedrock, but the toes of the piles should be reinforced to prevent damage.

G. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.

Because of the apparent uniform bedrock surface level, the length of end-bearing piles can be accurately specified.

It would be advisable to obtain information on the possible corrosive effects of the natural gas present in the area on steel and concrete piles.

The backfilling behind the culvert has been discussed in Item (a).

(c) Bridge-type structure

The foundation considerations for a bridge-type structure are similar as for a reinforced concrete arch culvert, and the values of allowable bearing capacity at the various levels, quoted in Item (b) are applicable. If the bridge superstructure can safely withstand settlement of abutments of up to four inches, then shallow footings may be permissible. On the other hand, a superstructure sensitive to settlements should be supported on piles, the bearing capacity of which is given above.

G. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.

2. Embankments and Slope Stability

Detailed survey of the site and realignment data was not available so that accurate investigation of the stability of the proposed embankment could not be performed; however, an approximate analysis of the embankment foundation stability was carried out, based on the subsoil profile through testholes 1 and 3, and assuming that the fill in the lowest section of the valley will extend not higher than elevation 615. The analysis indicated no apparent danger of slip failure of subsoil below the embankment, and a more detailed analysis does not appear justified. The stability will improve with time after construction, as the softer layers of subsoil consolidate under the weight of the embankment. However, in order to prevent the occurrence of possible toe failures due to a localized build-up of pore water pressures, it is recommended to place the fill over a period of several weeks, rather than in one rapid operation, so that the pore pressures set up by the loading of the subsoil may have a chance to dissipate. It is further recommended to place the fill commencing with the toe of the embankments and to work inward towards the centre line.

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

The potential settlement of the subsoil below the weight of the proposed fill has not been analysed in detail, but is not expected to exceed four to six inches. Additional settlement may occur within the embankment itself, the magnitude depending on the type of fill and the quality of compaction. More detailed analysis was not considered justified as it will be of minor importance and should be complete within one or two years. The final road surface should be laid after most of the vertical movement has ceased.

Comments regarding the use of local soil in the embankment construction were made on page 6.

Evidence of recent slip failures of the existing valley slopes has been observed; one such failure was evident directly opposite testhole 4. The failures were probably caused by the undercutting of the valley walls by the stream, the bed of which meanders across the floor of the valley from one slope to the other. Analysis of stability of the slopes is outside the scope of this investigation and would require additional testholes, laboratory testing and detailed survey of the slopes. However, it is considered that the stability will be improved when the

G. CONCLUSIONS AND RECOMMENDATIONS Cont'd.

stream becomes realigned through the culvert and is prevented from undercutting the valley walls. It would also be advisable to regrade the slopes in the immediate vicinity of the embankments to a gradient of 2. 5: 1, horizontal to vertical, and to protect the slopes with sodding and surface drainage measures.

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/vm

Report No. 63177.

October, 1963.

LABORATORY TEST RESULTS

ATTERBERG LIMIT TEST RESULTS

Soil Type	E. H. / Sa. No.	Depth ft.	Liquid Limit, %	Plastic Limit, %	Plasticity Index	In situ water content
Stiff silty clay, some pebbles	1 / 3	7	34.0	16.5	17.5	20.9
	4 / 5	8	25.5	16.2	19.3	17.3
Firm, sensitive layered clay	1 / 13	16	49.5	22.4	27.1	37.3
	2 / 20	22	42.6	19.7	22.9	36.7
Stiff sandy clay with pebbles	1 / 23	31	29.8	16.9	12.9	18.4

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UNCONFINED COMPRESSION TEST RESULTS

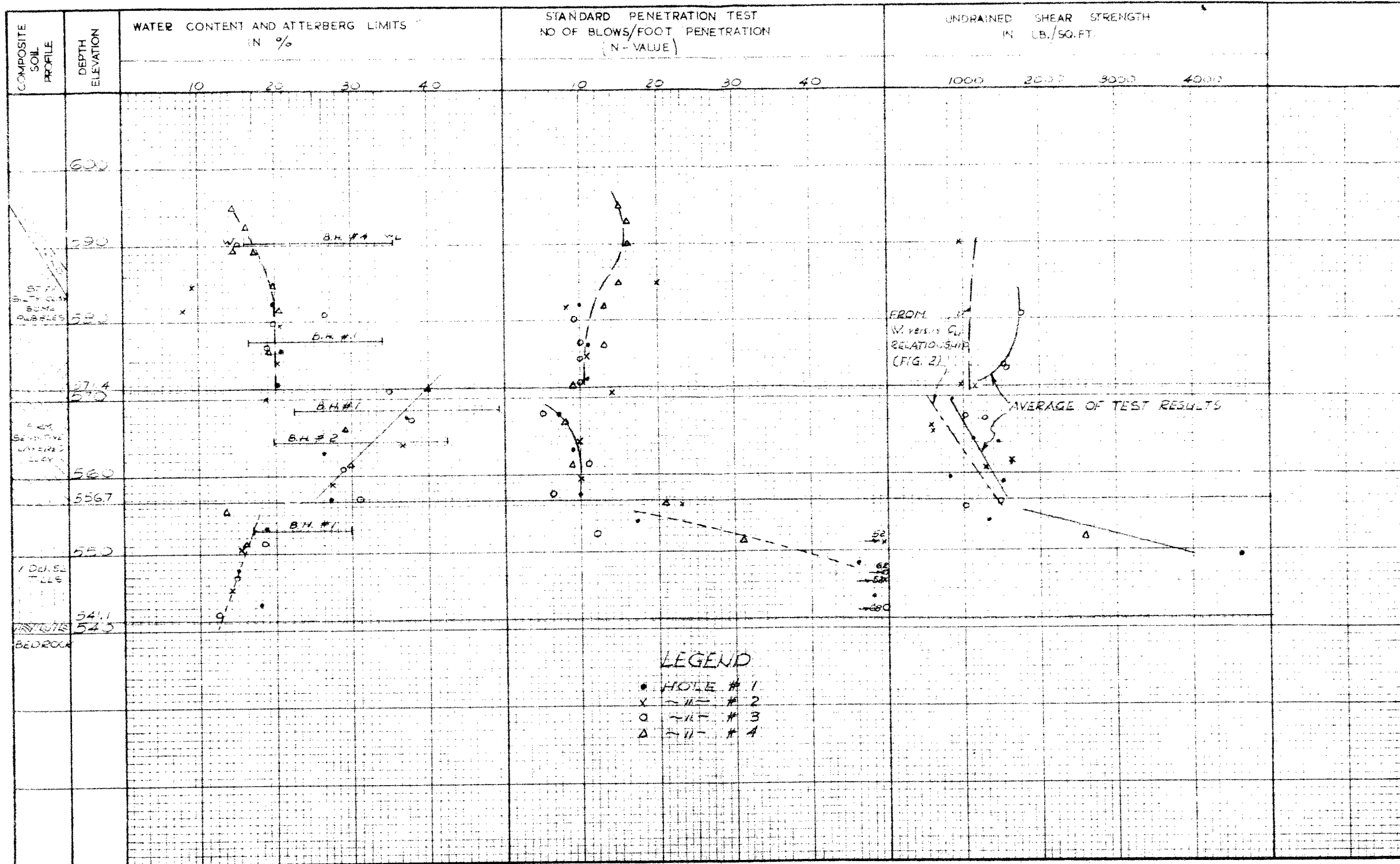
P. H. No.	Sample No.	Depth, ft.	Nat. M. C. %	Densities, p.c.f.		Void Ratio, e	u/c Shear Strength p.s.f.	% Strain at Failure
				Wet	Dry			
1	23	20'0"-21'6"	15.0	157.0	119.0	0.42	1384	
1	24	25'0"-26'6"	10.6	144.0	130.0	0.30	7500	
2	3	2'0"-2'6"	26.5	122.5	97.0	0.74	1772	
4	5	7'0"-8'6"	14.6	138.0	120.5	0.40	975	
4	20	45'0"-46'6"	14.6	142.0	124.0	0.39	2592	
5	12	16'0"-16'6"	29.9	118.0	91.0	0.81	1054	
11	13	16'6"-17'0"	25.4	122.0	97.4	0.73	1297	
2	21	27'0"-27'6"	28.2	118.0	92.0	0.83	1460	
11	22	27'6"-28'0"	28.6	119.2	92.7	0.82	1005	
2	12	15'0"-15'4"	17.5	130.0	110.5	0.52	973	
2	14	15'4"-15'8"	18.7	129.0	103.4	0.55	1168	
2	13	20'3"-21'0"	37.0	116.0	85.0	1.00	568	
2	19	21'0"-21'6"	37.1	113.0	82.5	1.04	584	
1	12	14'8"-15'0"	29.1	115.0	89.0	0.89	843	
1	21	25'8"-26'0"	21.4	120.0	98.8	0.70	1508	
1	20	25'4"-25'8"	32.2	122.5	92.5	0.87	973	
3	7	9'2"-10'0"	17.4	130.0	110.5	0.52	1523	
3	3	10'0"-10'4"	13.5	130.5	115.0	0.46	1572	
1	16	19'8"-20'0"	25.4	121.0	96.5	0.75	1130	20.0
1	17	20'0"-20'4"	28.2	120.0	93.5	0.80	1460	14.0
2	23	25'8"-26'0"	24.4	124.0	99.5	0.69	1620	7.5
2	24	26'0"-26'4"	22.9	124.0	101.0	0.67	1300	9.0

GEOTECHNICAL SOIL PROPERTIES

APPENDIX

FIG.

1


 DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT

U.S. GEOLOGICAL SURVEY
WATER RESOURCES DIVISION
ALBUQUERQUE, NEW MEXICO

359.71

UNDRAINED SHEAR STRENGTH, c_u lb./sq. in.

RELATIONSHIP BETWEEN UNDRAINED SHEAR STRENGTH & WATER CONTENT

LEGEND

- x FIRM, SENSITIVE LAYERED CLAY
- STIFF SILTY CLAY, SOME PEBBLES

$W_L = 35$
 $I_P = 18$

$W_L = 35$
 $I_P = 24$

WATER CONTENT, w %

CB # 63177 FIG. 2
e.m. petro associate, ltd
OCT. 1963
A.K.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

e. m. peto associates ltd.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

BOREHOLE LOG

Borehole No. 2
Boring Date Sept. 19 & 20, 1963
Checked By S. B.

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

[illegible]

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT





BOREHOLE LOG

Bonnie Doon Bridge.
 Job Name County Bridge C-12-1
 The County of Lambton
 Client City of S. D. Nisbet Const. Eng
 Elevation 583.41

Job No. 63177
41
Casing
Compiled By R. K.

Borehole No. 3
Boring Date Sept. 20, 21, 1963.
Checked By S. B.

SAMPLE CONDITION

	UNDISTURBED
	FAIR
	DISTURBED
	LOST

SAMPLE TYPE

A.S. AUGER SAMPLE
C.S. CASING SAMPLE
S.L. 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

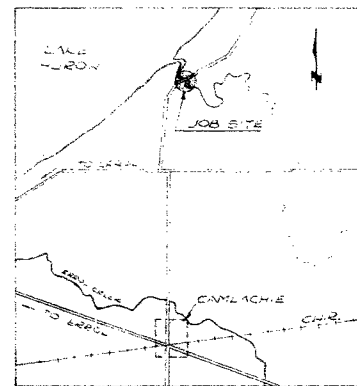
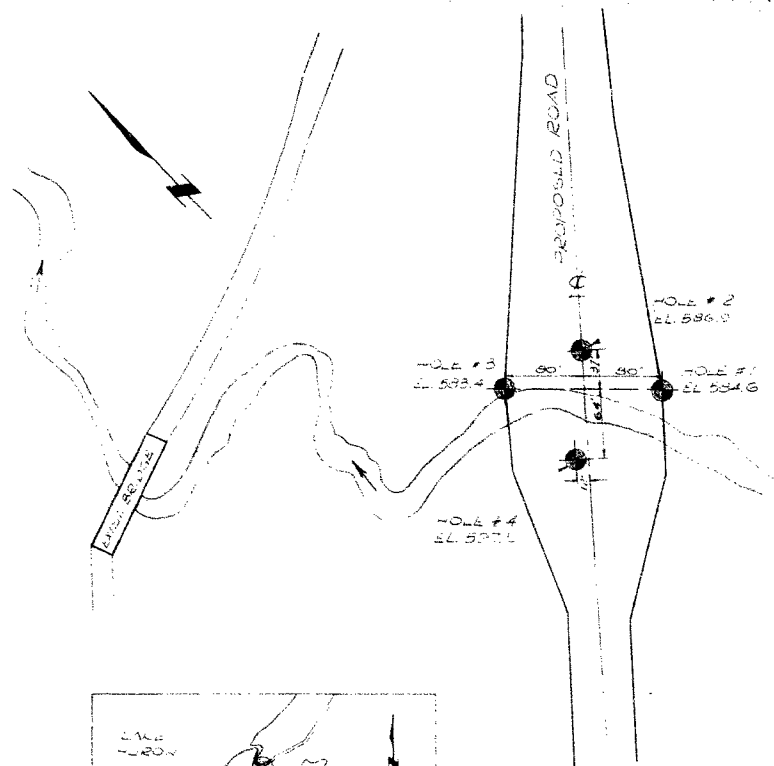
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Borehole No. 4
Boring Date Sept. 23, & 24, 1963
Checked By S. B.

ABBREVIATIONS

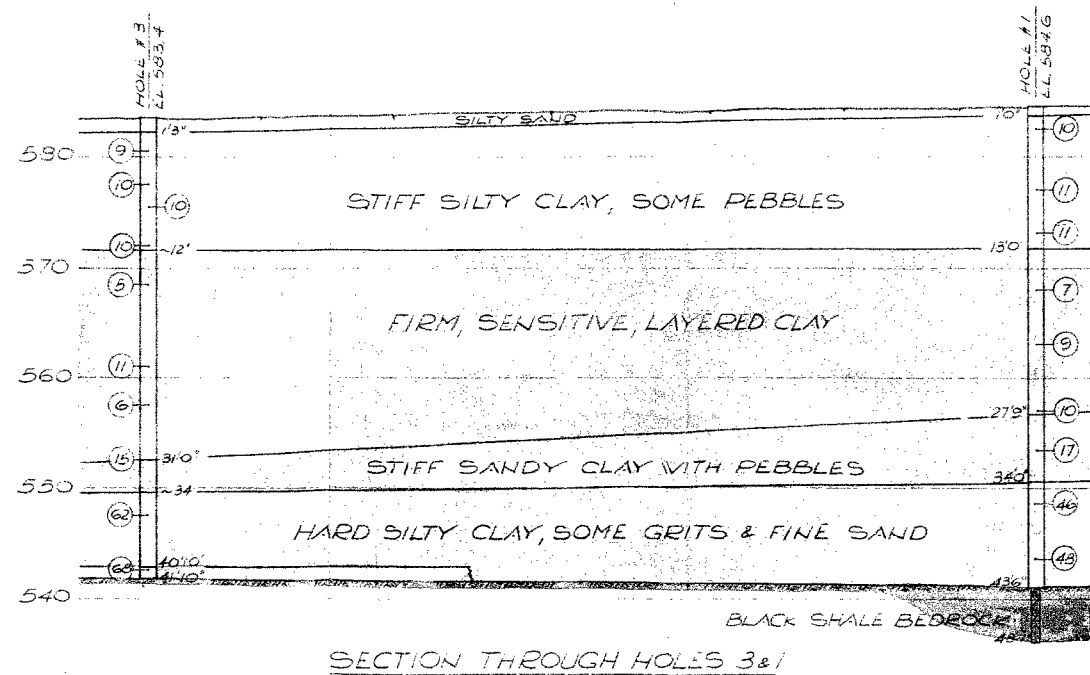
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M.	MOIST
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D.T.F.L.	DRIER THAN PLASTIC LIMIT
P.	ABOUT PLASTIC LIMIT

[illegible]



KEY PLAN
1 mi. = 1 1/4 in.

SITE PLAN
SCALE: 100' TO 1" (APPROX.)



SECTION THROUGH HOLES 3 & 1

LEGEND

- BOREHOLE
- BLOW 5/FOOT S.P.T.

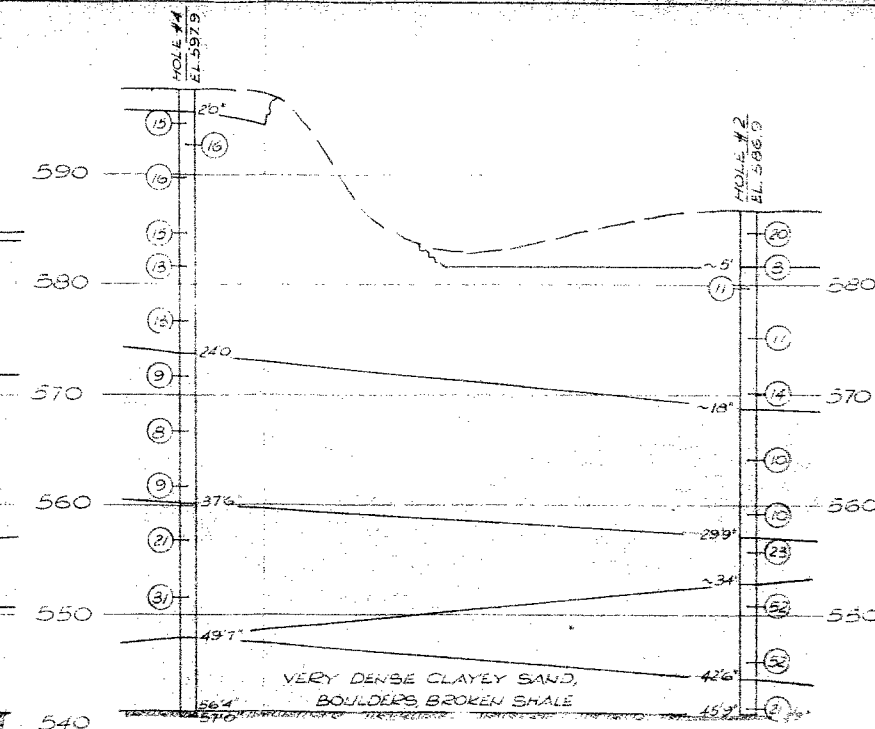
NOTES:

- a) SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.
- b) BOREHOLE ELEVATIONS HAVE BEEN SUPPLIED BY JAMES D. NISBET, CONS. ENGINEER

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.



HOR.: 20' TO 1"
SECTION SCALES
VERT.: 10' TO 1"



SECTION THROUGH HOLES 4 & 2

THE COUNTY OF LAMBTON
By JAMES D. NISBET, CONSULTING ENGINEER

BONNIE DOON BRIDGE
(COUNTY BRIDGE # C-12-1)

PREPARED BY
e.m. peto associates Ltd.

JOB NO. 63177	DATE OCT. 1963	DWN. BY: K.K.	CHECKED BY: RK
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DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT