

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

August 17, 1960.

D.E.C. FOUNDATION INVESTIGATION

W.P. 29-61 -- W.J. 60-P-65.

Attention: Mr. E. McCombie.

Re: Proposed 15-Mile Creek Bridge -
5 Miles West of St. Catharines, Ont.
District No. 4.

Accompanying this memo, is our detailed report on
the subsoil conditions existing at the above site.

The conclusions and recommendations to be followed
in your future design work, are summarized in the report,
and are self-explanatory.

Should any questions arise in connection with this
project that you would like to discuss, please do not hesitate
to contact our Office.

LGS/Hief
Attach.

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsey
I. Campbell
R. E. Richardson
T. J. Lovich
A. Watt

Foundations Office
Gen. Files. ✓

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DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

FOUNDATION INVESTIGATION

At

Site of the Proposed 15-Mile Creek Bridge
5 Miles West of St. Catharines, Ontario.
W.P. 29-61 -- W.J. 60-7-65 -- District 4.

1. INTRODUCTION:

About 5 miles west of St. Catharines, the Q.E.W. crosses on a causeway, a strip of water known as the 15-Mile Creek. A bridge is located on this causeway roughly halfway across. It is proposed to construct a service road on the north side of the Q.E.W. extending from Gregory Road to Sixteen Mile Creek. The proposed service road will be parallel to the Q.E.W., and distant from it 110' centre line to centre line. A structure will be built to span the watercourse of 15-Mile Creek, which is some 40' wide at this point.

A total of four boreholes, and eight penetration tests was carried out, at this location, the purpose of which was:-

(1) To determine the subsoil conditions existing at the site of the new proposed structure.

(2) To determine the subsoil conditions existing at the site of the approaches for the proposed structure.

2. DESCRIPTION OF THE SITE:

The site is located some five miles west of St. Catharines on the north side of the Q.E.W. It consists of a strip of water about 600' wide. During construction of the Q.E.W. the area was backfilled for a distance about 250' either side of centre line of the boulevard, to an elevation about six feet above waterlevel, leaving only a narrow water course some forty feet in width. The existing bridge for the Q.E.W. spans this watercourse, and is

2. DESCRIPTION OF THE SITE: (cont'd.) ...

founded on piles. Average height of the approaches for the Q.E.W. is 15' above the level of the main backfilled area.

The centre line of the proposed service road is about 110' north of centre line of the Q.E.W. This means that the entire construction of the proposed bridge and approaches will take place upon the above mentioned backfilled area except for a small section at the east end of the east approach. At the site of the proposed bridge the ground is fairly flat, and is about 3' above the level of the water in the creek. The direction of flow of the water in the creek is north to Lake Ontario, the main shoreline of which, is approximately 1000' north of the Q.E.W.

3. FIELD INVESTIGATION PROCEDURE:

A total of four borings was carried out using conventional diamond drilling equipment adapted for soil sampling purposes. The holes were cased to 10' below centre line with MK pipe, and from then on no further casing was required until borings reached the bedrock level.

Samples were recovered in the disturbed state using a 2-inch O.B. split spoon. Undisturbed samples were recovered using 2-inch I.B. Shelby tubes. The split spoon samplers were driven into the soil with a driving energy of 350 ft. lbs. per blow. In all cases the Shelby tubes were pushed by hand.

Wherever possible, in-situ vane tests were performed, to determine the shear strength of the clay deposits at elevations 12" below the bottom of the various sample depths.

Dynamic cone penetration tests were carried out adjacent to the four borings prior to boring operations, and also at four other locations.

3. FIELD INVESTIGATION PROCEDURE: (cont'd. ...)

Driving energy to advance the cone was 350 ft. lbs. per *blow*.

Waterlevel observations were taken in each borehole as the work progressed, and the waterlevel in each hole was recorded twenty-four hours after completion of boring.

The locations, and elevations of the boreholes shown on Drawing 60-F-65A, which forms part of this report, were established by the Field Engineer.

A P.M. at Elev. 267.04' - cut cross on north end of south wingwall of existing bridge - was used to establish ground elevations at locations of all boreholes, and penetration tests.

4. LABORATORY INVESTIGATIONS:

Tests were carried out in the laboratory on a selection of samples to determine the following properties:-

- (1) Atterberg Limits.
- (2) Natural Moisture Content.
- (3) Bulk Density.
- (4) Undrained Shear Strength (a) - Undisturbed
(b) - Remoulded

Complete results of all laboratory tests are shown in Appendix I of this report.

5. DESCRIPTION OF SOIL TYPES AND SOIL CONDITIONS:

5.1 General:

Detailed descriptions of the various soil types encountered in each boring are given below, and are also shown in Appendix I of this report.

5. DESCRIPTION OF SOIL TYPES AND SOIL CONDITIONS (contd.) ...

The estimated stratigraphical profiles of Drawing 60-F-85A are based upon this information.

From ground level downward the various soil types are as follows:

5.2 Fill Material:

This material covers the entire site to a depth of about 6.0'. Apparently it has been placed during the construction of the approaches for the existing bridge on the Q.E.W. It consists mostly of a brown oxidized silty clay containing some sand, and gravel in varying quantities. The consistency of the material varies somewhat from medium to stiff. Due to the presence of gravel, and sand, however, it was not possible to perform vane tests to measure the in-situ shear strength. Penetration tests give an average of 7-8 blows per foot. It is estimated that the shear strength of this material is 700-800 p.s.f.

5.3 Soft-Medium Organic Clay Silt of Intermediate Plasticity:

This material immediately underlies the fill described above, and extends for an average depth of 35.0' over the entire area. It is highly organic in content, and some organic particles are easily visible to the naked eye. Field vane tests gave a shear strength of about 650 p.s.f. at the surface of the stratum, increasing with depth, to about 1000 p.s.f. in a distance of twenty feet. Remoulded vane tests indicated an average sensitivity of about 5.0. Undrained triaxial tests performed in the laboratory on the undisturbed samples gave shear strength results which were on the average, 200 p.s.i. less than the Field vane tests. However, undrained triaxial tests performed on remoulded samples gave results which corresponded very well with the Field vane remoulded tests. In view of these

5. DESCRIPTION OF SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.3 Soft-medium Organic Clay Silt of Intermediate Plasticity: (cont'd.) ...

In fact it can be assumed that the undisturbed samples had in fact suffered some degree of disturbance before being tested in the triaxial machine. This would of course result in lower values for the undrained shear strengths. Therefore, it seems highly likely that the Field vane tests give a more accurate picture of the shear strength conditions than the undrained triaxial tests performed in the laboratory.

Average value of the bulk density of this material was about 130 p.s.f. This is fairly low, but is explained by virtue of the high moisture content which was about 50 - 60%.

Complete details of all physical properties of the material, determined by laboratory tests, are given in Appendix I of this report.

5.4 Stiff Clay Containing Gravel & Sand (Glacial Till):

This is a heterogeneous mixture of stiff grey silty clay, fine - coarse sand, and fine - coarse gravel. It immediately underlies the organic clay silt stratum, and averages about four feet in depth. Average value of penetration resistance is about 20 blows per foot, and it is estimated that the shear strength is in excess of 3000 p.s.f.

5.5 Bedrock:

This consists of red shale containing layers of grey shale. It immediately underlies the stiff clay stratum, and was proved in B. H's #1 and #2 by taking 10' of rock core. At other locations the elevation of the top of this stratum was determined by driving penetration cones to refusal. Average depth to bedrock over the whole site is about 40'.

5. DESCRIPTION OF SOIL TYPES AND SOIL CONDITIONS: (cont'd.) ...

5.6 Ground Water:

The water table over the entire site is at Elev. 247.0', which is the same as the waterlevel in the creek at the location of the proposed structure.

6. DISCUSSION AND RECOMMENDATIONS:

Proposals for this location deal with the construction of a bridge to span 15-Mile Creek, and with the approaches. Particular problems dealt with in this report are discussed under the appropriate headings.

6.1 Structure Foundations:

The soft to medium stiff organic clay silt in this area has a shear strength which is inadequate to provide suitable bearing capacity for a spread footing type of foundation. In view of this it is recommended that the structure be supported on H piles driven to bedrock. Elevations of bedrock surface are given on Drawing 60-P-65A in order that the required length of piling may be estimated. A design load of 60 tons per pile will be suitable in this case.

6.2 Structure Approaches:

Examination of the existing ground profile at the location of the proposed structure shows that most of the structure approaches will have an embankment height of 15' or less if, as is assumed, the profile grade at the centre of the proposed structure is at Elev. 265.0'. Between Sta. 12/30, and 13/10 the height of the embankment will be about 20'. In view of these facts it is not anticipated that any major stability problems will occur. If surplus material from the cut between St 1400 and 12/00 is

- 7 -
6. DISCUSSION AND RECOMMENDATIONS: (cont). ...

6.2 Structure Approaches: (cont) ...

available it should be disposed of by filling up the low lying area to the right of Sta. 13/00 to Elev. 253.0' for a distance about 100' from centre line. This would increase the stability of the 20' high embankment. It is emphasized that all fill material must be thoroughly compacted during construction, and strict control must be maintained during this operation.

7. SUMMARY:

With regard to the proposed structure at the 15-Mile Creek, the following recommendations are made:-

(1) The structure should be founded on H Piles driven to bedrock. A design load of 60 tons per pile will be suitable.

(2) No major stability problems are anticipated during, and after construction of the approach embankments, provided that the heights of these do not exceed the assumed heights mentioned in 6.2. Surplus material from the cuts should be disposed of as outlined in 6.2.

8. MISCELLANEOUS:

This investigation was carried out during the period July 20th - 28th 1960, under the supervision of Mr. K. G. Selby of this section. Equipment used was owned and operated by the Department of Highways.

August 1960.

REPORT PREPARED BY: K. Selby,
Project Foundation Engr.

REPORT APPROVED BY: A. Steruac,
Foundations Office Engr.

JOB 60-P-65

W.P. 29-61

SUMMARY OF FIELD & LABORATORY TESTS

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETN RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT PCF	REMARKS
1	S1	3'-4.5'	Clay - med. stiff - oxidized (Fill)	6	-	-	-	-	-	
	S2	5.5'-7'	" " " "	5	-	-	-	-	-	
	T3	10'-11.5'	Soft-med. org. clay silt of intermediate plasticity-dark in colour.	P	46.3	59.4	130.0	230	87	
	VANE	13'		-	-	-	-	640	-	Sens: 4.9
	T4	15'-16.5'	" " " "	P	-	-	-	-	-	
	VANE	18'		-	-	-	-	800	-	Sens: 4.0
	T5	20'-21.5'	" " " "	P	67.9	34.2	53.8	625	90	
	VANE	23'		-	-	-	-	1040	-	Sens: 5.2
	T6	25'-26.5'	" " " "	P	-	-	-	-	-	
	VANE	28'		-	-	-	-	960	-	Sens: 6.0
	T7	30'-31.5'	" " " "	P	45.4	24.9	43.2	540	112	
	VANE	33'		-	-	-	-	1200	-	Sens: 3.8
	T8	35'-36.5'	" " " "	P	-	-	-	-	-	
	VANE	38'		-	-	-	-	1080	-	Sens: 3.4
	S9	40'-41.5'	Clay-sand-gravel stiff (Glacial Till)	23	-	-	-	-	-	
	BC10	44'-55'	Red shale-upper two feet weathered	-	-	-	-	-	-	
2	S1	3'-4.5'	Clay, med.-stiff, oxidized (Fill)	6	-	-	-	-	-	

Cont'd. P. 2

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-P-65

W.P. 29-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETN RESIST BLOWS FT	MOIST. CONT %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT PCF	REMARKS
2	92	6'-7.5'	Clay, med.-stiff, oxidized (Fill)	5	-	-	-	-	-	
	T3	10'-11.5'	Soft-med. org. clay silt of intermediate plasticity, dark in colour.	P	83.2	-	-	250	88.6	
	VANE	13'		-	-	-	-	640	-	Sens: 10.7
	T4	15'-16.5'	" " " "	P	54.5	42.7	93.4	-	-	
	VANE	18'		-	-	-	-	1000	-	Sens: 3.6
	T5	20'-21.5'	" " " "	P	-	-	-	-	-	
	VANE	23'		-	-	-	-	1280	-	Sens: 6.4
	T6	25'-26.5'	" " " "	P	44.7	27.0 24.1	40.3 35.2	530	112.0	
	VANE	28'		-	-	-	-	>2000	-	
	T7	31.5'-33'	" " " "	P	-	-	-	-	-	
	VANE	33'		-	-	-	-	>2000	-	
	T8	35'-36.2'	" " " "	P	26.7	20.7	30.4	1300	122.0	
	RC9	39'-49'	Red shale-upper two feet weathered.	-	-	-	-	-	-	50% Recovery
3-5			Cone Penetrations Only							
4	S1	3'-4.5'	Clay, med., oxidized (Fill Material)	14	-	-	-	-	-	
	S2	6'-7.5'	" " " "	7	-	-	-	-	-	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-7-65

W.P. 29-61

WELL NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETROMETER RESIST. BLOWS/FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH (PSF)	UNIT WEIGHT (PCF)	REMARKS
6	VANE	9'		-	-	-	-	960	-	Sens: 2.2
	T3	10'-11.5'	Soft-med. org. clay silt of intermediate plasticity, dark in colour.	P	43.2	25.6 27.2	42.1 47.3	425	109	
	VANE	13'		-	-	-	-	720	-	Sens: 6.0
	T4	15'-16.5'	" " " "	P	-	-	-	-	-	
	VANE	18'		-	-	-	-	640	-	Sens: 4.0
	T5	20'-21.5'	" " " "	P	46.7	27.2	48.8	520	109	
	VANE	23'		-	-	-	-	960	-	Sens: 4.8
	T6	25'-26.5'	" " " "	P	-	-	-	-	-	
	VANE	28'		-	-	-	-	1360	-	Sens: 4.3
	T7	30'-31.5'	" " " "	P	29.8	34.1	64.5	730	100	
	VANE	33'		-	-	-	-	1440	-	Sens: 3.6
	T8	35'-36.5'	" " " "	P	-	-	-	-	-	
7	VANE	38'		-	-	-	-	1120	-	Sens: 4.7
	S9	42'-47.5'	Clay, sand, gravel, mix-stiff (Glacial Till)	15	-	-	-	-	-	
7	31	3'-4.5'	Clay, med.-stiff, oxidized fill material.	4	-	-	-	-	-	
	32	6'-7.5'	Soft med. org. clay silt of intermediate plasticity, dark in colour.	4	-	-	-	-	-	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-P-65

W P 29-61

SOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENETN RESIST. BLOWS FT	MOIST CONT %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT PCF	REMARKS
7	VANE	9'		-	-	-	-	1080	-	Sens: 2.3
	T3	10'-11.5'	Clay, med.-stiff, oxidized fill material.	P	-	-	-	-	-	
	VANE	13'		-	-	-	-	960	-	Sens: 4.8
	T4	15'-16.5'	" " " "	P	29.2	50.4	99.0	425	92	
	VANE	18'		-	-	-	-	1040	-	Sens: 4.3
	T5	20'-21.5'	" " " "	P	-	-	-	-	-	
	VANE	23'		-	-	-	-	1200	-	Sens: 5.0
	T6	25'-26.5'	" " " "	P	54.6	29.5 28.0	45.3 41.9	640	99	
	VANE	28'		-	-	-	-	1440	-	Sens: 3.6
	S7	31'-32.5'	Clay, sand, gravel mix, stiff (Glacial Till)	23	-	-	-	-	-	
8			Cone Penetration Only							
			S denotes Split Spoon Sample T denotes Shelby Tube Sample RC denotes Rock Core Sample							

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 29-61

BORE HOLE NO. 1

JOB 62-7-63.

STATION 15+65 18' Rt.

DATUM G. S. Co.

COMPILED BY B. K.

BORING DATE July 26/60. CHECKED BY K. G. S.

2" DIA. SPLIT TUBE
2" SPLIT TUBE
2" SPLIT TUBE
2" DIA. CORE
2" DIA. CORE
2" DIA. CORE
2" DIA. CORE
2" DIA. CORE

LEGEND

1/2 UNCONFINED COMPRESSION (QU) — 0
VANE TEST (C) AND SENSITIVITY (S) — 0
NATURAL MOISTURE AND LIQUIDITY INDEX — 0
FLOW LIMIT — 0
PLASTIC LIMIT — 0

DEPTH FEET	ELEV. FEET
0	251.0
10	247.0
20	244.0
30	242.5
40	207.0
50	196.0
60	196.0

Groundlevel

Clay-medium-stiff-oxidised (Till) M. in

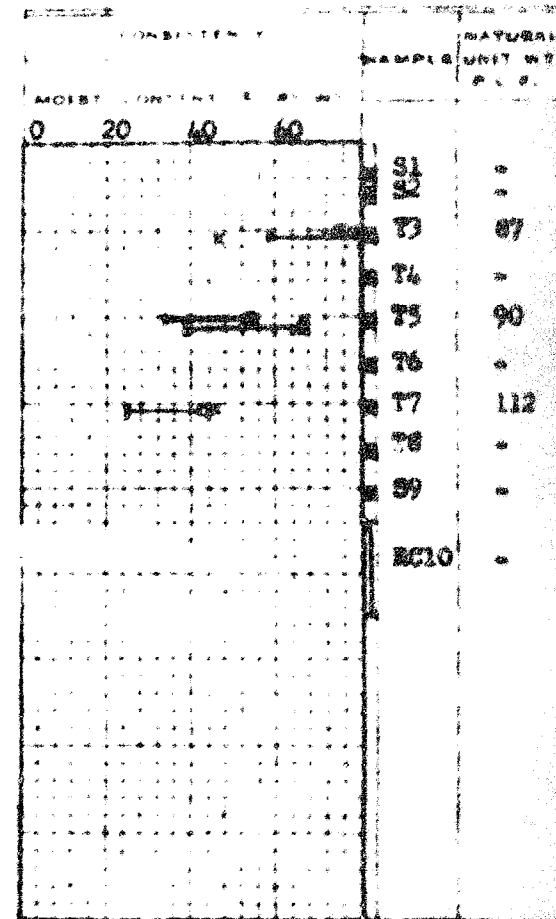
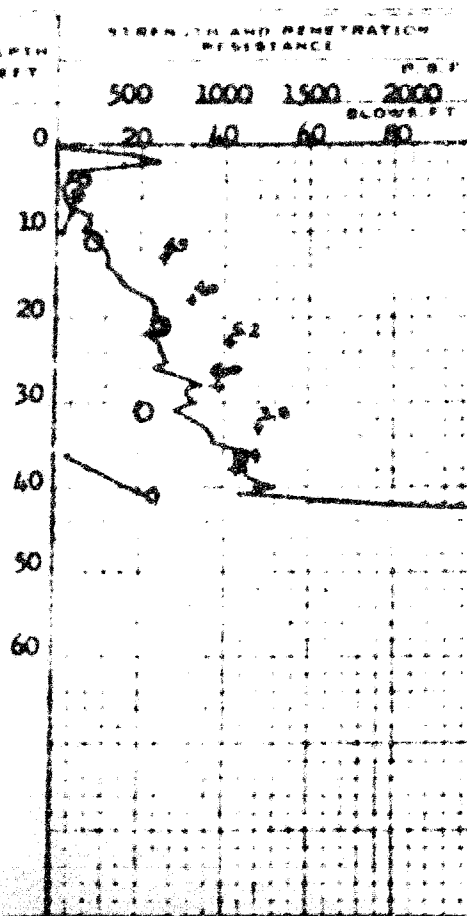
Soft-medium/organic clay silt of intermediate plasticity-dark in colour.

Clay-sand-gravel-stiff (Glacial Till)

Red Shale-upper two feet weathered

End of Borehole

Waterlevel: 247.0



Penetration Resistance Profile shown obtained by driving a 2" dia. cone from ground surface to depth noted. Cone driven with energy equal to 350 ft./lbs. per blow.

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 29-61

BORE HOLE NO. 2

JOB 60-P-65

STATION 14+75 18' Left

DATUM G. S. C.

COMPILED BY B. K.

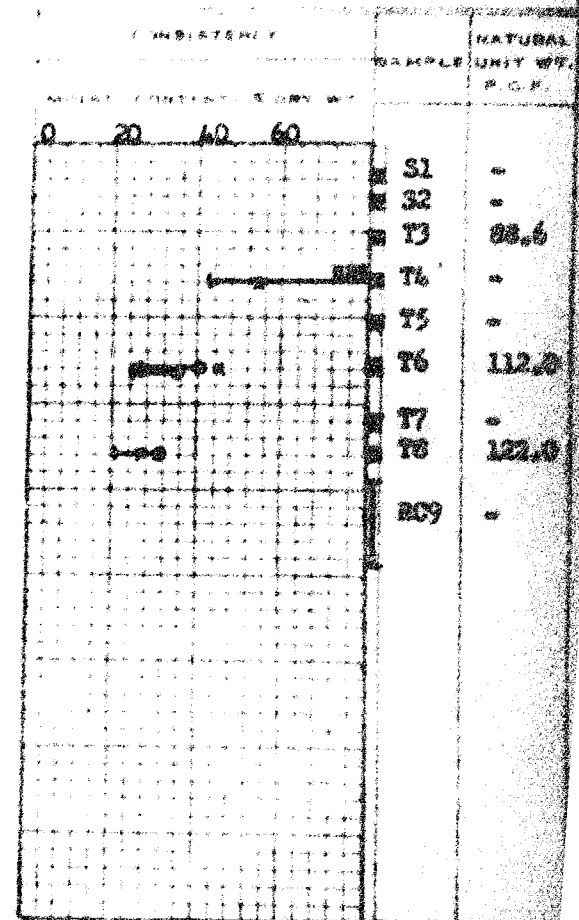
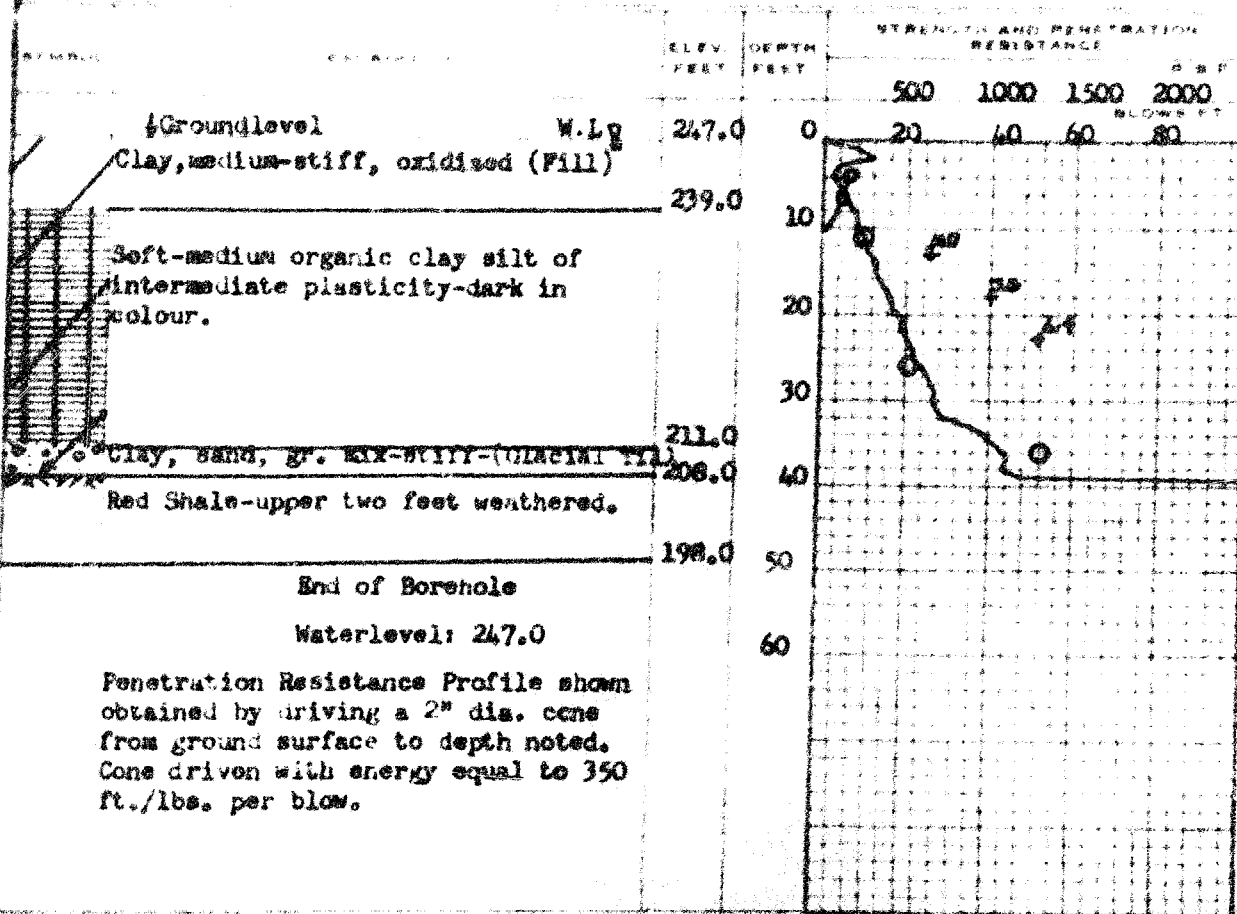
BORING DATE July 26/60.

CHECKED BY K. G. S.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

UNCONFINED COMPRESSION (QU) — Q
VANE TEST (C) AND SENSITIVITY (S) — S
NATURAL MOISTURE AND LIQUIDITY INDEX — LI
LIQUID LIMIT — LL
PLASTIC LIMIT — PL



DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W P 29-61

JOB 60-P-65

DATUM G. S. C.

BOHRING DATE July 27/60.

BORE HOLE NO 3

STATION 14+65 18' Rt.

COMPILED BY B. K.

REMARKS BY K. G. S.

2" DIA SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA CONE

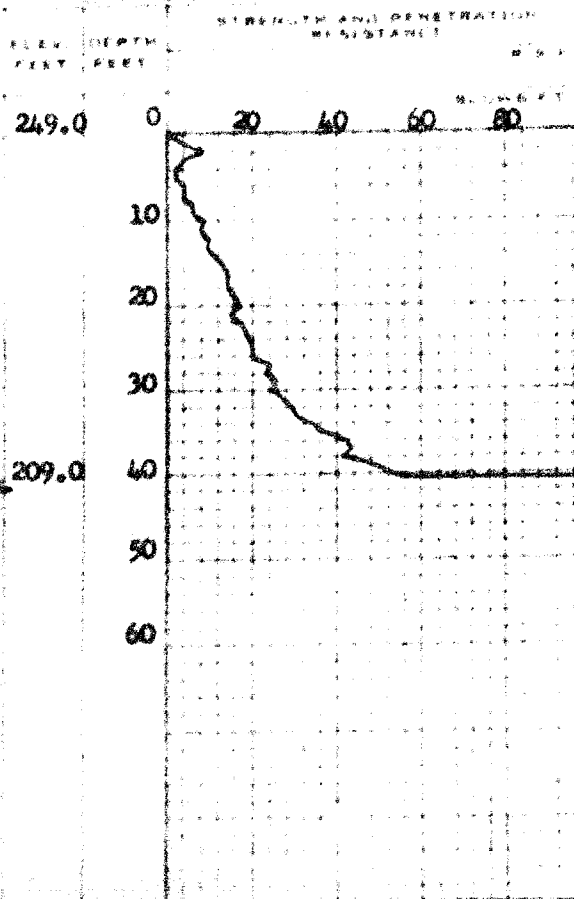
2" SHELBY

CASING

LEGEND

UNCONFINED COMPRESSION (QU) — 0
 VANE TEST (C) AND SENSITIVITY (S) — 4
 NATURAL MOISTURE AND — 11
 LIQUIDITY INDEX — 1
 PLASTICITY INDEX — 1

Ground Level.



Bedrock assumed-End of penetration - Cone Test only.

Water level: 24.7.0

Penetration Resistance Profile shown obtained by driving a 2" dia. cone from ground surface to depth noted. Cone driven with energy equal to 350 ft./lbs. per blow.

DEPARTMENT OF HIGHWAYS - ONTARIO

29-61

BORE HOLE NO 4

JOB 60-F-65

STATION 15/65 18' Lt.

DAFUM G. S. C.

COMPILED BY B. K.

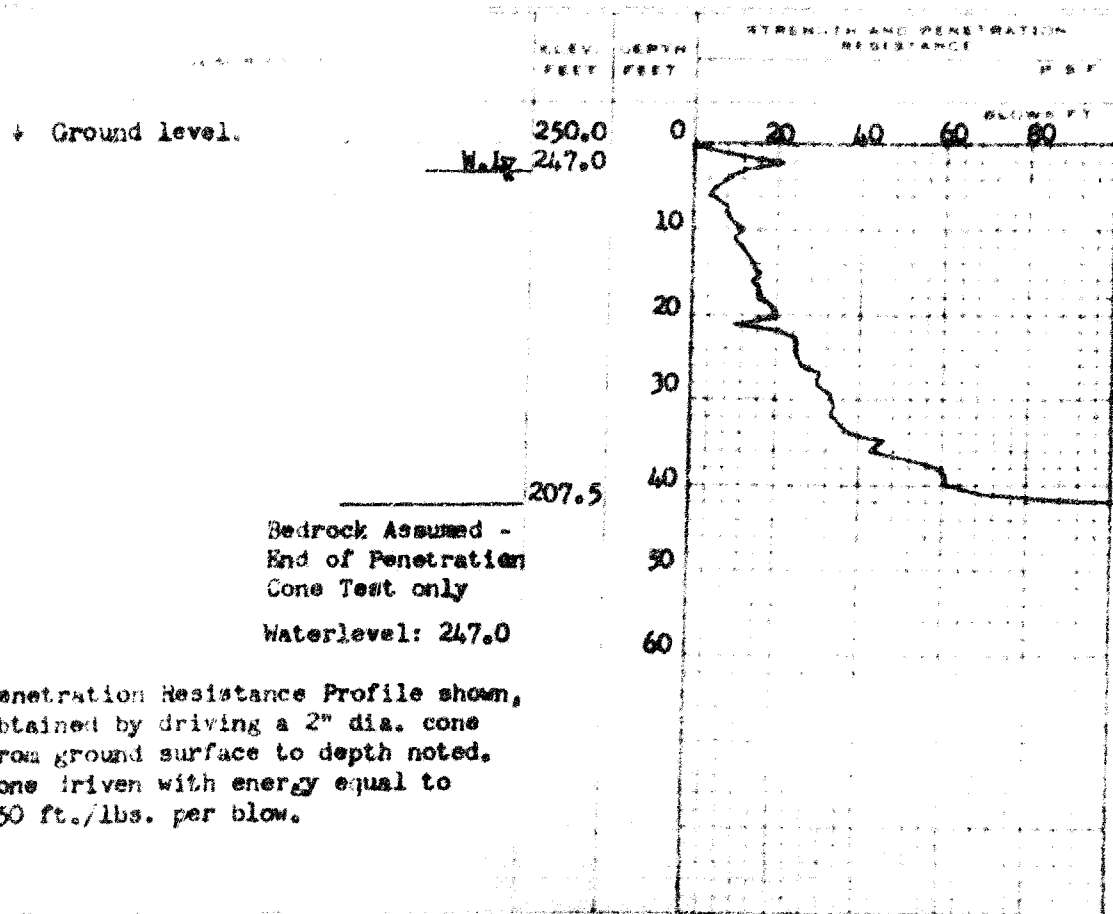
BOHRING DATE July 27/60.

K. C. S.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

UNCONFINE COMPRESSION (QU)	0
WATER TEST AND SENSITIVITY (SI)	+
NATURAL MOISTURE AND	
LIQUIDITY INDEX	1
LIQUID LIMIT	
PLASTIC LIMIT	

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 29-61 BORE HOLE NO. 2

JOB 60-7-65 STATION 13+94.41 R.C.

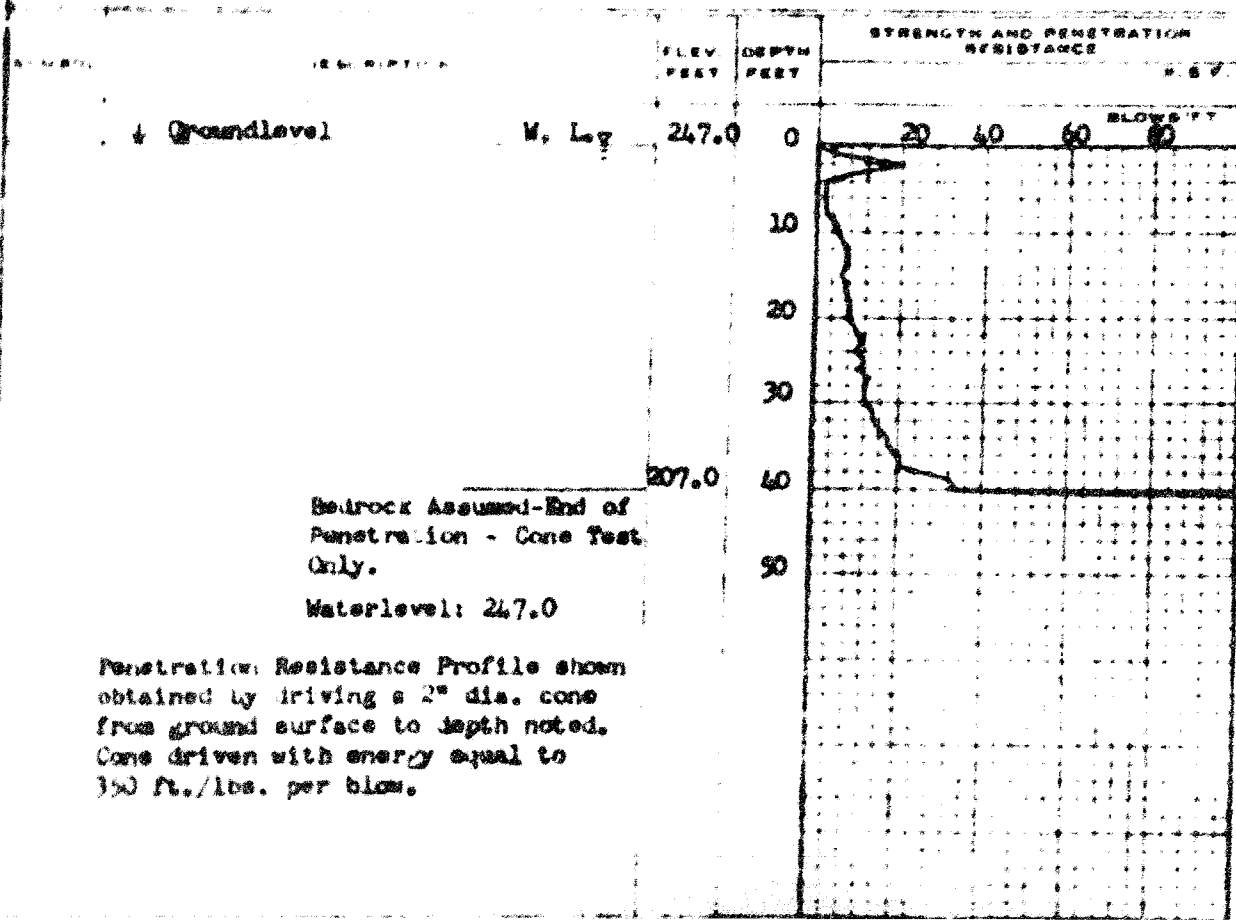
DATUM G. S. C. COMPILED BY B. K.

BORING DATE July 27/60. CHECKED BY K. G. S.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Q_u) O
VANE TEST (C) AND SENSITIVITY (S) +
NATURAL MOISTURE AND LIQUIDITY INDEX LI
LIQUID LIMIT LL
PLASTIC LIMIT PL



CONSISTENCY	SAMPLE	NATURAL
MOIST CONTENT	DRY WT	UNIT WT
		PCF

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 29-61

BORE HOLE NO. 6

JOB 60-F-65

STATION 16+65 E

DATUM G. S. C.

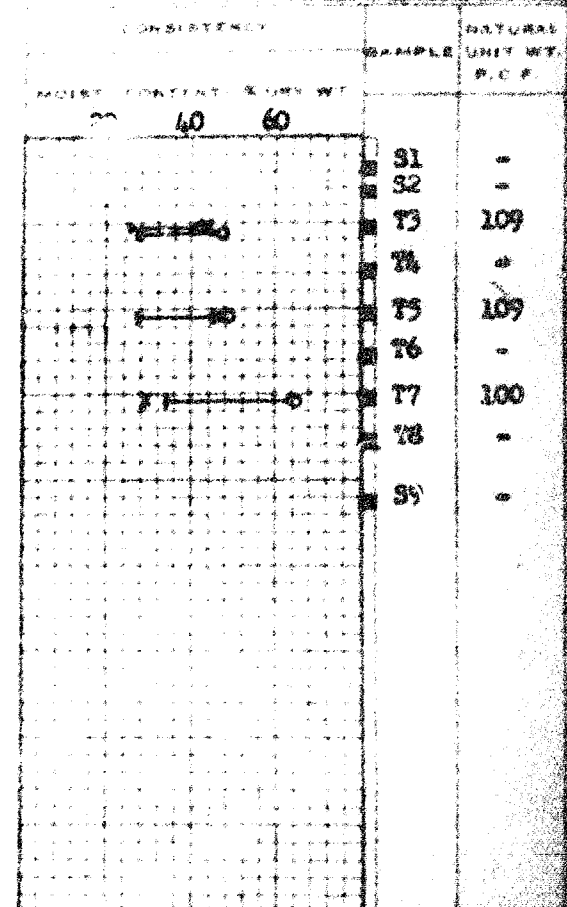
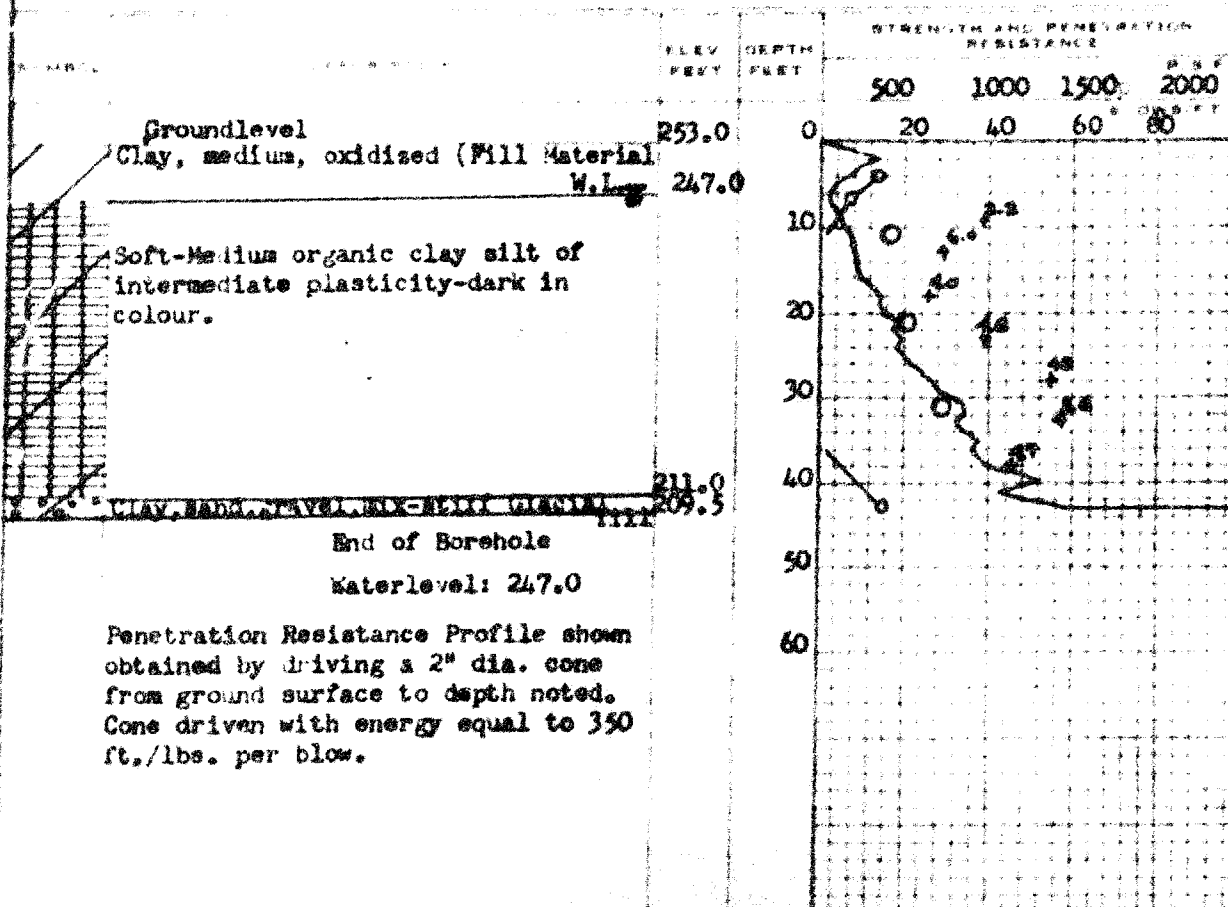
COMPILED BY B. K.

BORING DATE July 28/60. CHECKED BY K. G. S.

2" DIA SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA CONE
 2" SHELBY
 CASING

LEGEND

UNCONFINED COMPRESSION (Q_u) — 0
 VANE TEST (AND SENSITIVITY (S)) — +
 NATURAL MOISTURE AND LIQUIDITY INDEX — 11
 LIQUID LIMIT — 4
 PLASTIC LIMIT — 2



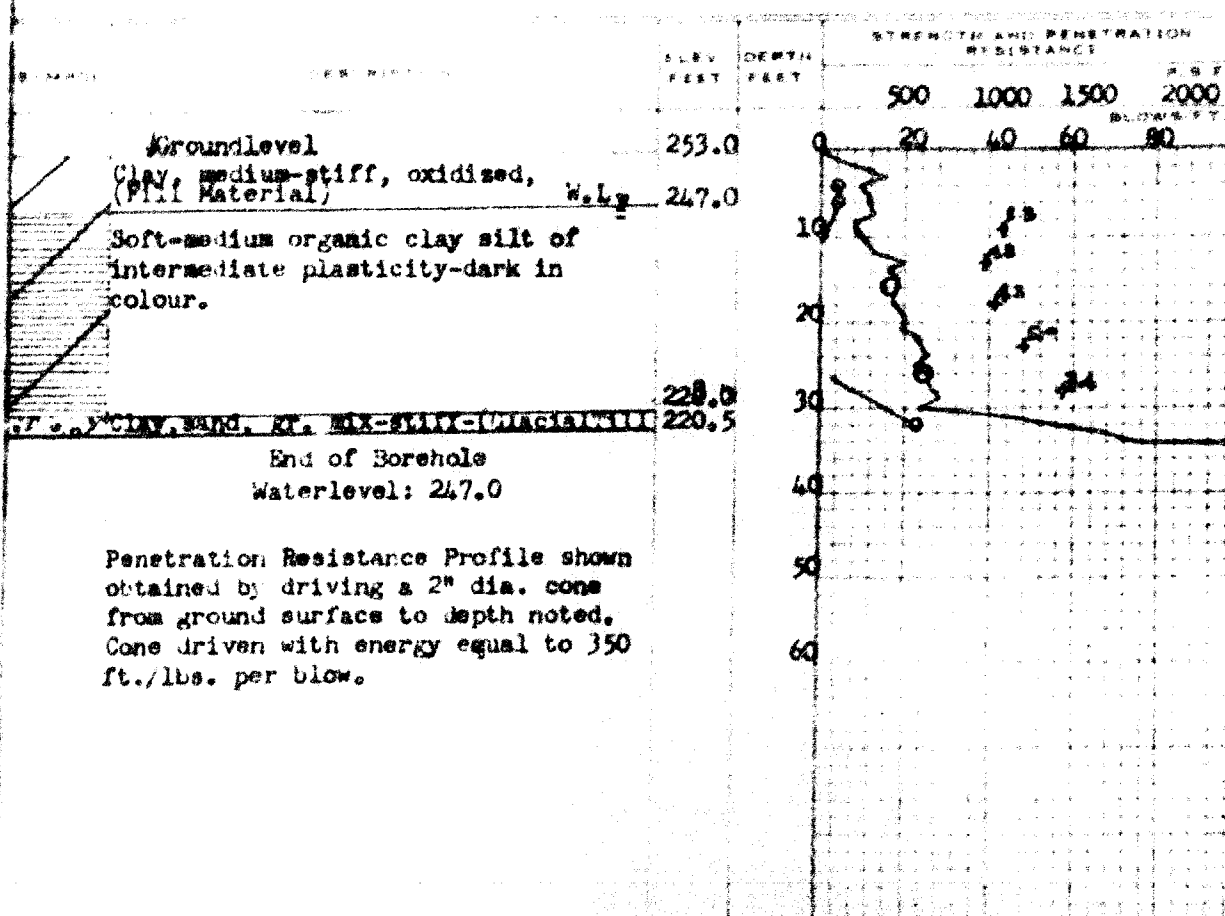
DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 29-61 BORE HOLE NO. 7
JOB 60-P-65 STATION 13+58 5' 14"
DATUM G. S. C. COMPILED BY B. K.
BORING DATE July 28/60. CHECKED BY K. G. S.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Q_u)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND
LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT



DEPTH	UNSATURATED	WATER CONTENT	LIQUIDITY INDEX	NATURAL UNIT WT. P.C.
0				
10				
20				
30				
40				
50				
60				
70				
80				
90				
100				
110				
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810				
820				
830				
840				
850				
860				
870				
880				
890				
900				
910				
920				
930				
940				
950				
960				
970				
980				
990				
1000				

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 27-61

BORE HOLE NO. B

JOB 60-P-65

STATION 16+65 50' Itt.

DATUM G. S. C.

COMPILED BY B. K.

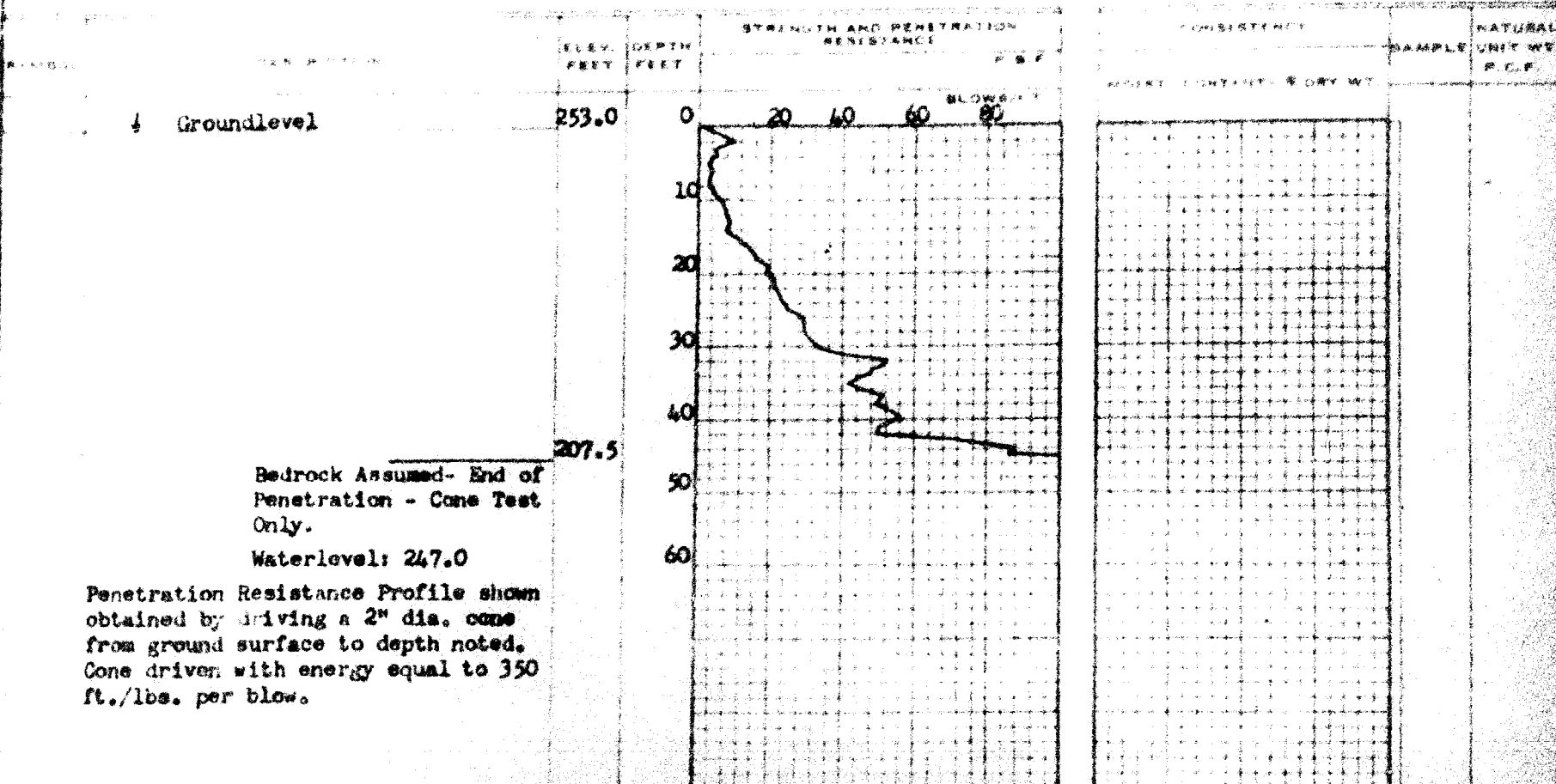
BORING DATE July 28/60.

CHECKED BY K. G. S.

2" DIA SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA CONE
 2" SHELBY
 CASING

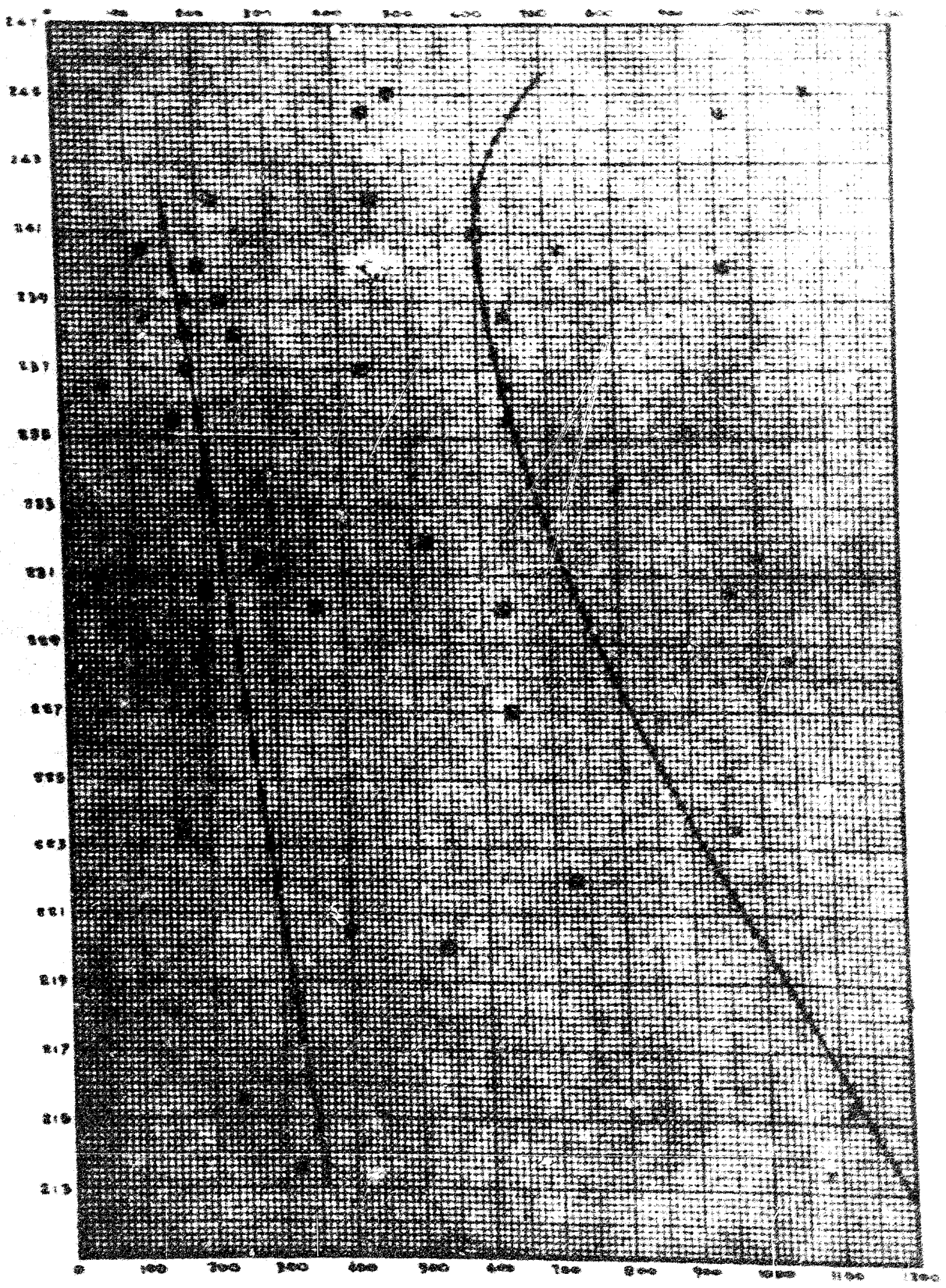
LEGEND

1/2 UNCONFINED COMPRESSION (Q_u)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUIDITY INDEX
 LIQUID LIMIT
 PLASTIC LIMIT



VARIATION OF SHEAR STRENGTH OF ORGANIC CLAY SILT WITH DEPTH

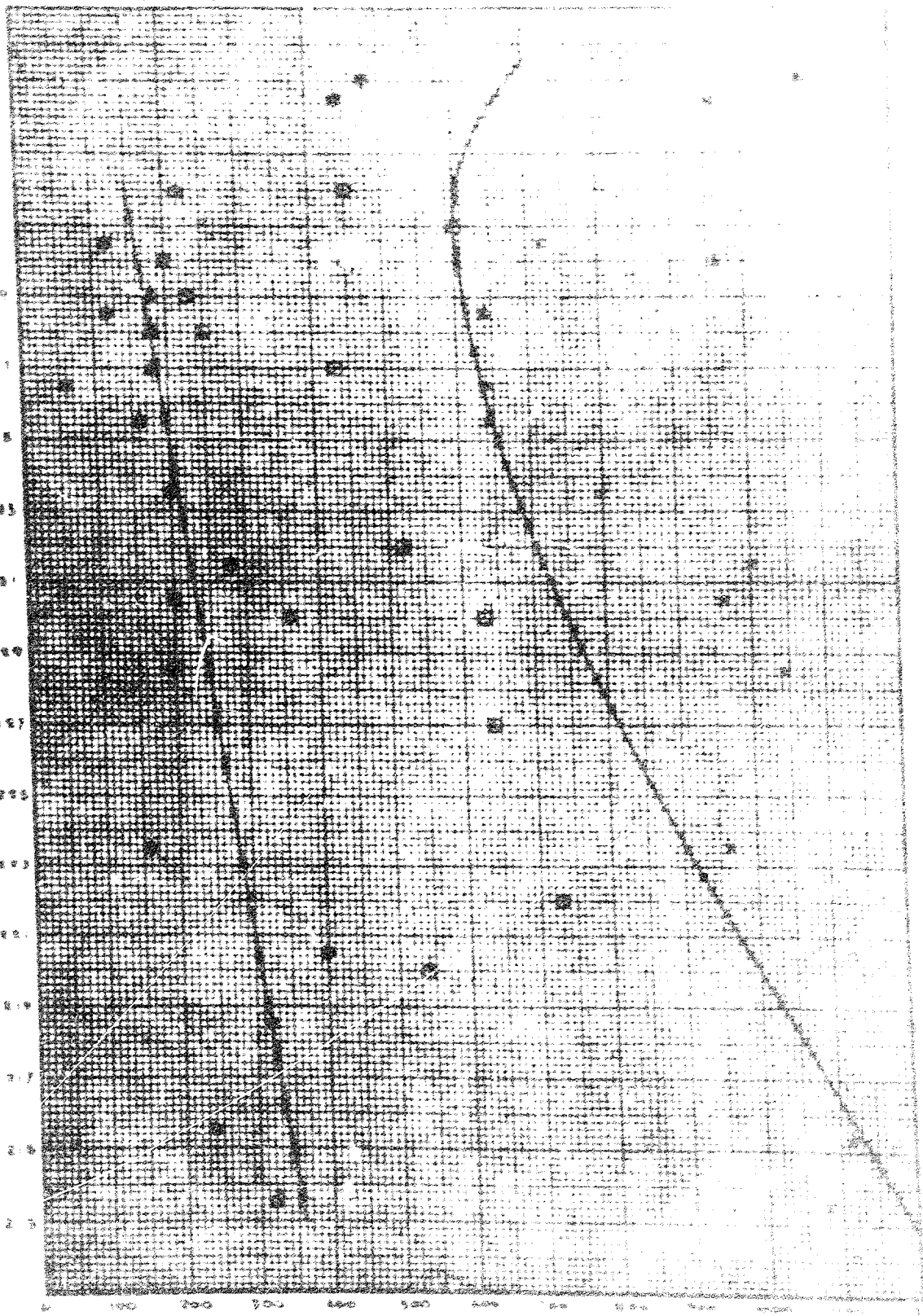
DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



X - IN-SITU VANE ○ - UNDISTURBED TRIAXIAL
● - REMOULDED VANE Δ - REMOULDED TRIAXIAL

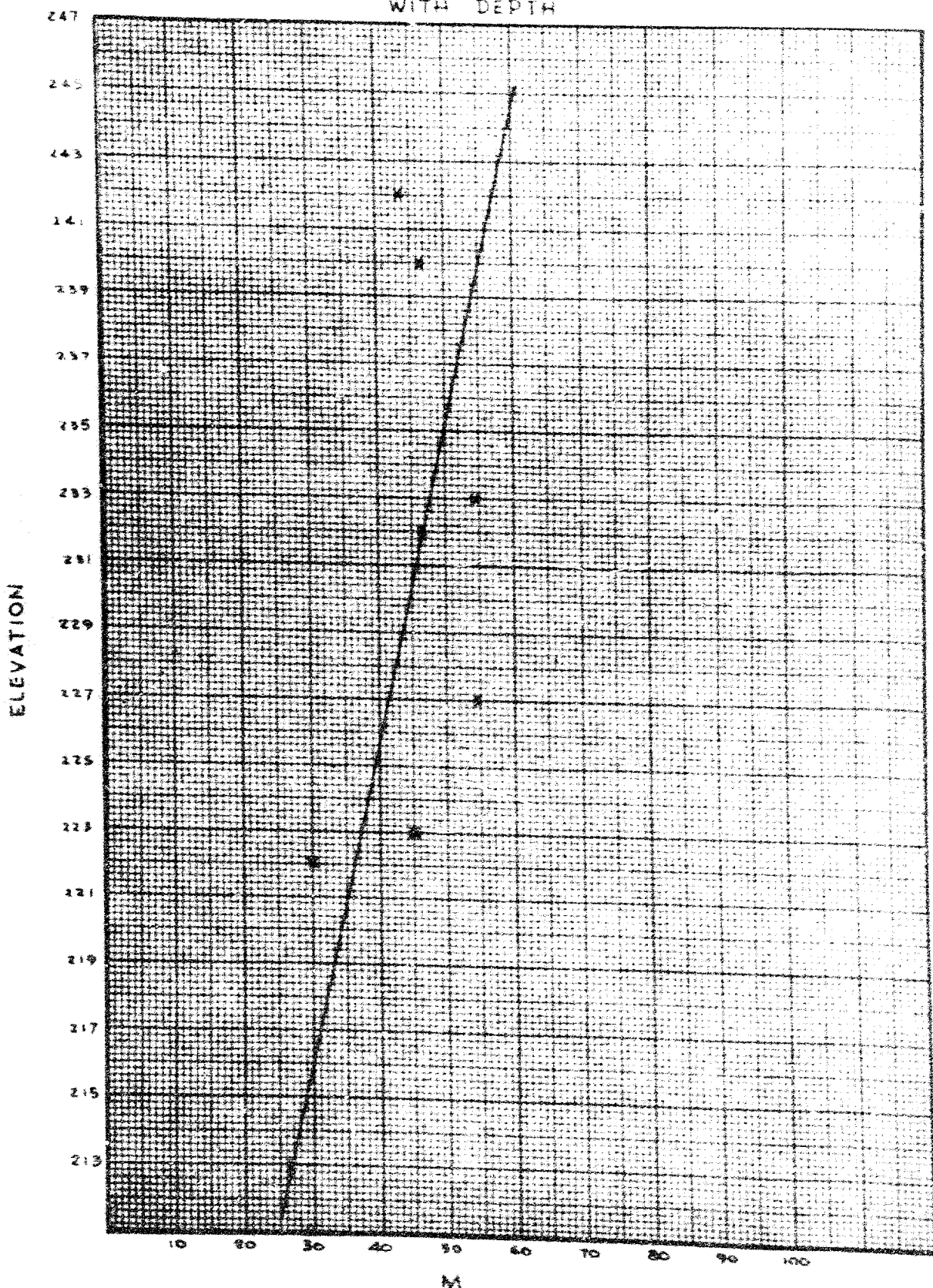
DO NOT WRITE IN THESE SPACES DUE TO
CONFLICT WITH OTHER DOCUMENT

ELEVATION



x - IN SILL VANE
● - SIMULATED VANE

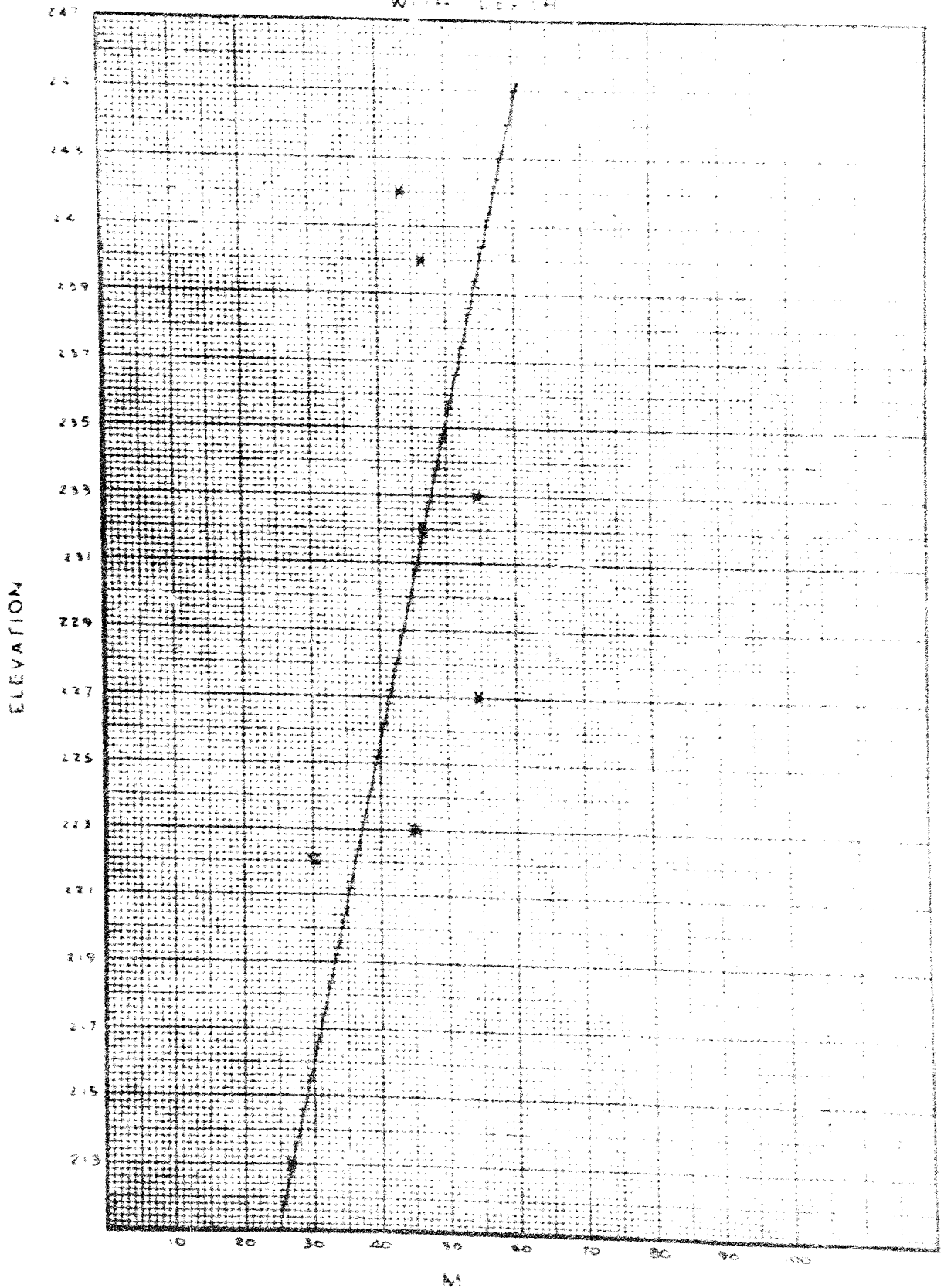
○ - IN SILL VANE
A - REMOVED VANE

VARIATION OF NATURAL MOISTURE CONTENT OF ORGANIC CLAY SILT
WITH DEPTH

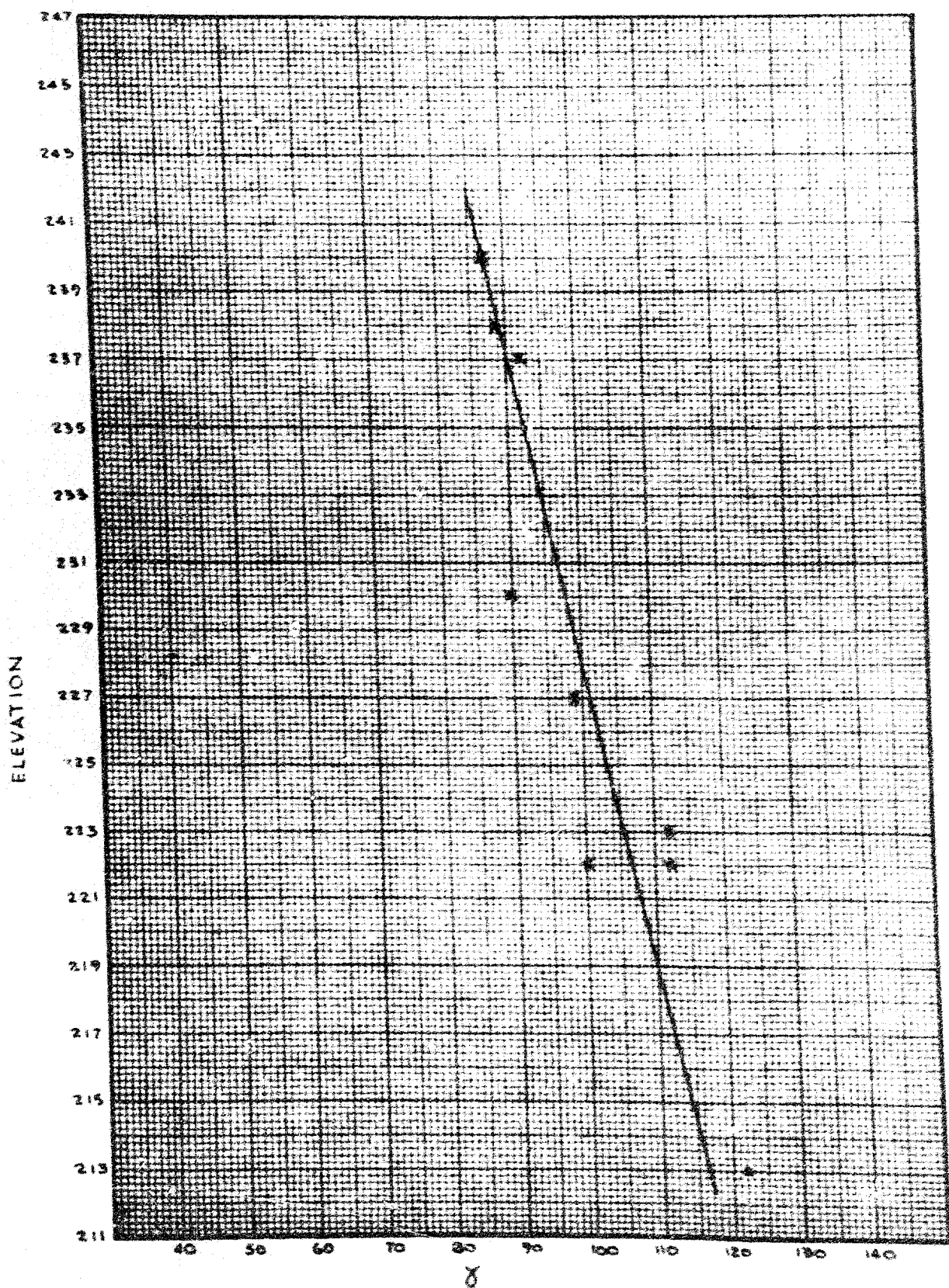
W.D. 29-61

JOB 60-F-65

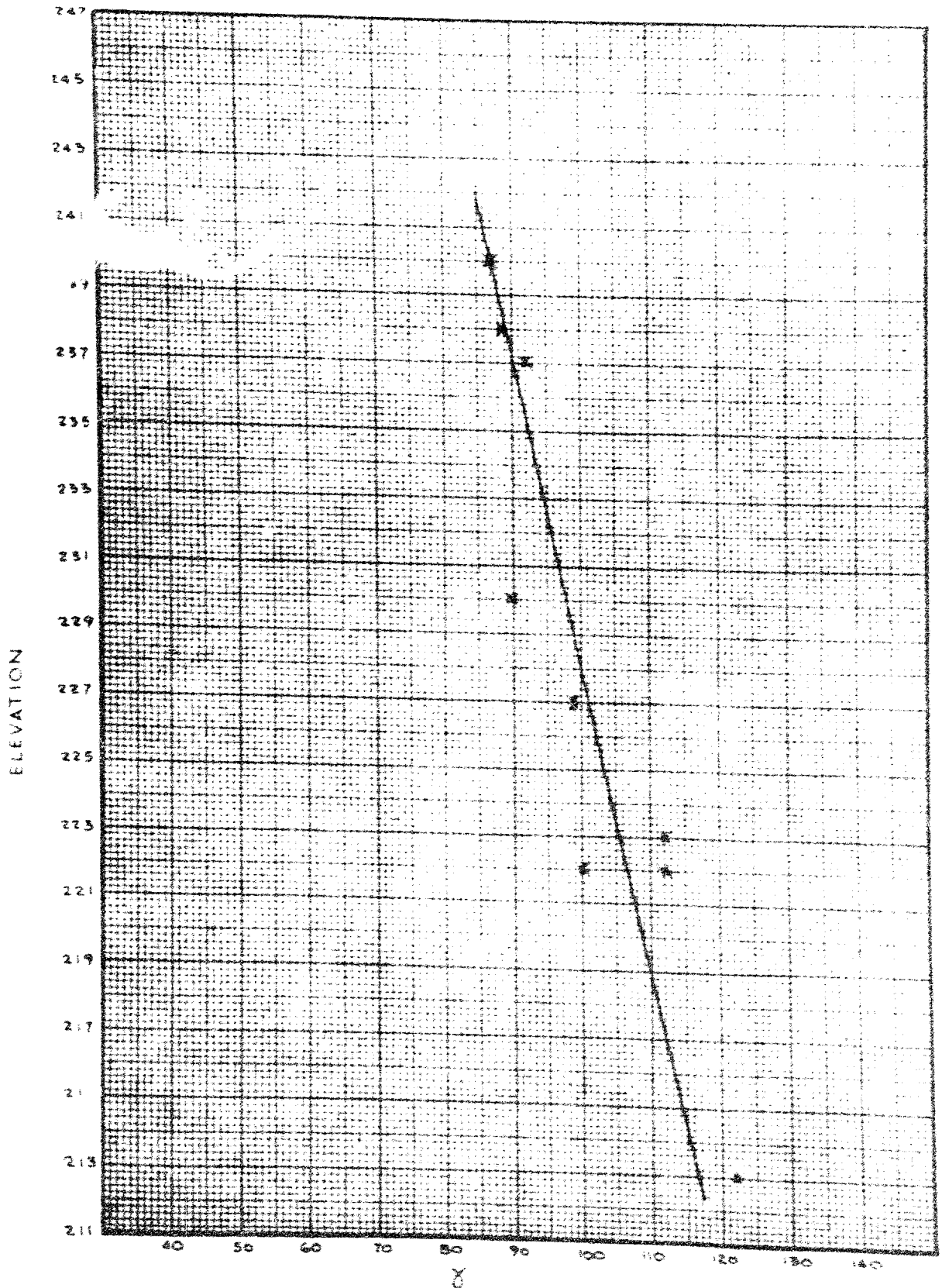
VARIATION OF NATURAL MOISTURE CONTENT OF ORGANIC SLATES WITH DEPTH



VARIATION OF BULK DENSITY OF ORGANIC CLAY SILT WITH DEPTH



VARIATION OF BULK DENSITY OF ORGANIC CLAY SILT WITH DEPTH



In our letter of _____ it was recommended that no excavation in the creek bed be carried out at present. Remedial measures have been suggested and a certain recommendation included.

As an additional safety measure, strutting of the bridge piers was decided upon by the Bridge Office. When the excavation of the creek bed was started a further movement of the piles & of the pier columns was observed. Immediately the excavation was stopped. The further sequence of remedial work was decided by the Bridge Office representatives and at present an excavation ~~of~~ behind the west pier is under way.

It is the purpose of this note to draw the attention to the dangerous and hazardous condition that is created with this excavation. The stability of the approach embankment is in question. This means that if any movements occur ~~to~~ not only will they affect the piers again but also the abutments.

The purpose of the mentioned excavation is not clearly understood. If a pressure release in the piers was intended then ~~the~~ a removal of material should have been undertaken on a larger scale and over a larger area. If it was the intention to enable the ^{moving} ~~positioning~~ of the piles back into the original position, then a much smaller and partial excavation would have been adequate if at all necessary. It is the opinion of the ~~author~~ that this was not necessary. The force required to move the piers ^{back} should be very small.

Since it is definitely decided to

carry out the bracing of the piers the following procedure is suggested:

1. Stopping of any further excavation of the material behind the piers.
2. Filling of ^{the above mentioned} ~~these~~ excavations with ^{preferably} the light weight material.
3. Placing of the piers in their original position by jacking, if necessary.
4. Erection of a temporary strutting arrangement at the level of the pile caps or higher.
5. Excavation of narrow strips, ~~preferably~~ ^{by hand}, for the permanent struts at the pile top levels. These excavations should be carried out alternatively i.e. not ~~only~~ ^{one adjacent} to the other.
6. After placing the strut into position the excavation should be filled immediately with granular material.

June 15, 1961
9.50 A.M.

A. J. Thomas

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN
L. G. SODERMAN

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
767-9201
763-4103

May 5, 1964

Department of Highways, Ontario,
Materials and Research Division,
Parliament Buildings,
Toronto 5, Ontario

Attn: Mr. A. G. Stermac, P. Eng.,
Principal Foundation Engineer

RE: SLOPE INDICATOR READINGS
15-MILE CREEK
ST. CATHARINES, ONTARIO

Dear Sirs:

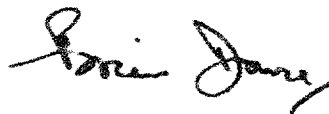
We enclose two final copies of the deflection diagram from Installation No. 1 and 2 at 15-Mile Creek, St. Catharines.

We have plotted the horizontal movement in both the north-south and east-west direction for both installations. It should be noted that in the east-west direction of both installations we have corrected a tabulation error; this results in revised deflection diagram for the June 10, 1964 readings.

We trust that this information is sufficient for your requirements.

Yours faithfully,

H. Q. GOLDER & ASSOCIATES LTD.



B. E. W. Dowse, P. Eng.

BEWD:IMB
Enc.
6358

GENERAL FILE
Mr. J. C. Thatcher,
District Engineer,
Hamilton, Ontario.

Materials & Research Section,
(Foundations Office).

May 12, 1961.

M.A.C. FOUNDATION INVESTIGATION
REPORT.
M.J. 60-5-65 -- M.J. 29-61.

Re: Proposed 15 Mile Creek Bridge,
5 Miles West of St. Catharines, Ont.
District #4.

A foundation investigation was carried out for the above structure by this Section, and the findings, together with the recommendations, were given in the Foundation Report dated August 17, 1960.

When the foundation work was completed and the piers constructed, movements of the piers were observed. According to the information provided by the D.H.O. Construction Supervisor, Alfred Caniletti, the movement of the West pier at the top, amounted to about 3.5", and of the top of the East pier, to about 5.0". The movements must have taken place during the period between May 1st and May 6th, because prior to this period, the constant measurements that were taken did not show any sign of movements.

To establish whether the movements had ceased, or if not, at what rate they were continuing, further measurements were taken twice a day. The results of these measurements are given in the attached Table I.

cont'd. /2 ...

As can be seen from the figures in the table, the total movements during the period May 5th to May 11th - i.e., during 7 days were:-

	<u>E.S. CORNER</u>	<u>CENTER</u>	<u>E.W. CORNER</u>
East Pier	0.94"	0.43"	0.44"
	<u>E.S. CORNER</u>	<u>CENTER</u>	<u>E.W. CORNER</u>
West Pier	0.18"	0.24"	0.15"

It should be kept in mind that the recorded movements represent movements of the pier tops. At the ground level these movements are smaller and are 0.36" maximum and 0.1" minimum. It is hard to consider that as a movement, especially because the presented measurements indicate also some movements in the opposite direction, (May 7th, 9th, and 10th). We believe that the movements have practically ceased and a stabilized condition is forming.

It should be mentioned that the fill is higher on the East side and that there is practically no rip-rap in front of the eastern pier. Even a small amount of soil or rock would provide a certain support or balancing influence, as in the case with the West pier.

Since some more material has to be placed in order to complete the construction, it was decided to undertake some measures which would result in an increase of the stability of the structure. A number of alternatives were studied and preference is given to the following recommendations:-

Part of the present fill material should be excavated, removed and replaced by a light-weight material - air cooled slag. The unit weight of the air cooled compacted slag is 55 lb./cu.ft. compared to the 125 - 130 lb./cu. ft. of the compacted granular material now in place, which means a reduction of 30 - 35% in weight. If the factor of safety at present is considered to be about 1.0, the new factor of safety is certainly going to be greater.

The excavation and replacement of the fill material should be carried out as follows:-

West Approach Fill:

Removal of present fill material down to elevation 252.0', this being 3 ft. below the footing bottom, and 40 ft. to the West measured from the front face of the abutment. The West slope of the excavation should be 1:1, or as convenient for construction purposes. The material in front of the abutment footing - i.e., to the East, should also be removed and replaced. However, the excavation being deeper than the footing bottom, should be started 2 - 3 ft. away from the footing to prevent the removal of the material underneath the footing.

East Approach Fill:

Removal of present fill material down to elevation 253.0', this being 4 ft. below the footing bottom, and 40 ft. to the East measured from the front face of the abutment. All other details should be equal to those of the West approach fill.

cont'd. / ...

The material that is being excavated should be pushed in the northern direction towards the lake. At the west approach, the material should be spread out as a thin layer over the filled-in area, while on the east side, a berm should be built covering the bulge which resulted from a local embankment failure. This should be a second berm and its height above lake level should not exceed 6 ft.

Rip-rapping at the West bank of the creek should be carried out. It is recommended that a thick rip-rap cover be placed because it will provide additional stability for the pier. No excavation of the creek bed should be undertaken at present.

Attention is drawn to the importance of preventing piling-up of material in any one place because overstressing of the ground and failures could result.

It is estimated that some 2,500 tons of the light-weight material will be needed. This material is available and can be purchased from: National Lag, Limited, Hamilton, for a price of \$ 2.50/ton, delivered to the site via tractor-trailer. If tandem trucks are used, the price is \$ 2.75/ton, also delivered to the site.

We believe that the above information and recommendations will be adequate for the future work. However, should there be any additional information or clarification required, this office should be contacted.

AGG/M&EF

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
I. C. Campbell
T. J. Kovich
L. A. Rennie
Foundations Office
Gen. Files.

L. G. Pedersen,
PRINCIPAL FOUNDATION ENGR.
Per:

Lyttelton
(L. G. Starnes,
SUPERVISING FOUNDATION ENGR.)

TABLE 1.

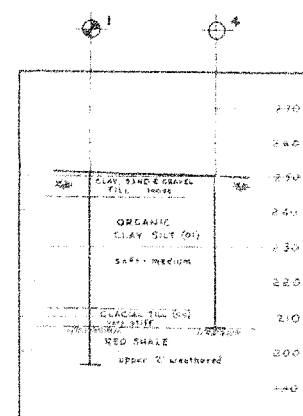
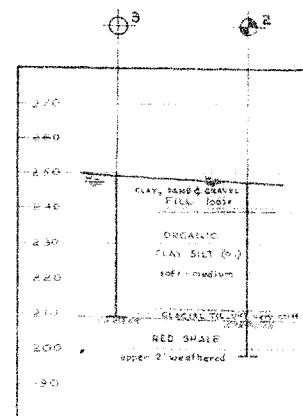
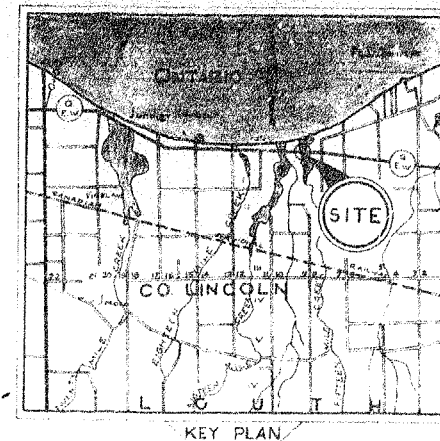
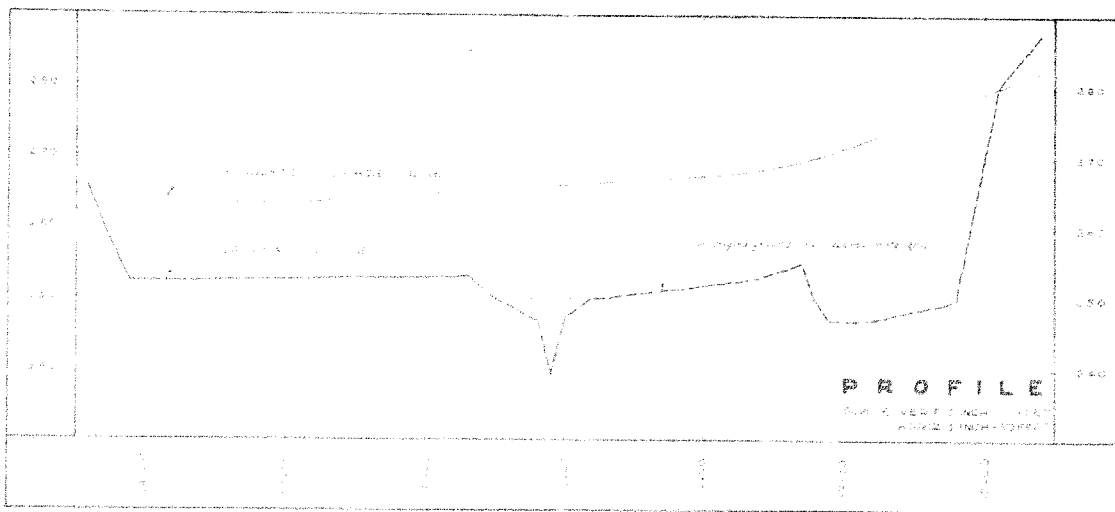
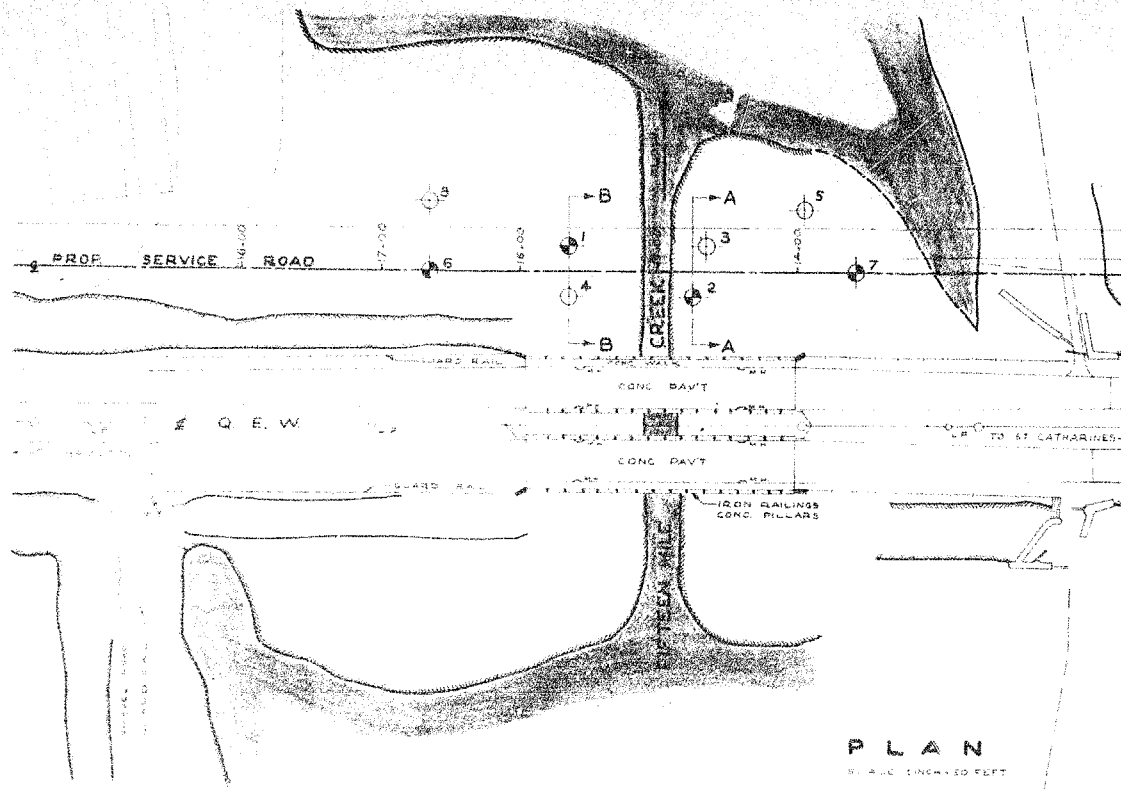
MOVEMENT OF EAST & WEST PIERS
AT FIFTEEN MILE CREEK BRIDGE

	S.E. Corner S.W. Corner (Respectively)	Centre	N.E. Corner N.W. Corner (Respectively)
<u>May 5 - 4:00 P.M.</u>			
East Pier	43.92	43.20	43.85
West Pier	43.50	43.30	43.57
<u>May 6 - 8:30 A.M.</u>			
East Pier	43.93	43.21	43.86
West Pier	43.49	43.30	43.57
<u>May 6 - 2:00 P.M.</u>			
East Pier	43.93	43.21	43.86
West Pier	43.49	43.30	43.57
<u>May 7 - 9:00 A.M.</u>			
East Pier	43.94	43.22	43.87
West Pier	43.49	43.30	43.57
<u>May 7 - 4:30 P.M.</u>			
East Pier	43.93	43.21	43.86
West Pier	43.49	43.30	43.57
<u>May 8 - 8:30 A.M.</u>			
East Pier	43.94 ⁵	43.22 ⁵	43.88
West Pier	43.50 ⁵	43.31	43.58

cont'd. /2 ...

TABLE I - Movement of E. & W. Piers at Fifteen Mile Creek Bridge, (cont'd.)

	<u>S.E. Corner</u> <u>S.W. Corner</u> <u>(Respectively)</u>	<u>Centre</u>	<u>N.E. Corner</u> <u>N.W. Corner</u> <u>(Respectively)</u>
<u>May 8 - 4:30 P.M.</u>			
East Pier	43.94 *	43.22	43.88
West Pier	43.52	43.32	43.59
<u>May 9 - 8:15 A.M.</u>			
East Pier	43.95	43.23	43.88 *
West Pier	43.51 *	43.31 *	43.58 *
<u>May 9 - 4:00 P.M.</u>			
East Pier	43.95	43.23	43.89 *
West Pier	43.52	43.32	43.59
<u>May 10 - 8:15 A.M.</u>			
East Pier	43.95 *	43.23 *	43.89 *
West Pier	43.51	43.31 *	43.58
<u>May 10 - 4:30 P.M.</u>			
East Pier	43.94	43.22 *	43.88 *
West Pier	43.51 *	43.32	43.58 *
<u>May 11 - 8:15 A.M.</u>			
East Pier	43.96 *	43.24	43.90 *
West Pier	43.51 *	43.32	43.58 *



LEGEND			
BORE & PENETRATION			
PENETRATION HOLE			
HOLE ELEVATION	STATION	DISTANCE	PROF. NO.
1	24.00	14.25	14.25
2	24.00	14.25	14.25
3	24.00	14.25	14.25
4	24.00	14.25	14.25
5	24.00	14.25	14.25
6	24.00	14.25	14.25
7	24.00	14.25	14.25
8	24.00	14.25	14.25

DEPARTMENT OF HIGHWAYS, ONTARIO
FIFTEEN MILE CREEK
 PROPOSED NORTH SERVICE ROAD
 DRAWING NO. 60-F-65A
 SCALE: 1 INCH = 10 FEET
 DATE: 1960
 BY: J. P. H. HARRIS
 CHECKED: J. P. H. HARRIS
 APPROVED: J. P. H. HARRIS
 60-F-65A