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W. O. No.

STR. SITE No.

HWY. No. 401

LOCATION Hwy 401 & TILBURY CREEK

No OF PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

DEPARTMENT OF HIGHWAYS OF ONTARIO
Toronto, Ontario

Report on
FOUNDATION CONDITIONS AT THE
TILBURY CREEK CROSSING HIGHWAY 401
(WP 160-58)
December 15, 1959

DEPARTMENT OF HIGHWAYS OF ONTARIO
Toronto, Ontario

40J8-21
GEOCRES No.

REPORT

on

FOUNDATION CONDITIONS AT THE TILBURY CREEK CROSSING HIGHWAY 401 (WP 160-58)

H.G. ACRES & COMPANY LIMITED
Consulting Engineers
Niagara Falls, Canada

December 15, 1959

DEPARTMENT OF HIGHWAYS OF ONTARIO
Toronto, Ontario

REPORT

on

FOUNDATION CONDITIONS AT THE TILBURY CREEK CROSSING ON HIGHWAY 401 (WP 160-58)

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DEPARTMENT OF HIGHWAYS OF ONTARIO
Toronto, Ontario

REPORT

on

FOUNDATION CONDITIONS AT THE TILBURY CREEK CROSSING ON HIGHWAY 401 (WP 160-58)

Introduction

A bridge will be constructed to carry Highway 401 across Tilbury Creek at the location shown on Plate I. Previous soil explorations were made at the site and the results of this work are contained in the following reports:

(a) - Soils Report for Highway 401 Crossing of Tilbury Creek in Tilbury North Township No. 2. February, 1959. By E.M. Peto Associates Ltd.

(b) - Foundation Investigation Highway 401 and Tilbury Creek Crossing. July 30, 1959. By Department of Highways of Ontario.

These explorations were of a preliminary nature and in order to obtain more detailed field and laboratory information, it was decided to carry out additional field work. This investigation was undertaken by H.G. Acres & Company Limited, with the F.E. Johnston Drilling Company Limited being retained to perform the soil drilling and

sampling. The drilling operations were supervised by Mr. G. Wilson of H.G. Acres & Company Limited. Field work commenced on September 8, 1959, and was completed on October 2, 1959. Laboratory testing of the soil samples was done in October and November, 1959. The results of this field and laboratory work are contained in the following report.

Exploratory Work

The exploratory work consisted of drilling a total of six holes, the locations of which are shown on Plate I. Hole No. 1B was used to install a piezometer; holes Nos. 2B and 3B were used to obtain soil samples and to perform vane tests; holes Nos. 4B, 5B and 6B were used to establish the elevation of bedrock, and are extensions of the holes Nos. 3A, 1A and 2A, respectively, which were drilled by the Department of Highways of Ontario. The drilling was performed by the wash boring method and the holes from which samples were obtained were fully cased. Vane tests were performed at least 18 inches below the bottom of the hole at maximum intervals of 10 feet. Soil samples were obtained with the use of a 3-inch diameter fixed piston sampler at maximum intervals of 10 feet. In holes Nos. 2B, 4B, and 6B, attempts were made to core the bedrock but, except in the case of hole No. 2B, very little core recovery was realized because of the soft nature of the rock.

Site Conditions and Soil Properties

The elevation of the general ground surface on either side of Tilbury Creek is approximately 578 feet. The banks of the creek have, however, been raised to form low dykes whose crest elevation is approximately 582 feet.

The materials which were encountered in the exploratory holes are described in the attached drilling logs, Plates II to VII inclusive. In general, the stratigraphy found in this investigation agrees closely with that found in the previous investigations; that is, a clay type till extending from the ground surface to bedrock, which is located generally at a depth of 120 feet.

(a) - Clay Till - The soil which overlies bedrock is primarily composed of what is thought to be a glacial till. This material is uniform in composition and consists of a broad gradation of particle sizes. Its colour varies from brown near the ground surface to grey below a depth of 30 feet. In holes Nos. 3B, 4B, 5B and 6B and in hole No. 1, drilled by E.M. Peto, this material was found to extend to a depth of approximately 120 feet, while in hole No. 2B it extends to bedrock at a depth of 90 feet. Vane tests were performed in, and tube samples were obtained from, holes Nos. 2B and 3B to a depth of 90 feet at which level bedrock was encountered in hole No. 2B, and a silty sand was encountered in hole No. 3B. The results of these field vane

tests and the summary of the results of laboratory classification, compression, and consolidation tests performed on the tube samples, are shown on Plate VIII and Plate IX.

Atterberg Limits - The soil was found to be of uniform composition as evidenced by the fact that the Atterberg limits are relatively constant with depth; the liquid limit ranges from 31.1 per cent to 38.0 per cent, and the plastic limit ranges from 15.4 per cent to 20.1 per cent, the higher values generally being found near the ground surface. These values agree closely with the test results noted in the previous reports.

Consolidation Properties - The results of consolidation tests on nine samples from hole No. 2B, and one sample from hole No. 3B, are shown on Plates X to XIX inclusive. These tests were of the rapid type, load changes being made at 25-minute intervals.

A comparison between this rapid type of consolidation test and the conventional method, where loads are applied at 24-hour intervals, was attempted. The two types of test were performed on specimens from sample No. 8 and the results are shown on Plate XV. These results indicate that greater compression was measured in the rapid test, which is contrary to what would be expected, and for this reason it is suspected that the soils making up the two specimens were different.

Additional comparison tests were not performed as the applicability of the rapid test for inactive, insensitive soils such as those at Tilbury, has been established previously by various researchers.

The results of these tests indicate that the till is preconsolidated to a depth of 30 to 40 feet, below which it is normally consolidated under the existing overburden pressure (Plate VIII). The preconsolidation pressures were determined by the Casagrande graphical method except in the case of the heavily overconsolidated sample No. 2. For this test (PlateX), it was assumed that the void ratio at a pressure of 100 tons per square foot would be 0.30 and that the preconsolidation pressure would be located at the intersection of the line joining this point and the last point on the compression curve, and the estimated natural reconsolidation line. The calculated preconsolidation pressures fall along a smooth curve, except in the cases of the test results for samples Nos. 6 and 13. Sample No. 6 was probably disturbed, as evidenced by the large strain required in the compression test to fail a natural specimen. Sample No. 13 exhibited a preconsolidation pressure of 2.0 tons per square foot, while the effective overburden pressure corresponding to a ground water elevation of 573 feet, is 2.8 tons per square foot (Plate XIX). If this test result is taken to be correct, this

difference between the preconsolidation pressure and the existing overburden pressure implies the existence of pore pressures in excess of those corresponding to a ground water elevation of 573 feet. This condition is quite possible as natural gas under considerable pressure (sufficient to blow water and soil out of hole No. 3B, which was more than 110 feet deep at the time) is known to exist in local areas in the bedrock or in pervious soil overlying the bedrock.

The coefficient of consolidation, c_v, was calculated for the conventional test performed on sample No. 8 and these results are shown on Plate XV. For a stress increase of three tons per square foot above the existing overburden pressure, c_v would be within the range of 6 to 10 x 10^{-4} cm^2/sec.

The compression index, Cc (slope of the estimated geological virgin compression curve), was determined for each consolidation test and the results are shown on Plate XX. It is seen that below elevation 545 feet, the compression index remains relatively constant at a value of 0.28. Above this level the compression index is lower, and this is indicative of a less sensitive soil structure caused, possibly, by weathering.

Shear Strength Properties - The shear strength of natural and remoulded Tilbury clay was determined in the field by means of vane tests, and in the laboratory by means of

unconsolidated undrained compression tests. The results are summarized on Plates VIII and IX. Fairly close agreement was found to exist between the two types of test; for undisturbed soil the undrained compression tests were generally lower by a small amount than the field vane tests, while for remoulded soil the agreement was generally good. The difference in shear strength values for natural soil is, possibly, due to the fact that the soil tested in the laboratory had suffered some structural disturbance during the sampling operations or subsequent handling.

The shear strength varies with depth in a similar manner to the preconsolidation load; that is, near the surface the shear strength is high and generally decreases with depth until it reaches a lower limit at a depth of approximately 40 feet, below which it increases again. It is seen that the natural and remoulded shear strengths measured in holes Nos. 2B and 3B are not in agreement between elevations 560 feet and 530 feet, while below elevation 530 feet their agreement is good.

To check whether the shear strength could be naturally variable, the vane tests performed in holes Nos. 2B and 3B were compared with the vane tests performed by the Department of Highways of Ontario in their July, 1959, investigation. These comparisons are shown on Plate XXI. In the case of

remoulded soil, the shear strength is quite variable but, in general, it varies in a similar manner to the estimated preconsolidation pressures. The comparison of the shear strength of natural soil shows good agreement only below elevation 530 feet, while above this elevation there is a wide scatter of the measured values. This scatter may be due to a natural variation of shear strength but it may also be caused by soil disturbance prior to vane testing; it was noted that in those tests where the Department of Highways investigation showed low natural shear strength, the measured sensitivities were also lower than might be expected and appreciably lower than the sensitivities measured in this investigation. For these reasons, it is concluded that above elevation 530 feet the natural shear strength of the soil might be variable but in general its profile varies in a similar manner to the estimated preconsolidation pressures.

Deformation Properties - Values of the apparent modulus of elasticity of the soil were determined from the results of the compression tests. Since most foundation design is made by using a factor of safety on the ultimate bearing capacity equal to at least 3, the apparent modulus of elasticity is defined as being equal to the slope of the line drawn from origin to the point on the compressive stress-strain curve where one-third the maximum compressive stress is mobilized.

These values are shown on Plate XXII and range from 30 to 240 kips per square foot. It is seen from this drawing that the minimum strain at failure in the compression tests was never less than 7 per cent, while it is known that the strain at failure for carefully taken block samples is generally of the order of 1 to 2 per cent. This indicates sample disturbance. In order to correct for the sample disturbance, it has been assumed that the compressive strengths at failure as measured in the laboratory tests, are approximately correct, while the strain at failure is in all cases equal to one per cent. The resulting values of apparent modulus of elasticity are shown on Plate XXII and for the normally consolidated and lightly overconsolidated soil below elevation 550 feet, an average value of 200 kips per square foot was found, while for the overconsolidated soil near the ground surface the value increases to greater than 700 kips per square foot. These corrected values, although determined by empirical means, are considered to be more typical of the properties of the natural soil than those values determined from the observed stress-strain properties of the sampled soil.

(b) - Silty Sand - In holes Nos. 3B, 5B, and 6B, silty sand was encountered at a depth of approximately 85 feet. It was first noticed in hole No. 3B when it was found impossible

to insert the vane or the piston sampler into the soil. An AX split-spoon sample was obtained and the particle size distribution curve, shown on Plate XXIII, indicates that the material is probably a reworked glacial till. The penetration test required 92 blows to advance the split-spoon 12 inches; it was generally found impossible to remove all the loose, washed, granular particles from the bottom of the hole and, therefore, these penetration results are doubtful as a blanket of this material would certainly influence the test results. Unfortunately, drilling equipment capable of penetrating this sandy material was not available at the site and, to avoid prolonged delays, it was decided to continue the drilling by jetting the wash rods to bedrock without the use of casing. The jetting indicated the presence of alternating bands of stiff and soft materials which were interpreted as being layers of clay till and silty sand. These materials appeared to be impermeable as there were no losses or gains of drilling water and a natural gas pocket existed under considerable pressure at the bedrock contact.

Wash borings in holes Nos. 5B and 6B appeared to indicate the presence of similar silty sand bands at the same depth as they were encountered in hole No. 3B, but no such seams were noted in holes Nos. 2B, 4B, or 1. It is concluded, therefore, that these deposits of silty sand represent isolated

bands of limited extent which may be expected to be found in such a material as glacial till.

(c) - Bedrock - The information concerning the geological characteristics of the bedrock in the Tilbury area has been obtained from Geological Memoir 278[1], from well records of the Ontario Department of Mines[2], from discussions with geologists of the Union Gas Company, and from an examination of bedrock exposures.

The rock formations, outcropping below the till, include the Hamilton and Dundee formations of the Devonian system. The Hamilton sequence, as shown on Plate I, includes a series of very soft calcareous shales called "soapstones" with interbedded limestones and black shales, the total thickness being in excess of 100 feet. The "soapstones" also contain nodules and lenticles of limestone. The Dundee formation is located below the Hamilton formation and it consists of hard, massive limestones and represents the only type of bedrock in the area which is structurally sound. The dip of these rock formations in the vicinity of Tilbury Creek is towards the east at an angle between 10 and 20 degrees.

The suboutcrop of the contact between the Hamilton and the underlying Dundee formation has been estimated to occur to the west of Tilbury Creek; the exact location of

(1) - Palaeozoic Geology of the Windsor-Sarnia area, Ontario. Geological Survey of Canada, Memoir 278.
(2) - Ground Water in Ontario, 1948, 1949, and 1950. Ontario Department of Mines, Bulletin 145.

this contact is not known and the geological succession below Tilbury Creek as shown on Plate I is conjectural. However, the rock drilling at the site proved the existence of the Hamilton formation and the Dundee limestones would be expected only at greater depths.

In order to obtain a clearer idea of the nature of the till/soapstone contact, a visit was made to an exposure in Rock Glen, Arkona, Ontario. This exposure shows that the soapstone is interbedded with nodules, lenticles, and thin bands of limestone, without continuity over any great distance. Large glacial erractic boulders are also embedded at the till/soapstone contact. A sample of the exposed soapstone which most closely resembled the fragments recovered from diamond drill cores, was obtained and tested. It may be described as an overconsolidated clay. The results of two unconsolidated undrained compression tests are shown on Plate XXIV and the resulting average shear strength was 2,490 pounds per square foot. The liquid and plastic limits were found to be 37.2 per cent and 19.2 per cent, respectively, which correspond very closely to the properties of the clay till. The consolidation characteristics of the material are shown on Plate XXV. It is seen that the rebound curve is positioned close to the compression curve, and this is considered evidence that this sample of shale has been consolidated

during geological time under very large pressures, but has absorbed water in its present exposed state subsequent to the removal of overlying material. Depending upon the nature and degree of any property changes which might have taken place in the "soapstones" which now underlie the clay till, the properties of the sample of shale outcrop which was tested may, or may not, represent those of the unexposed rock.

Conclusions

(a) - On the basis of the drilling work done at the site, the general soil profile consists of clay till overlying bedrock. In four holes, bedrock was located at a depth of approximately 120 feet, while in one other hole it was located at a depth of approximately 90 feet. The ground water level was found to correspond approximately with the water level in Tilbury Creek.

(b) - The results of field and laboratory tests on the clay till indicate that it exists in a preconsolidated state to a depth of 30 to 40 feet, and in a normally-consolidated state below this level. The average undrained shear strength of the surface 30 feet is in excess of two kips per square foot, while below this level the average value is one kip per square foot.

(c) - The clay till below the 30-foot level is a relatively compressible soil and has a compression index of 0.28. The results of preliminary settlement calculations for a strip loading of 1.3 kips per square foot which is 140 feet wide have been summarized below.

Thickness of clay till	90 ft	120 ft
Elastic settlement	0.4 ft	0.5 ft
Consolidation settlement ..	1.2 ft	1.4 ft
Total settlement	1.6 ft	1.9 ft

These results indicate that large total settlements can be expected for wide strip loadings in excess of one kip per square foot. For the conditions under which drainage takes place only in a vertical direction, and the bedrock acts as an impervious boundary, and the average coefficient of consolidation is 6×10^{-4} square centimeters per second, the time required for the average degree of consolidation to become 50 per cent was calculated to be 100 years. The actual rate of settlement is largely dependent upon the drainage conditions of the clay till, and the existence of several pervious strata may easily increase the rate at which consolidation takes place by a factor greater than ten.

(d) - The bedrock underlying the clay till is the Hamilton formation which consists of soft shales interbedded with limestones and black shales. The total thickness of

this formation in the Tilbury Creek area is greater than 100 feet. As shown on Plate I, the surface of the bedrock has irregularities which are possible as a result of differential erosion caused by the heterogeneous nature of the rock. The properties of this rock are unknown as it was found impossible to recover core samples, and because outcrop samples of this rock have probably been changed physically and chemically by natural processes. However, in general, it is weak, and to develop the capacity of bearing piles, large penetrations may be required. In certain areas the piles may come to rest on limestone deposits but this is not likely to be a general condition.

The actual bearing capacities of piles driven into this bedrock formation can only be determined on the basis of pile loading tests.

SUMMARY OF FIELD VANE TEST RESULTS - HOLE NO. 2B

Elevation Feet	Undrained Shear Strength, Psf		Sensitivity
	Natural	Remoulded	
566	2,950	1,480	2.0
564	2,460	1,105	2.2
561	1,940	1,130	1.7
555	1,550	872	1.8
549	1,240	726	1.7
544	807	355	2.3
539	727	258	2.8
534	807	339	2.4
529	888	388	2.3
524	903	388	2.3
517	1,050	436	2.4
509	968	565	1.7
504	1,210	549	2.2
499	1,195	565	2.1
494	968	420	2.3

SUMMARY OF FIELD VANE TEST RESULTS - HOLE NO. 3B

Elevation Feet	Undrained Shear Strength, Psf		Sensitivity
	Natural	Remoulded	
571	7,000+	-	-
567	2,460+	-	-
562	860	541	1.8
557	984	541	1.8
552	1,105	664	1.6
547	1,350	861	1.6
542	1,230	737	1.7
537	1,180	639	1.9
532	984	443	2.2
527	1,130	615	1.8
522	862	492	1.8
517	935	492	1.9
512	1,033	517	2.0
507	984	492	2.0
502	984	492	2.0
497	1,230	640	1.9

SUMMARY OF LABORATORY TEST RESULTS

Hole No.	Sample No.	Elevation Feet	Water Content %	Liquid Limit %	Plastic Limit %	Su_n Psf	Failure Strain %	Su_r Psf	St
2B	2	569	20.6	38.0	18.3	3,780	20	2,440	1.6
	3	563	21.3	32.6	15.7	1,862 1,715	16 20	1,192	1.6
	4	559	22.8	31.1	16.0	1,500	18	814	1.9
	5	553	24.1	33.5	16.5	1,018	20	739	1.4
	6	551	23.9	33.2	15.6	-	-	-	-
	8	543	27.9	33.0	16.8	704	7	339	2.1
	9	533	27.2	34.0	20.1	909	7	358	2.5
	11	515	26.9	34.1	17.9	1,080	12	409	2.6
	12	507	27.0	-	-	868	12	345	2.5
	13	498	28.2	36.5	17.0	923	12	405	2.3
3B	14	570	21.4	36.1	18.4	3,810	11	2,540	1.5
	15	560	25.3	33.0	15.4	750	9	435	1.7
	16	550	23.3	32.8	16.4	1,100	12	682	1.6
	17	540	25.3	33.8	16.4	1,177	8	504	2.3

Su_n - Natural undrained shear strength
Su_r - Remoulded undrained shear strength
St - Sensitivity

LIST OF PLATES

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Plate IV - Drilling Report, Hole No. 3B

Plate V - Drilling Report, Hole No. 4B

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Plate VII - Drilling Report, Hole No. 6B

Plate VIII - Summary of Drilling and Test Results, Hole No. 2B

Plate IX - Summary of Drilling and Test Results, Hole No. 3B

Plate X - Consolidation Test; Hole No. 2B, Sample Elevation 569 Feet

Plate XI - Consolidation Test; Hole No. 2B, Sample Elevation 563 Feet

Plate XII - Consolidation Test; Hole No. 2B, Sample Elevation 559 Feet

Plate XIII - Consolidation Test; Hole No. 2B, Sample Elevation 551 Feet

Plate XIV - Consolidation Test; Hole No. 3B, Sample Elevation 550 Feet

Plate XV - Consolidation Test; Hole No. 2B, Sample Elevation 543 Feet

Plate XVI - Consolidation Test; Hole No. 2B, Sample Elevation 533 Feet

Plate XVII - Consolidation Test; Hole No. 2B, Sample Elevation 515 Feet

Plate XVIII - Consolidation Test; Hole No. 2B, Sample Elevation 507 Feet

Plate XIX - Consolidation Test; Hole No. 2B, Sample Elevation 498 Feet

Plate XX - Variation of Compression Index with Depth

Plate XXI - Comparison of Results of Field Vane Tests

Plate XXII - Variation of Apparent Modulus of Elasticity with Depth

Plate XXIII - Grain Size Distribution; Hole No. 3B, Sample Elevation 494 Feet

Plate XXIV - Results of Compression Tests on Sample of Shale Outcrop

Plate XXV - Consolidation Test; Bedrock Outcrop from Rock Glen, Arkona, Ontario.

OVERSIZE DRAWING

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 1-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd STARTED 11:30 a.M. September 9 1959
FINISHED 5:00 p.M. September 11 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. 4 inch
ROCK: -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 48+65
DEPARTURE 160 Ft Right
BEARING -
INITIAL DIP 90 degrees
OTHER DIPS -

ELEVATIONS: DATUM C.S.C.
DRILL PLATFORM -
GROUND SURFACE 579.0
ROCK SURFACE -
BOTTOM OF HOLE 544.0
WATER TABLE 571.3

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
	Topsoil							
2.0								
	Clay till	Grey						
35.0								
		Note: Carborundum piezometer tip installed 26 feet below ground surface						

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR G. Wilson
LOGGED BY W. Wilson
APPROVED D. H. MacDonald.
DATE December 11, 1959

Plate II A

FORM NO. 91-A

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Highways of Ontario — JOB No. 837

PROJECT Hwy 401 and Tilbury Creek Crossing — HOLE No. 1-B

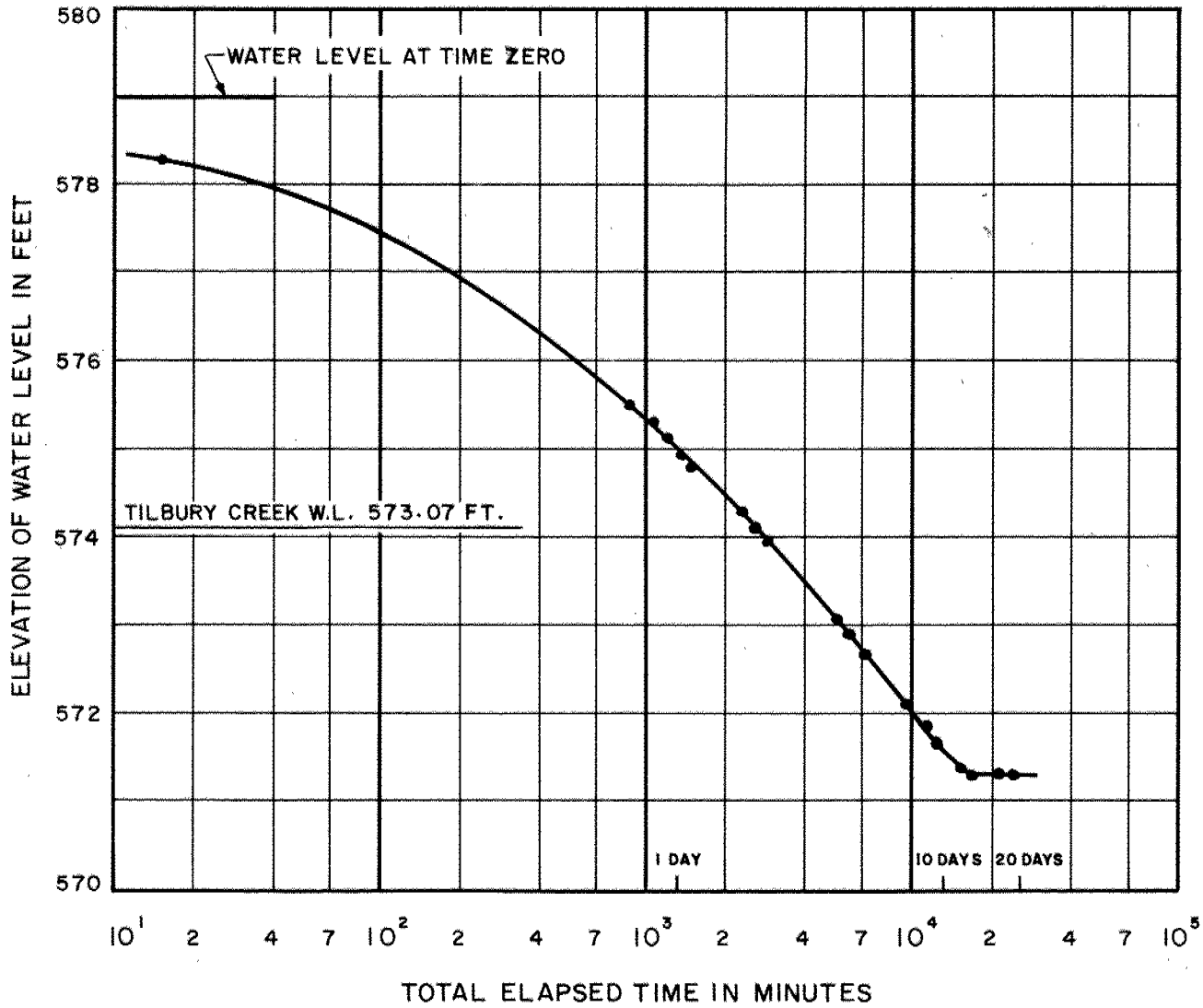
SITE Tilbury Creek, Ontario — SHEET No. 2 OF 2

Date	Time	Total Elapsed Time in Minutes	Elevation of Water Level in Piezometer Tube
September 14, 1959	1730	0	578.97
September 14, 1959	1745	15	578.26
September 15, 1959	800	870	575.51
September 15, 1959	1100	1,050	575.30
September 15, 1959	1315	1,185	575.09
September 15, 1959	1535	1,325	574.91
September 15, 1959	1728	1,438	574.78
September 16, 1959	745	2,295	574.28
September 16, 1959	1200	2,550	574.12
September 16, 1959	1730	2,880	573.97
September 18, 1959	815	5,205	573.09
September 18, 1959	1730	5,760	572.89
September 19, 1959	730	6,600	572.68
September 21, 1959	800	9,510	572.15
September 22, 1959	1200	11,190	571.84
September 23, 1959	800	12,390	571.66
September 25, 1959	730	15,270	571.39
September 26, 1959	700	16,650	571.30
September 29, 1959	800	21,030	571.34
September 30, 1959	800	22,470	571.32
October 1, 1959	1200	24,150	571.34

NOTE: Ground surface elevation 578.97

Plate IIB

FORM NO. 91-E



NOTE:

MEASUREMENTS WERE STARTED ON SEPT. 14, 1959 AT 1730 HOURS.

WATER LEVEL AT TIME ZERO WAS EL. 578.97 FT.

H. G. ACRES & COMPANY, LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY. 401 AND TILBURY CREEK CROSSING	
ELEVATIONS OF WATER LEVEL IN PIEZOMETER TUBE, HOLE 1B	
APPROVED	DATE: DEC. 11, 1959
D. H. MacDonald H.G. ACRES & COMPANY LTD.	SCALE / JOB No. 837
	PLATE - II C

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 2-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 5:00 p.m. September 11 19 59; FINISHED 2:30 p.m. September 19 19 59

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. 4 inch; ROCK: Diamond drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 48+ 20; DEPARTURE 55.0 Ft Right; BEARING -; INITIAL DIP 90 degrees; OTHER DIPS -

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM -; GROUND SURFACE 579.4; ROCK SURFACE 490.6; BOTTOM OF HOLE 485.6; WATER TABLE 573

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION ~~TEST~~
0					Inches	Feet	Feet	
	Topsoil	Dark grey to black, very dry with shrinkage cracks, contains well rounded pebbles and occasional boulders	1	CO	3	5.0	1.0	Tapped
				Vane	Test	7.0		Tapped
				Vane	Test	8.0		Tapped
2.0			2	CO	3	10.0	1.5	Tapped
	Clay till	Brown with grey streaks, weathered, fissured, but not stratified, contains angular pebbles		Vane	Test	13.5		Tapped
				Vane	Test	15.0		Pushed
14.0			3	CO	3	16.0	1.5	Pushed
	Clay till	Grey with brown streaks, stiff, weathered, not stratified, contains angular pebbles, water content increasing with depth		Vane	Test	18.5		Pushed
			4	CO	3	20.0	1.5	Pushed
				Vane	Test	23.5		Pushed
20.0				Vane	Test	24.5		Pushed
	Clay till	Grey with brown streaks, firm, slightly weathered, not stratified, contains angular pebbles	5	CO	3	25.0	1.5	Tapped
			6	CO	3	28.0	1.5	Pushed
28.0								
	Clay till	Grey, unweathered, firm but remoulds to soft clay, contains pebbles as above		Vane	Test	30.0		Pushed
			7	CO	3	31.5	1.5	Pushed

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N - INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED: D. H. MacDonald

DATE: December 11, 1959

Plate IIIA

FORM NO. 91-A

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Highways of Ontario — JOB No. 837

PROJECT Hwy 401 and Tilbury Creek Crossing — HOLE No. 2-B

SITE Tilbury Creek, Ontario — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE	SIZE	DEPTH	RET'D	PENETRATION
					Inches	Feet	Feet	
35.0								
	Clay till	Grey, slightly bluish in patches, soft, contains pebbles as above	8	CO	3	36.5	1.5	Pushed
				Vane	Test	40.0		Pushed
				Vane	Test	45.0		Pushed
50.0								
	Clay till	Grey, firm but remoulds to soft clay with pebbles as above	9	CO	3	46.5	1.5	Pushed
				Vane	Test	50.0		Pushed
89.0				Vane	Test	55.0		Pushed
	Bedrock	Grey hard fossiliferous limestone with thin hard brown shale bands, nodules of pyrite Dip of bedding varies from horizontal to 20 degrees. Rock is not weathered.	10	CO	3	56.5	1.5	Pushed
				Vane	Test	60.0		Pushed
				Vane	Test	62.5		Pushed
			11	CO	3	64.0	1.5	Pushed
94.0								
	Note:	No noticeable water loss or gain during drilling for full depth of hole. Hole quickly filled with water after casing was withdrawn. This might possibly be due to water flowing in from fissured zone near surface.		Vane	Test	67.5		Pushed
				Vane	Test	70.0		Pushed
			12	CO	3	71.5	1.5	Pushed
				Vane	Test	75.0		Pushed
				Vane	Test	80.0		Pushed
			13	BO	3	81.6	1.5	Pushed
				Vane	Test	85.0		Pushed
				Vane	Test	89.0		Pushed

Plate IIIB

FORM NO 91 B

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 3-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 7:00 a.m. September 21 1959; FINISHED 5:30 p.m. September 25 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. 4 inch; ROCK: -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 47 + 17; DEPARTURE 8.0 Feet Left; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM --; GROUND SURFACE 576.9; ROCK SURFACE 458.9; BOTTOM OF HOLE 458.9; WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION ~~TEST~~
					Inches	Feet	Feet	
0								
	Topsoil	Dark grey, very stiff, fissured		Vane	Test	5.8		Pushed
2.0								9" only
	Clay till	Brown, very stiff, weathered, fissured, not stratified, contains angular pebbles	14	BO	3	6.5	1.5	Pushed
				Vane	Test	10.0		Pushed
				Vane	Test	15.0		Pushed
13.0								
	Clay till	Grey with some brown streaks, firm, not stratified	15	CO	3	16.5	1.5	Pushed
				Vane	Test	20.0		Pushed
35.0				Vane	Test	25.0		Pushed
	Clay till	Grey, firm to soft, not stratified, contains angular pebbles	16	CO	3	26.5	1.5	Pushed
83.0				Vane	Test	30.0		Pushed
	Stratified clay till and silty sand.	Alternating bands of compact silty sand and soft clay till.		Vane	Test	35.0		Pushed
			17	CO	3	36.6	1.5	Pushed
				Vane	Test	40.0		Pushed
				Vane	Test	45.0		Pushed
			18	CO	3	46.5	1.5	Pushed
116.0								
	Rock			Vane	Test	50.0		Pushed
				Vane	Test	50.0		Pushed

SAMPLING METHOD
*A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson — APPROVED: D. H. MacDonald.

LOGGED BY: G. Wilson — DATE: December 11, 1959

Plate IVA

FORM NO. 91-A

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 3-B

SITE: Tilbury Creek, Ontario — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE	SIZE Inches	DEPTH Feet	RET'D Feet	PENETRATION
	Note:	At depth of 118 feet, struck very hard formation which was either bedrock or boulder lying on bedrock	19	CO	3	56.5	1.5	Pushed
				Vane	Test	60.0		Pushed
				Vane	Test	65.0		Pushed
		Very strong natural gas supply blew all water from the casing. Pressure dropped gradually with time but further drilling was made impossible	20	CO	3	66.5	1.5	Pushed
				Vane	Test	70.0		Pushed
				Vane	Test	75.0		Pushed
		No water was lost or gained	21	BO	3	76.5	1.5	Pushed
		Hole was sealed with cement and soil		Vane	Test	83.0		Tapped but no penetration
			22	AR	2	83.0 to 84.5	0.5	92 blows 140-lb hammer falling 30"

Plate IVB

FORM NO. 91 B

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 4-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 2:00 p.M. September 30 1959; FINISHED 3:00 p.M. October 1 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. AX; ROCK: Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 49+ 30; DEPARTURE 45.0 Feet Left; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM --; GROUND SURFACE 577.4; ROCK SURFACE 464.4; BOTTOM OF HOLE 452.4; WATER TABLE 565

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
	Previously drilled hole by Department of Highways - Hole No. 3							
60.0								
	Clay till	Grey, firm to soft, becoming siltier towards the bottom, contains angular pebbles						
113.0								
	Bedrock	Soft calcareous shale with limestone bands and nodules. No water gain or loss Practically no core recovery						
125.0								

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson — APPROVED: D. H. MacDonald

LOGGED BY: G. Wilson — DATE: December 11, 1959

Plate V

FORM NO. 91-A

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 5-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 3:00 pm. September 29 1959
FINISHED 12:30 pm. September 30 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. AX
ROCK: -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 47+42
DEPARTURE 42.0 Feet Left
BEARING --
INITIAL DIP 90 degrees
OTHER DIPS --

ELEVATIONS: DATUM G.S.C.
DRILL PLATFORM --
GROUND SURFACE 576.8
ROCK SURFACE 459.8
BOTTOM OF HOLE 459.8
WATER TABLE 571

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
	Previously drilled hole by Department of Highways - Hole No. 1							
55.0								
	Clay till	Grey, firm to soft, contains angular pebbles						
83.0								
	Silty sand	Compact. No water loss or gain.						
86.0								
	Clay	Brown, soft and plastic						
117.0								
	Rock	Struck bedrock or boulder lying on bedrock						

SAMPLING METHOD
*A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson
LOGGED BY: G. Wilson

APPROVED: D. H. MacDonald
DATE: December 11, 1959

Plate VI

FORM NO. 91-A

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 6-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 2:00 p.M. September 26 19 59
FINISHED 5:30 p.M. September 28 19 59

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. AX
ROCK: Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 46 + 82
DEPARTURE 40.0 Feet Right
BEARING --
INITIAL DIP 90 degrees
OTHER DIPS --

ELEVATIONS: DATUM G.S.C.
DRILL PLATFORM --
GROUND SURFACE 576.9
ROCK SURFACE 457.9
BOTTOM OF HOLE 446.9
WATER TABLE 571

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
		Previously drilled hole by Department of Highways - Hole No. 2						
55.0								
	Clay till	Grey, contains angular pebbles						
84.0								
	Stratified clay, till and silty sand	Alternating bands of compact silty sand and soft clay till.						
119.0								
	Bedrock	6-inch gneiss boulder lying on top of soft calcareous shale with fossils and interbedded with thin hard shales and limestone lenticles						
130.0								

SAMPLING METHOD
*A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson
LOGGED BY: G. Wilson
APPROVED: D. H. MacDonald.
DATE: December 11, 1959

Plate VII

FORM NO. 91-A

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS NIAGARA FALLS CANADA

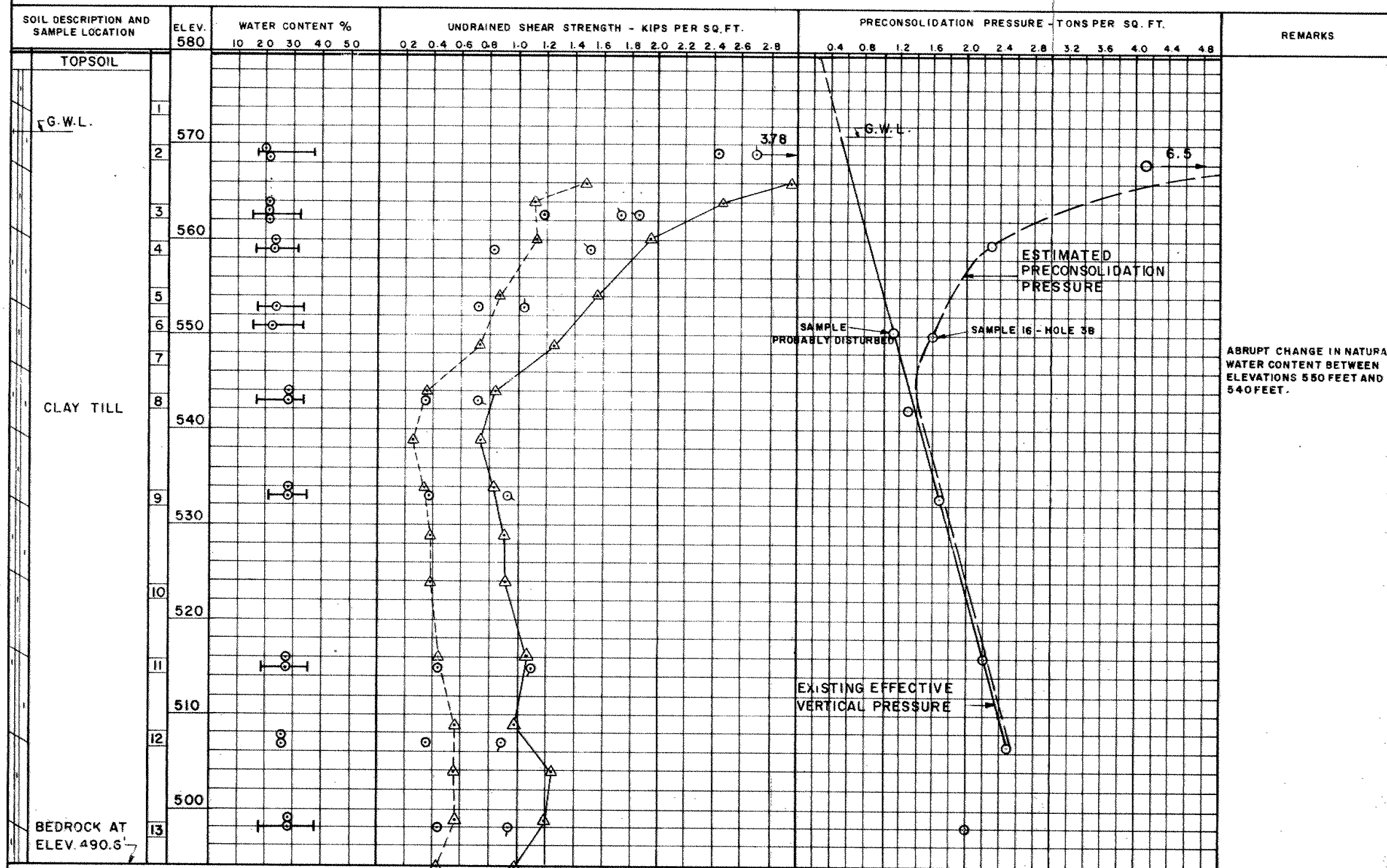
SUMMARY OF DRILLING & TEST RESULTS

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO. 837

PROJECT HIGHWAY 401 AND TILBURY CREEK CROSSING — HOLE NO. 2B

SITE TILBURY CREEK, ONTARIO — SHEET 1 OF 1

LOCATION CH. 48 + 20, 55 FEET RIGHT — ELEVATION DATUM G.S.C.



⊙ NATURAL WATER CONTENT
—I LIQUID LIMIT
⊢ PLASTIC LIMIT
⊙ UNDRAINED COMPRESSION TEST
△ FIELD VANE TEST
3 SOIL SAMPLE - SEE DRILLING REPORTS
15-○-5 (0, 10) FAILURE STRAIN
——— NATURAL STRENGTH
- - - - REMOULDED STRENGTH

APPROVED D. H. MacDonald
DATE DEC. 11, 1959

PLATE VIII

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS NIAGARA FALLS CANADA

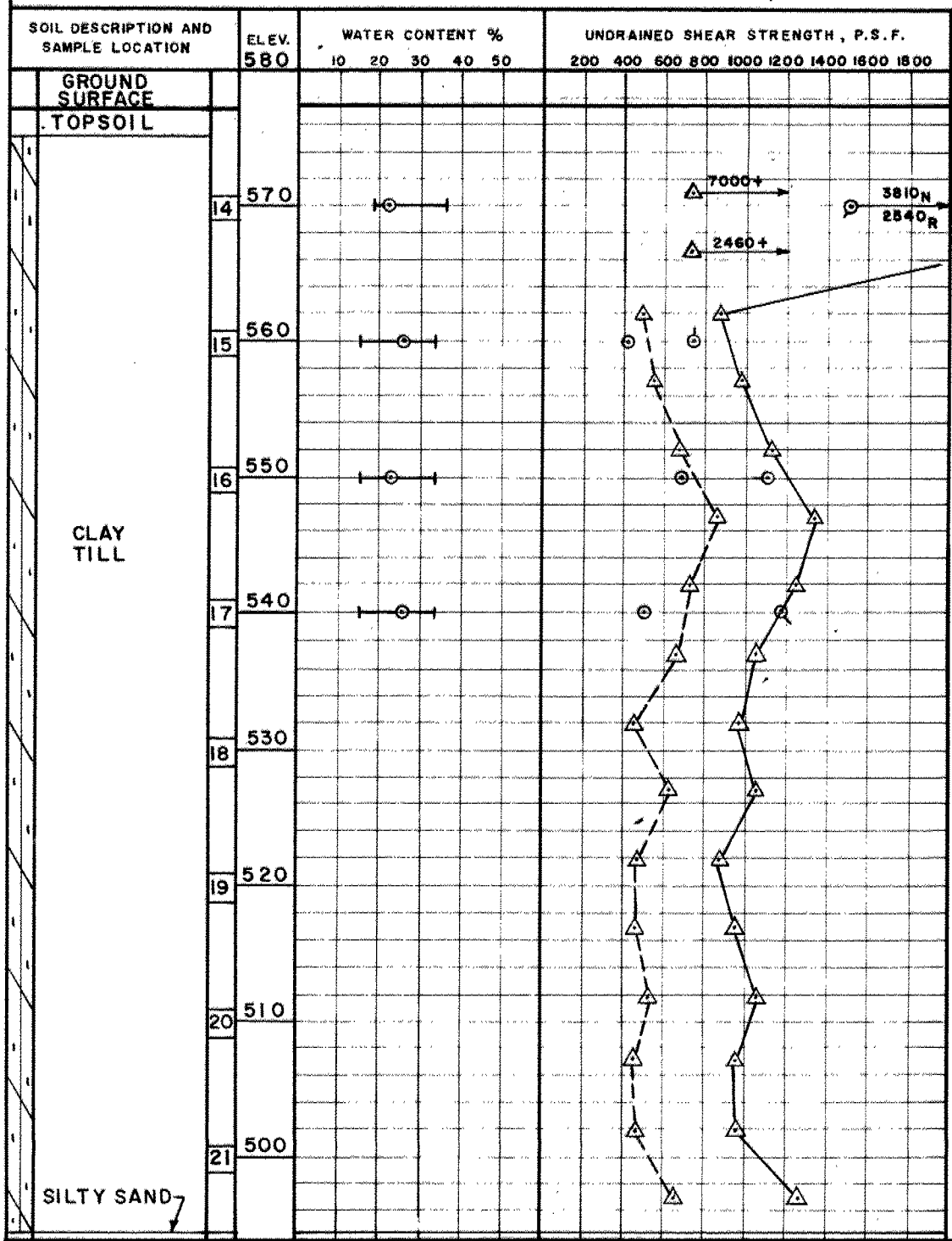
SUMMARY OF DRILLING & TEST RESULTS

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO JOB NO. 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING HOLE NO. 3B

SITE TILBURY CREEK, ONTARIO SHEET NO. 1 OF 1

LOCATION CH. 47 + 17, 8.0 FEET LEFT ELEVATION DATUM G.S.C.



⊙ – NATURAL WATER CONTENT
⊣ – LIQUID LIMIT
⊢ – PLASTIC LIMIT
⊙ – UNDRAINED COMPRESSION TEST
△ – FIELD VANE TEST
3 – SOIL SAMPLE - SEE DRILLING REPORTS

— NATURAL STRENGTH
--- REMOULDED STRENGTH
0 15 5 10 FAILURE STRAIN

APPROVED D. H. MacDonald
DATE DEC. 11, 1959

PLATE - IX

H. G. ACRES & COMPANY LIMITED

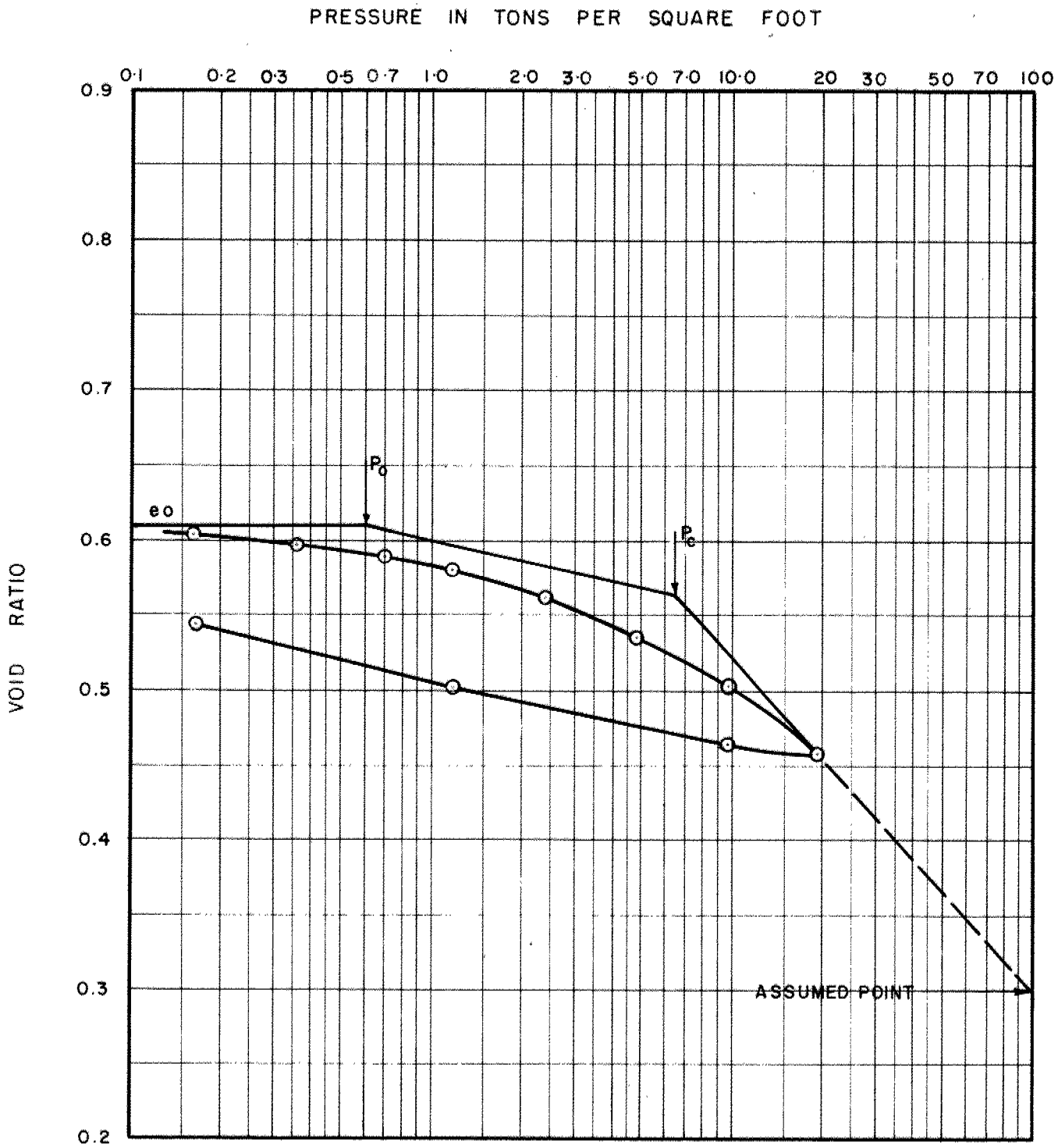
CONSULTING ENGINEERS NIAGARA FALLS CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO JOB NO. 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING HOLE NO. 2B

SITE TILBURY CREEK ONTARIO SAMPLE ELEVATION 569



P_o – OVERBURDEN PRESSURE

P_c – PRECONSOLIDATION PRESSURE

LOADING INTERVAL – 25 MIN.

APPROVED D. H. MacDonald

DATE DEC. 11, 1959

PLATE – X

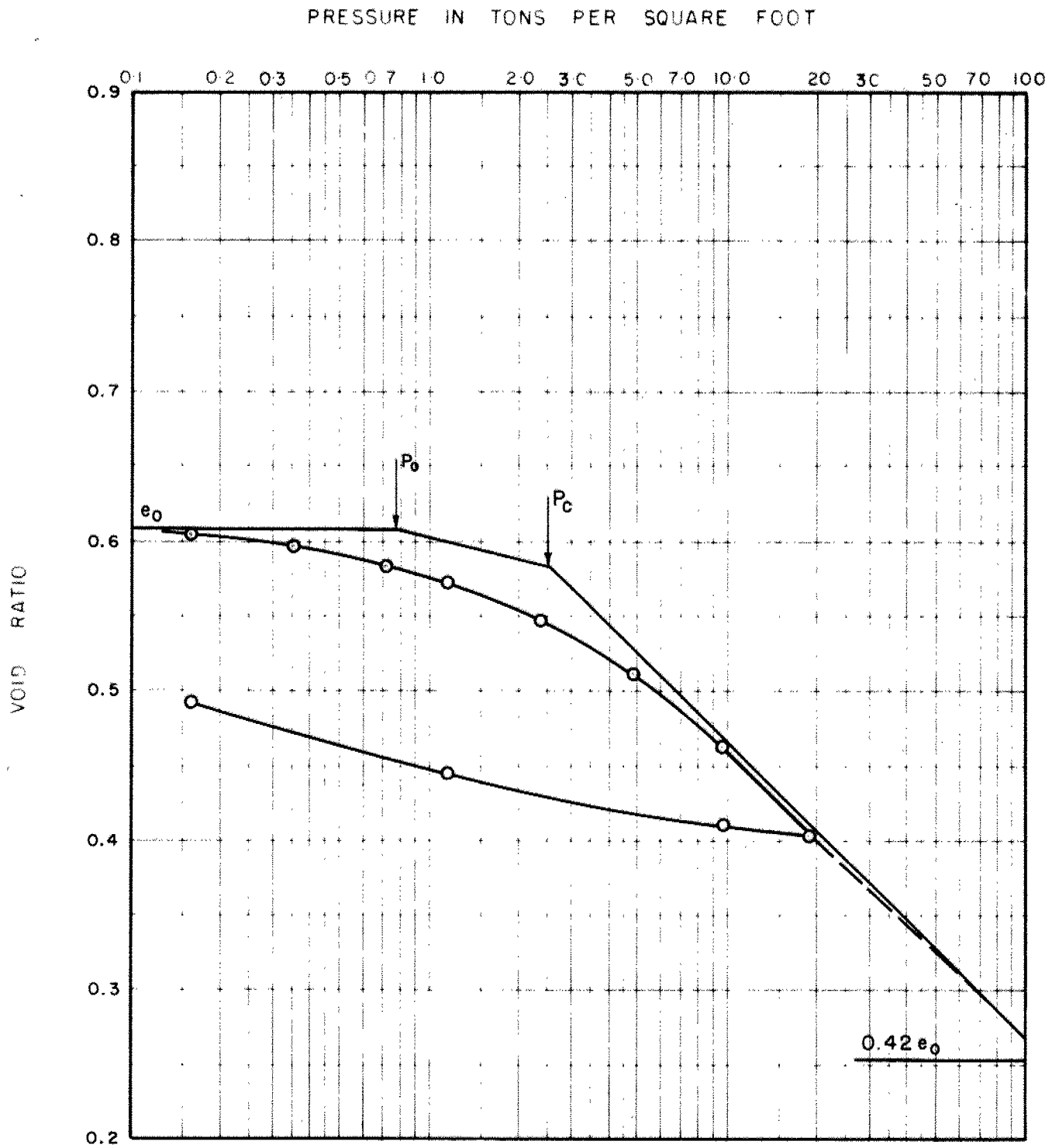
H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING — HOLE NO 2B

SITE TILBURY CREEK ONTARIO — SAMPLE ELEVATION 563



P_0 - OVERBURDEN PRESSURE
P_c - PRECONSOLIDATION PRESSURE
LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald.
DATE DEC. 11, 1959

PLATE — XI

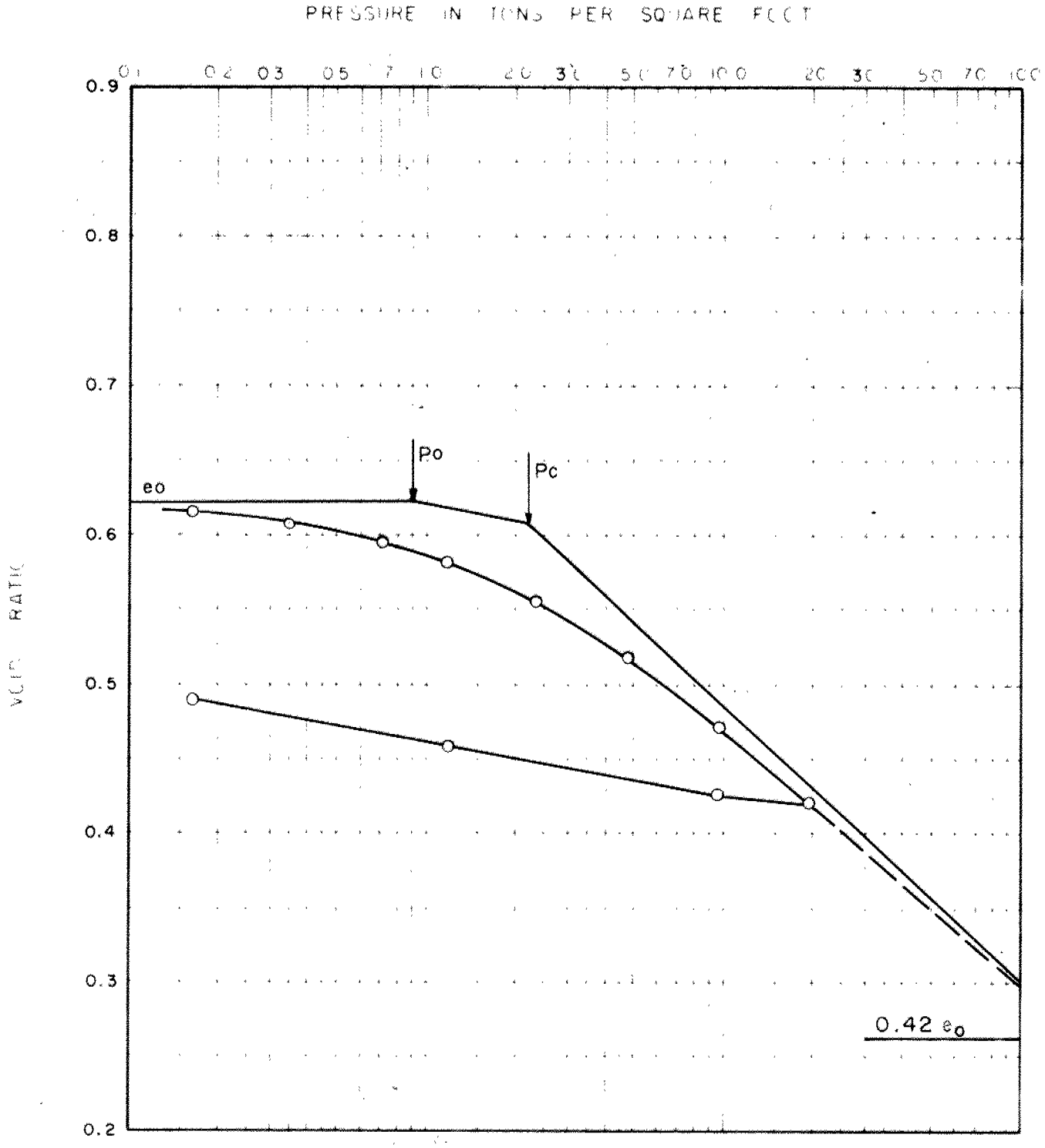
H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO. 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING — HOLE NO. 2B

SITE TILBURY CREEK ONTARIO — SAMPLE ELEVATION 559



P_o - OVERBURDEN PRESSURE
P_c - PRECONSOLIDATION PRESSURE
LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald
DATE DEC. 11, 1959

PLATE — XII

H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS

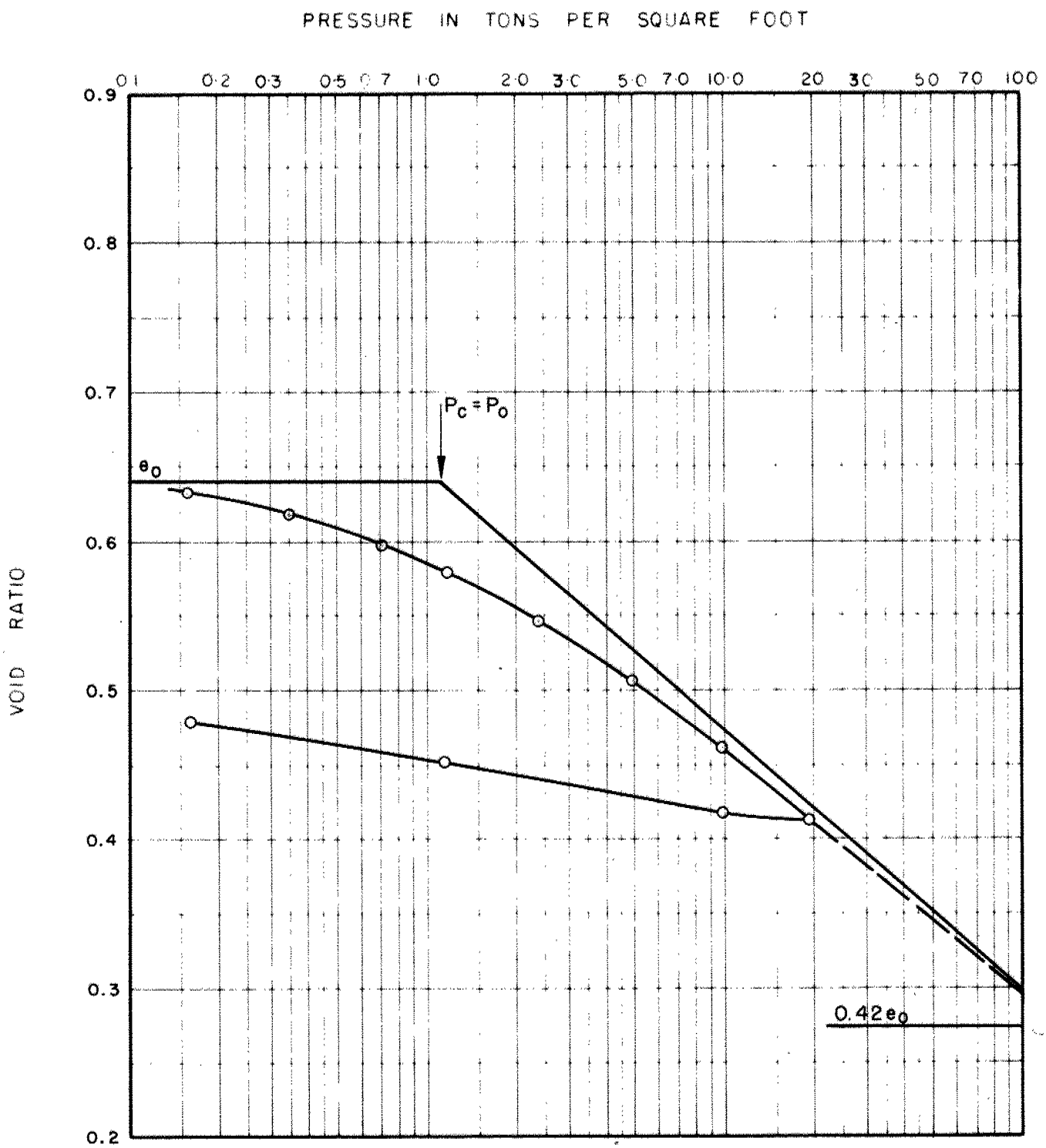
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING — HOLE NO 2B

SITE TILBURY CREEK ONTARIO — SAMPLE ELEVATION 551



P_o - OVERBURDEN PRESSURE

P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald.

DATE DEC. 11, 1959

PLATE — XIII

H. G. ACRES & COMPANY LIMITED

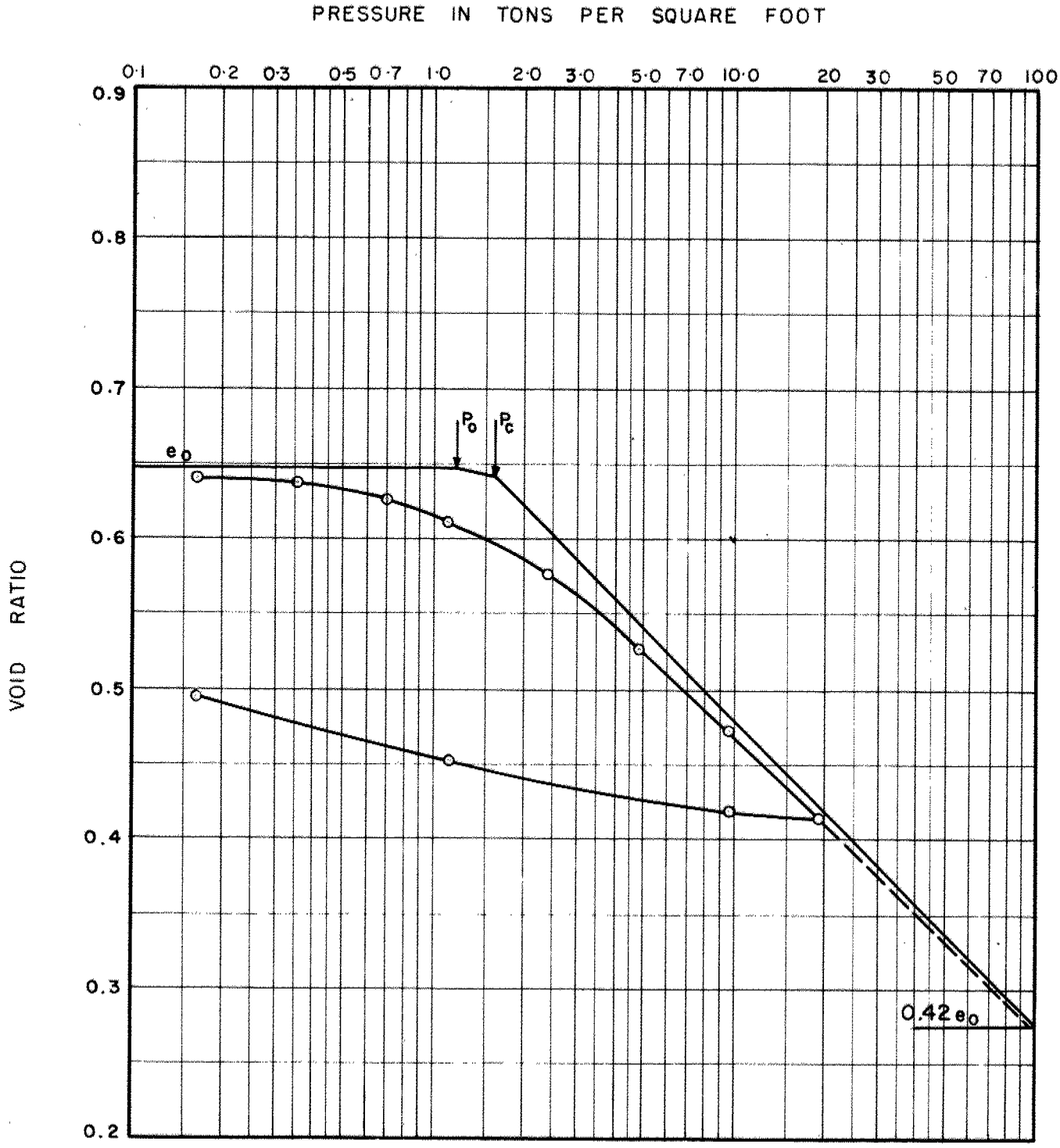
CONSULTING ENGINEERS NIAGARA FALLS CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO JOB NO. 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING HOLE NO. 3B

SITE TILBURY CREEK, ONTARIO SAMPLE ELEVATION 550



P_0 - OVERBURDEN PRESSURE

P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald

DATE DEC. 11, 1959

PLATE- XIV

H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS

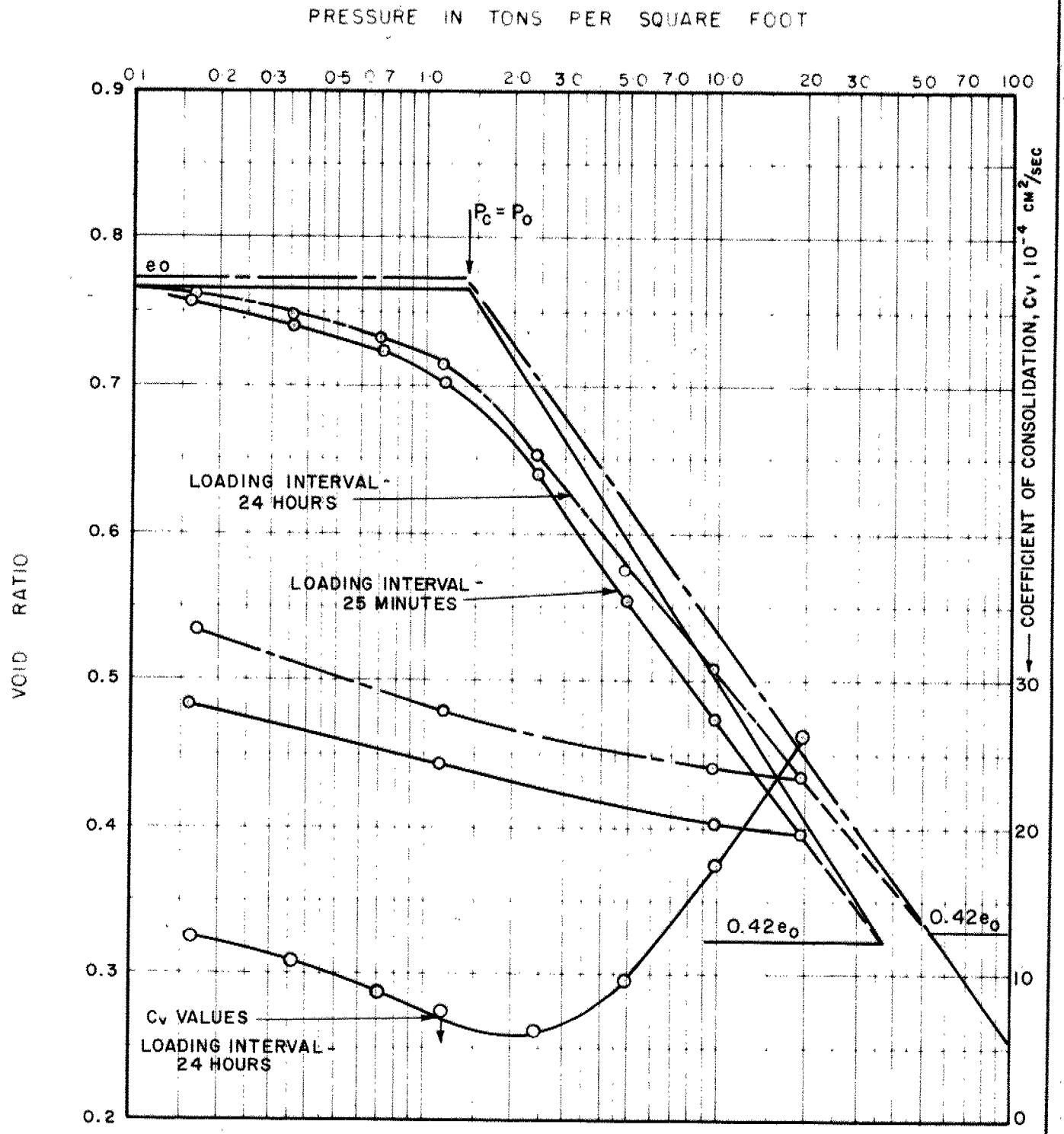
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING — HOLE NO 2B

SITE TILBURY CREEK ONTARIO — SAMPLE ELEVATION 543



P_o - OVERBURDEN PRESSURE

P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - AS NOTED

APPROVED D. H. MacDonald

DATE DEC. 11, 1959

PLATE — XV

H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS

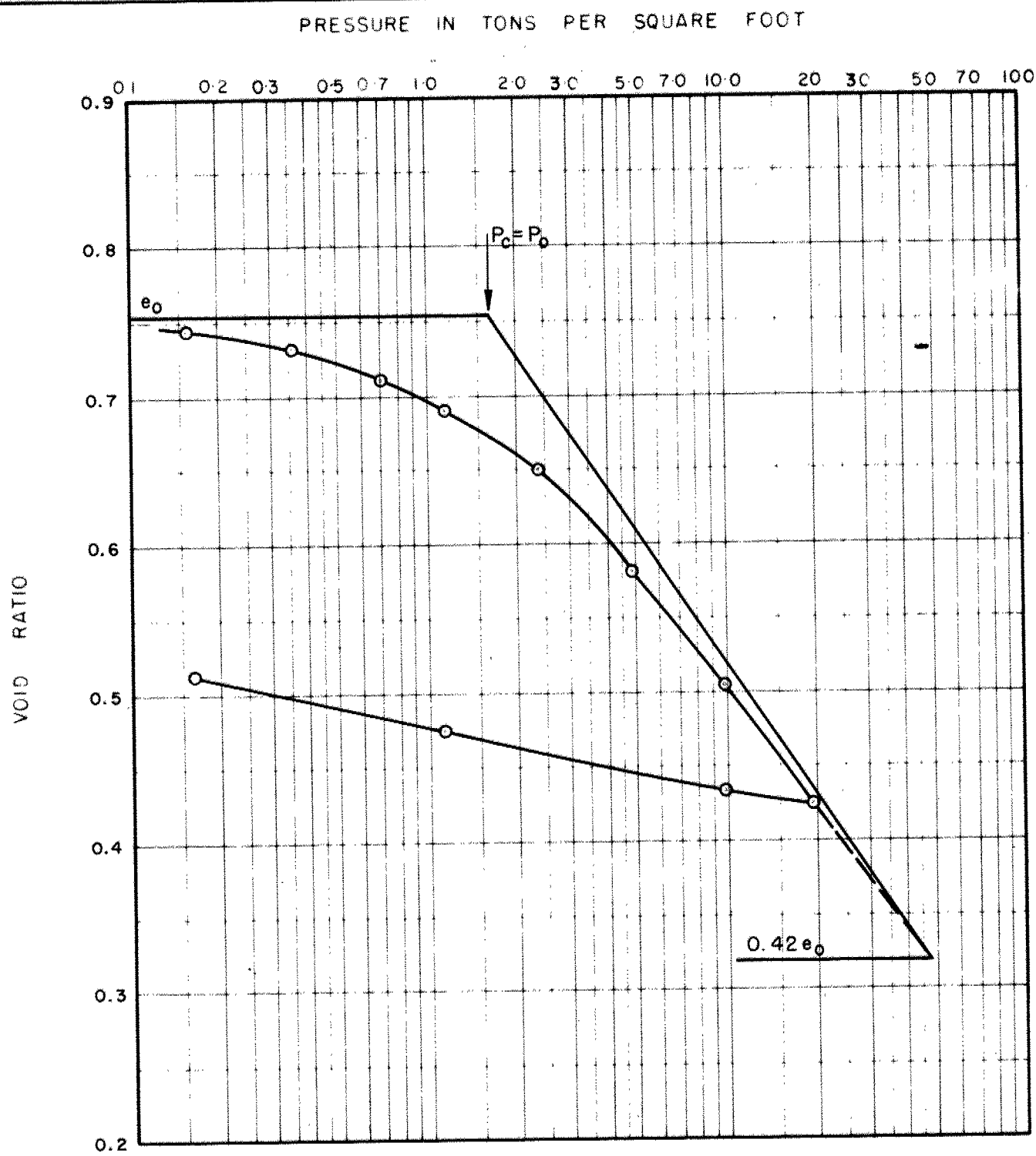
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO JOB NO. 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING HOLE NO. 2B

SITE TILBURY CREEK ONTARIO SAMPLE ELEVATION 533



P_o - OVERBURDEN PRESSURE

P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald.

DATE DEC. 11, 1959

PLATE — XVI

H G ACRES & COMPANY LIMITED - CONSULTING ENGINEERS

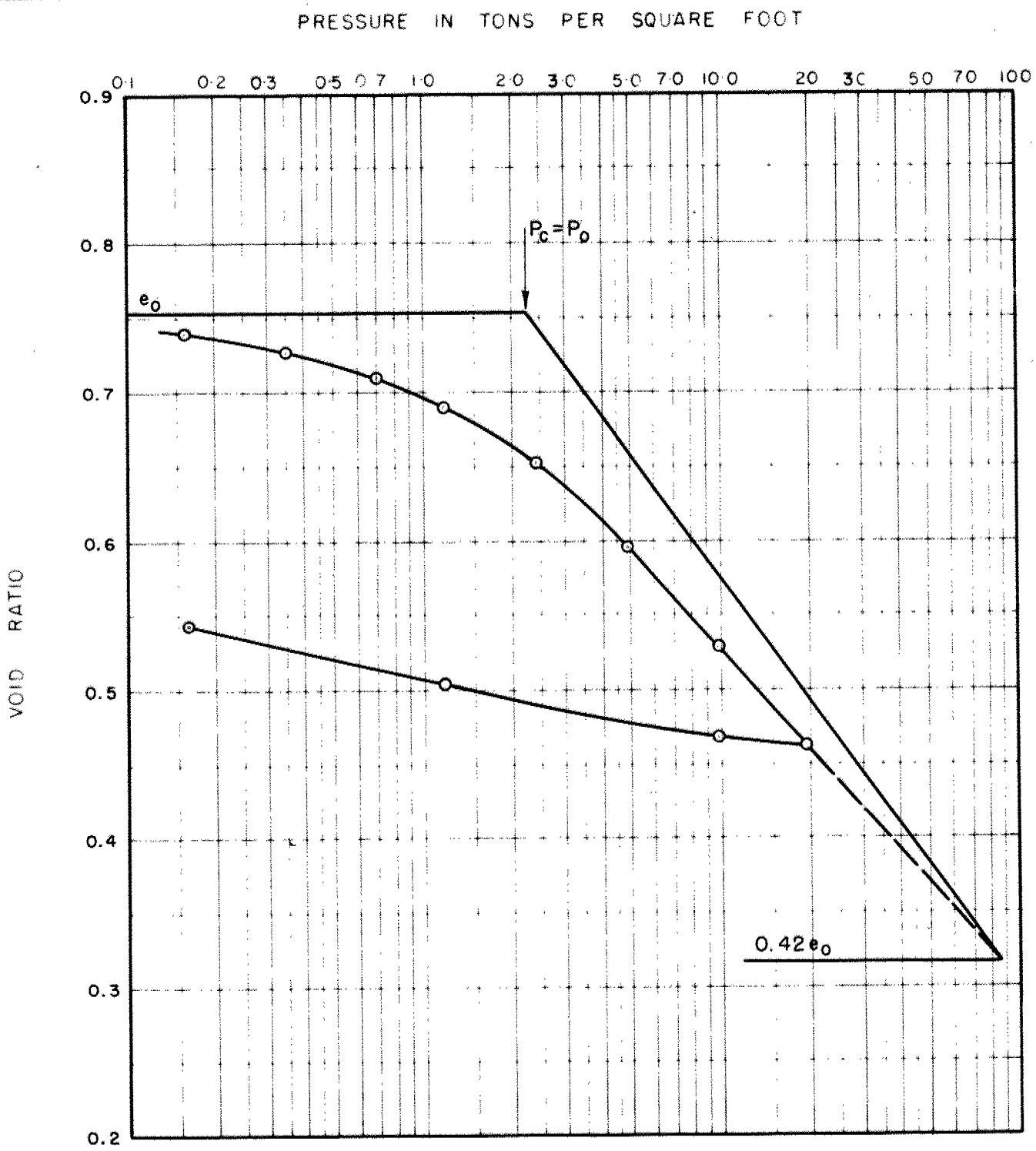
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING — HOLE NO 2B

SITE TILBURY CREEK ONTARIO — SAMPLE ELEVATION 515



P_o - OVERBURDEN PRESSURE

P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald.

DATE DEC. 11, 1959

PLATE — XVII

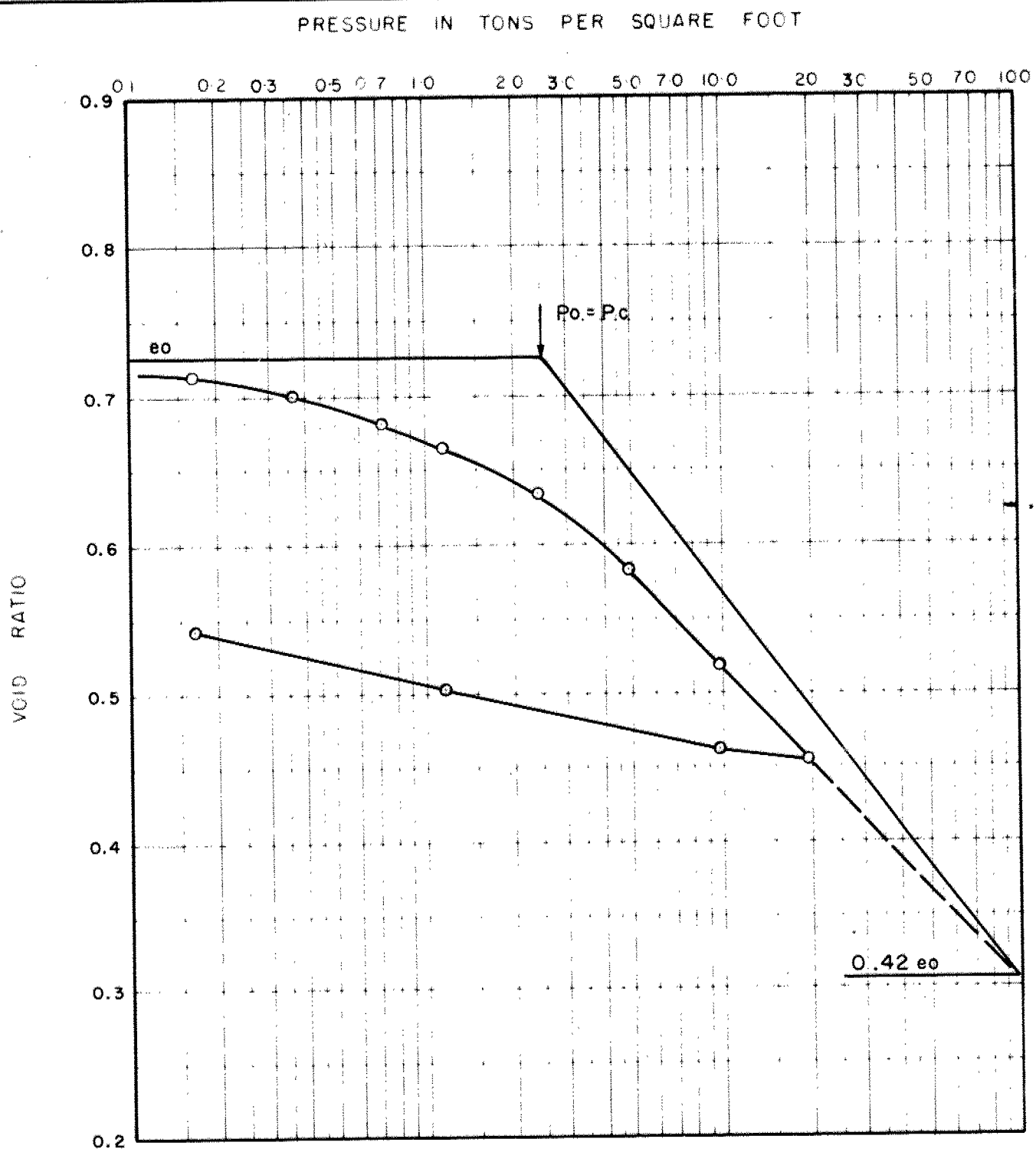
H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO — JOB NO 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING — HOLE NO 2B

SITE TILBURY CREEK ONTARIO — SAMPLE ELEVATION 507



P_o - OVERBURDEN PRESSURE
P_c - PRECONSOLIDATION PRESSURE
LOADING INTERVAL-25 MIN.

APPROVED D. H. MacDonald
DATE DEC. 11, 1959

PLATE — XVIII

H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS

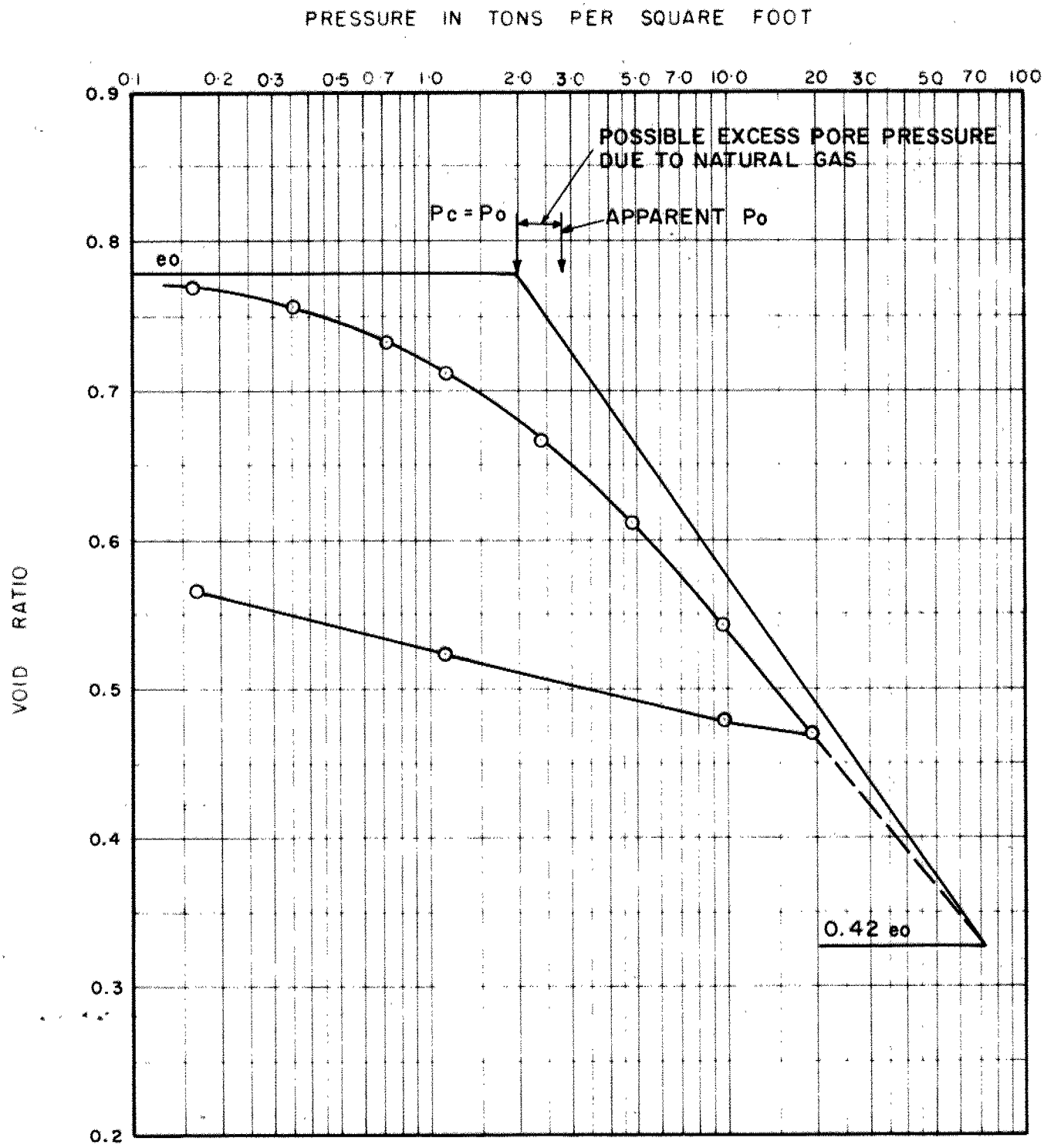
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO JOB NO 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING HOLE NO 2B

SITE TILBURY CREEK ONTARIO SAMPLE ELEVATION 498



P_o - OVERBURDEN PRESSURE

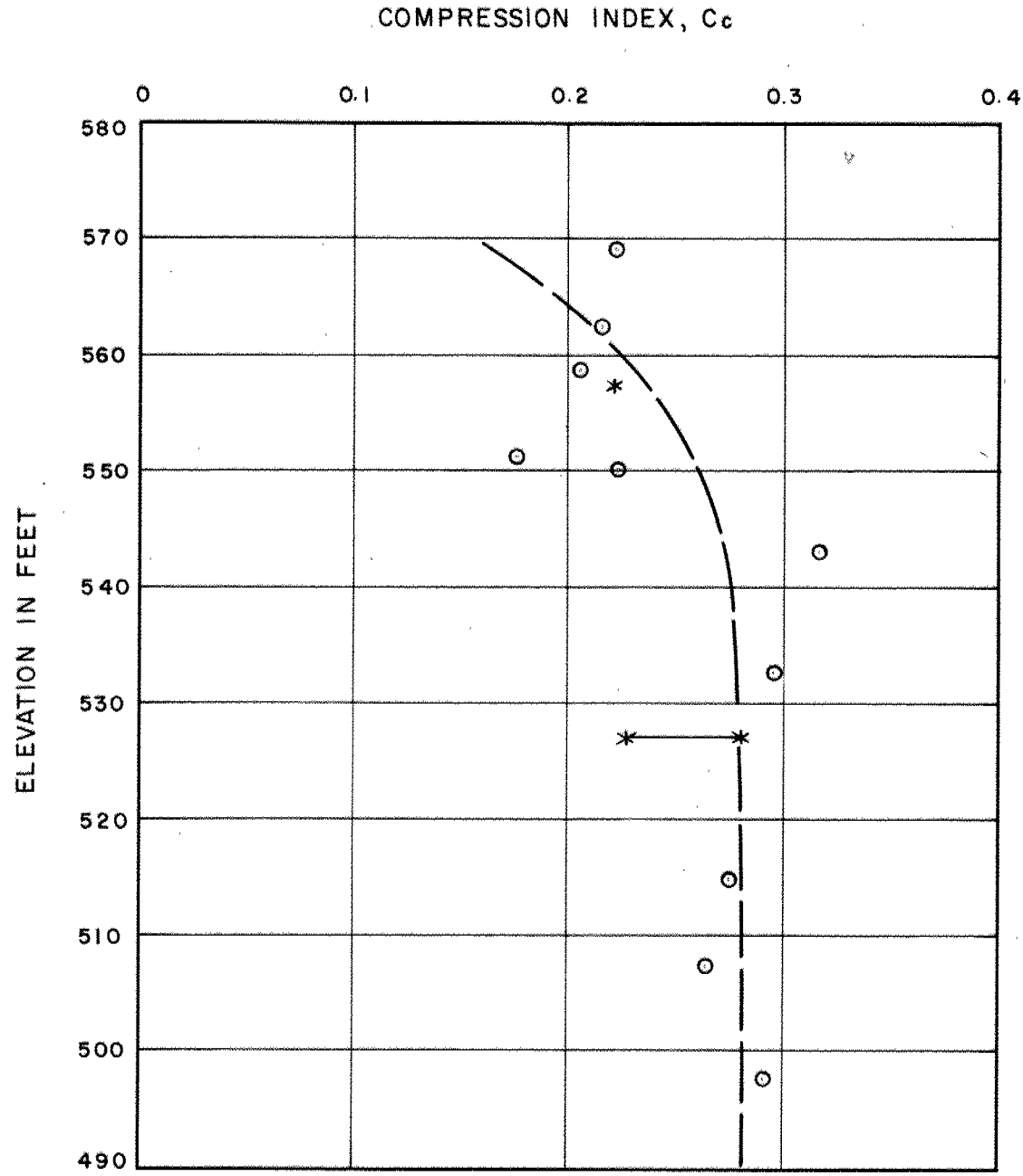
P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - 25 MIN.

APPROVED D. H. MacDonald.

DATE DEC. 11, 1959

PLATE — XIX



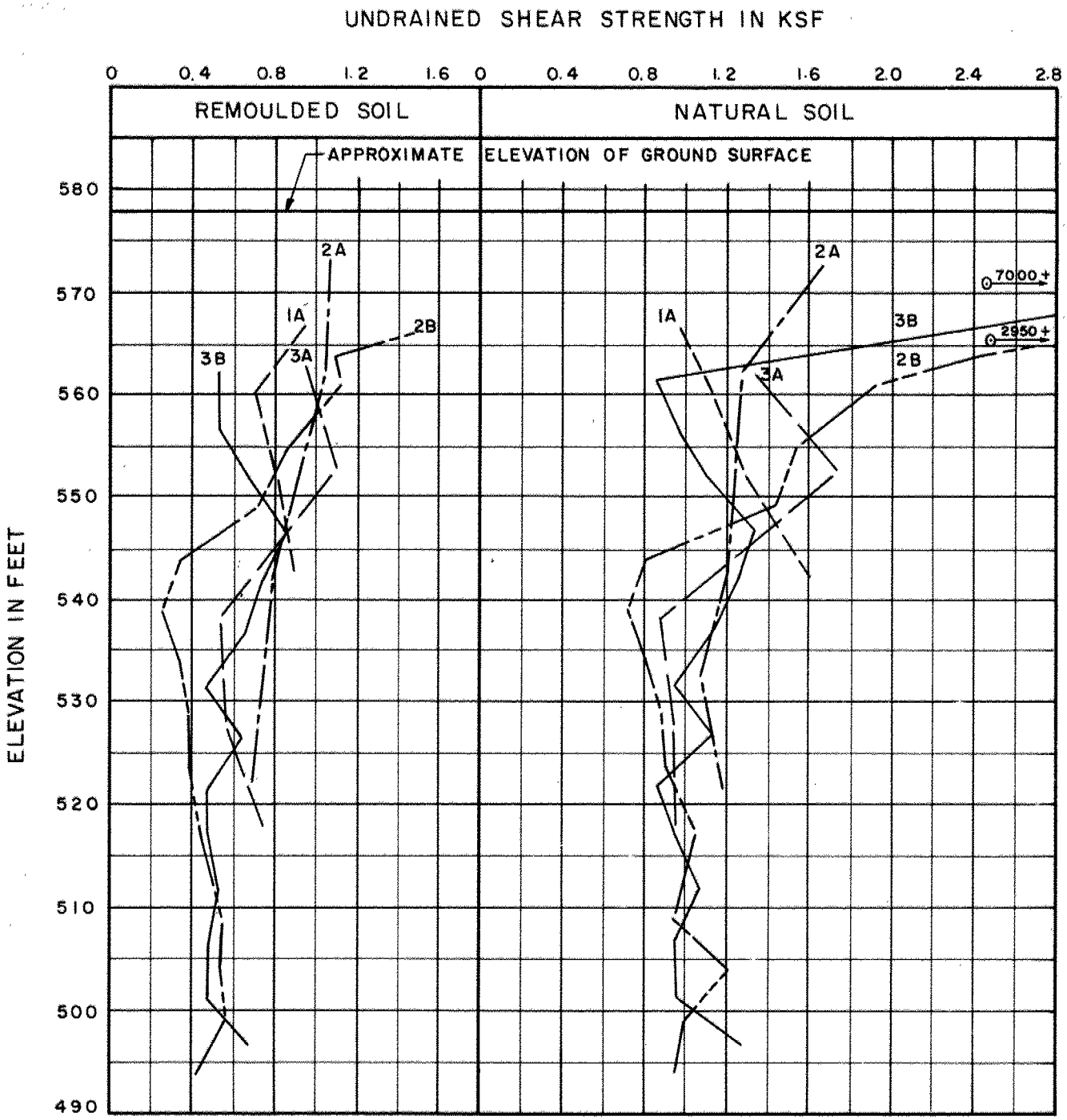
$$Cc = \frac{e_0 - e}{\log_{10}\left(\frac{P_0 + \Delta P}{P_0}\right)}$$

LEGEND:

⊙-LABORATORY TEST BY ACRES

✱-LABORATORY TEST BY PETO

H. G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY. 401 AND TILBURY CREEK CROSSING	
VARIATION OF COMPRESSION INDEX WITH DEPTH	
APPROVED	DATE: DEC. 11, 1959
D. H. MacDonald H. G. ACRES & COMPANY LTD.	SCALE / JOB No. 837
	PLATE - XX



NOTE:

HOLES No. 1A, 2A, 3A - DEPARTMENT OF HIGHWAYS OF ONTARIO.

HOLES No. 2B, 3B - ACRES

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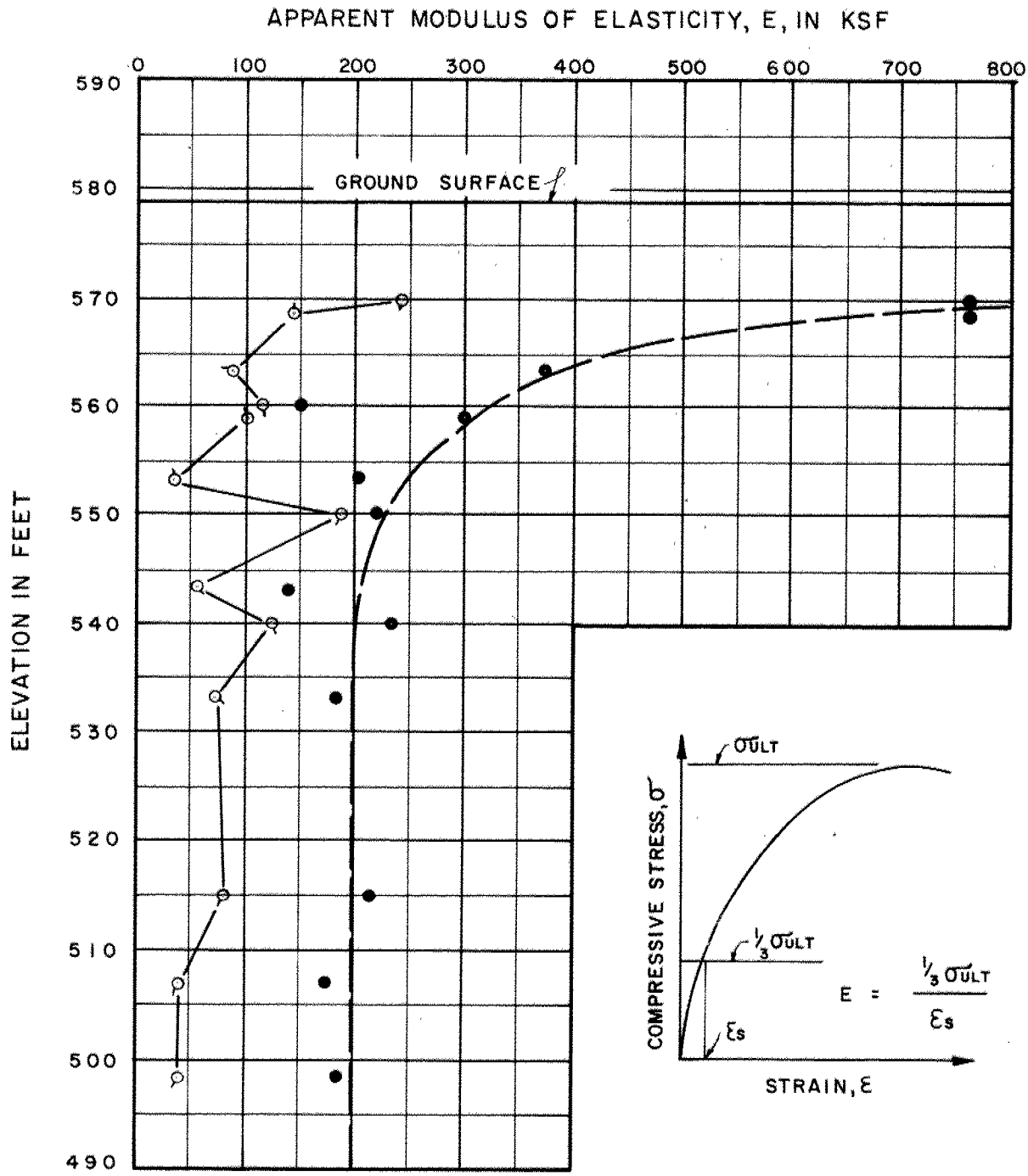
NIAGARA FALLS CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

HWY. 401 AND TILBURY CREEK CROSSING

COMPARISON OF RESULTS OF FIELD VANE TESTS

APPROVED	DATE: DEC. 11, 1959	
D. H. MacDonald.	SCALE	JOB No. 837
H. G. ACRES & COMPANY LTD.	PLATE - XXI	



LEGEND:

- ○ LABORATORY TEST RESULT
- ● CORRECTED LABORATORY TEST RESULT ASSUMING STRAIN AT FAILURE TO BE 1.0%.

0
15 — 5
10
OBSERVED FAILURE STRAIN

H. G. ACRES & COMPANY LIMITED
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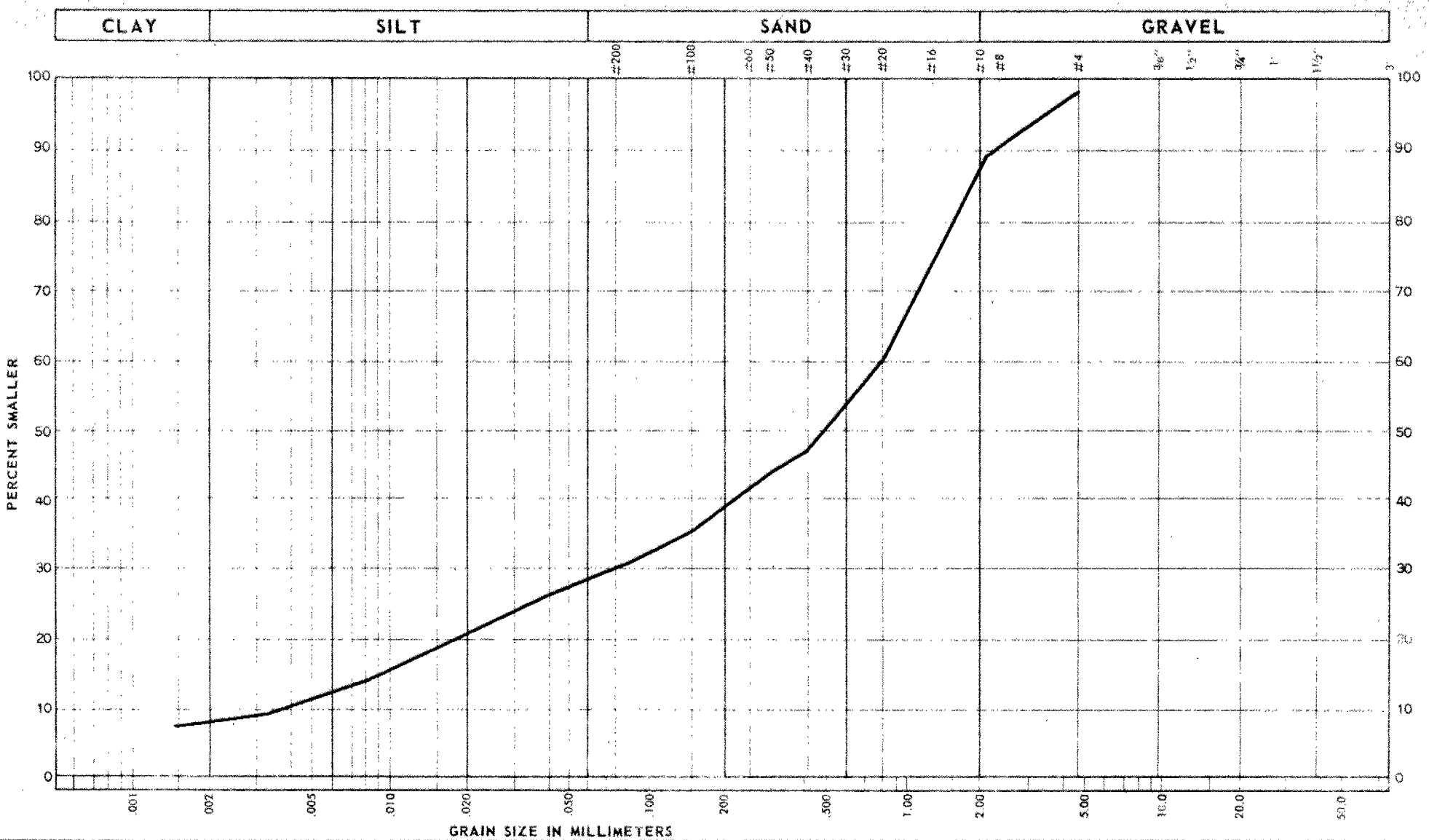
NIAGARA FALLS CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

HWY. 401 AND TILBURY CREEK CROSSING

VARIATION OF APPARENT MODULUS OF ELASTICITY WITH DEPTH

APPROVED	DATE: DEC. 11, 1959	
D. H. MacDonald	SCALE	JOB No. 837
H. G. ACRES & COMPANY LTD.	PLATE - XXII	



REMARKS:

SILTY SAND - EL. 494.0'

GRAIN SIZE DISTRIBUTION

HOLE No. 3B	DATE: DEC. 11, 1959
SAMPLE No. CO-22	TESTED BY
DEPTH 83 FT.	APP. D. H. MacDonald
DWG. No. PLATE-XXIII	JOB No. 837

H. G. ACRES & COMPANY LIMITED

CONSULTING ENGINEERS

NIAGARA FALLS, CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

HWY. 401 AND TILBURY CREEK CROSSING

FORM NO. 12-F.P.

H. G. ACRES & COMPANY, LIMITED
NIAGARA FALLS, CANADA

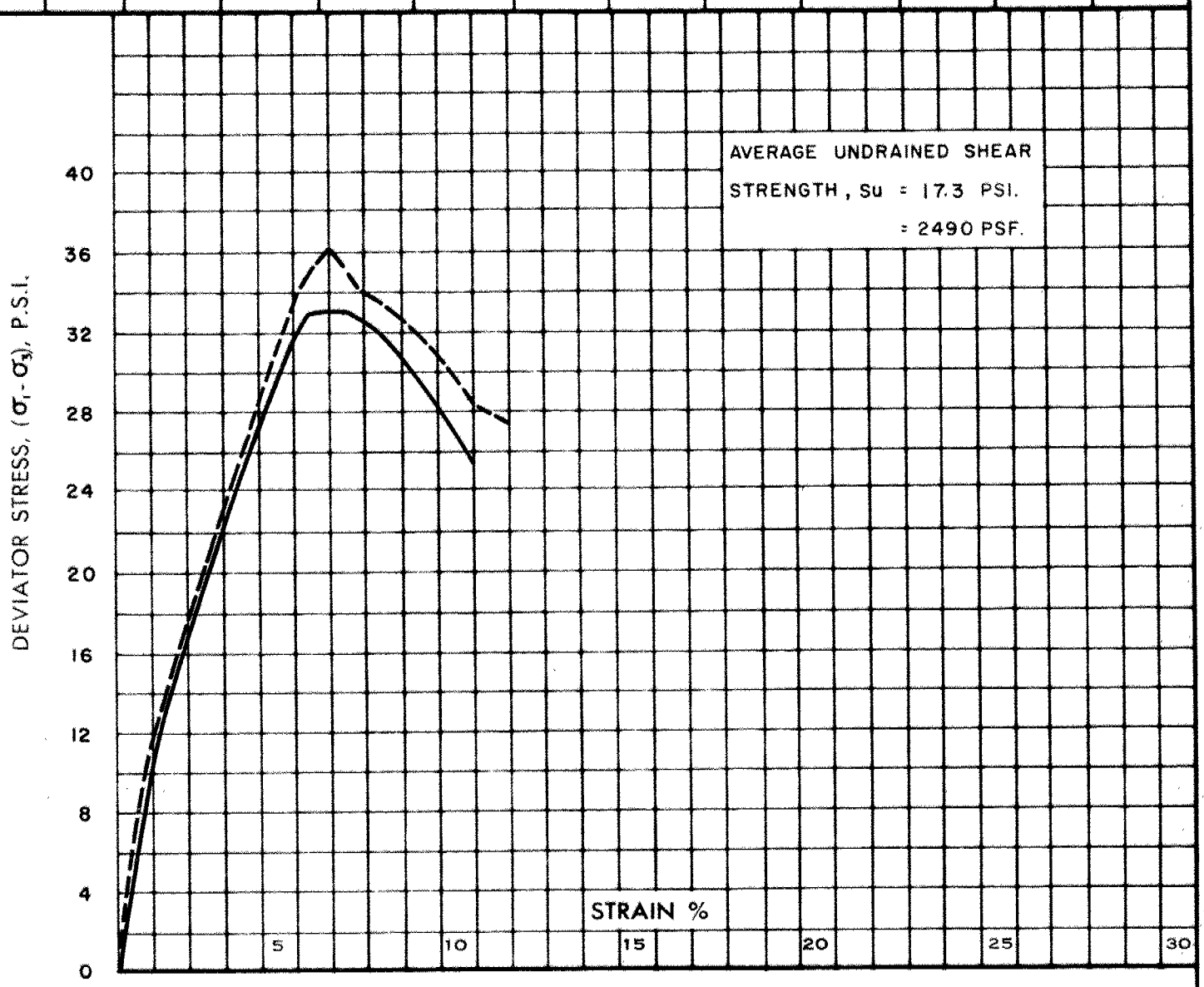
GEOTECHNICAL LABORATORY

TRIAXIAL COMPRESSION TEST
TEST RESULTS

SAMPLE No.	837	HS	1

SAMPLE LOCATION: ROCK GLEN, ARKONA, ONTARIO

TEST No		σ_c PSI	σ_3 PSI	ε_f %	u_f PSI.	σ'_{3f} PSI.	$(\sigma_1 - \sigma_3)_f$ P.S.I.	σ'_{1f} P.S.I.	ω_o %	γ_{Do} PCF.		
12	18	—	25	6.0	—	—	33.2	—	14.08	123.0		
12	19	—	25	6.0	—	—	36.1	—	14.06	124.0		



DEPARTMENT OF HIGHWAYS OF ONTARIO

HWY. 401 AND TILBURY CREEK CROSSING

RESULTS OF COMPRESSION TESTS ON SAMPLE OF SHALE OUTCROP

APPROVED	DATE: DEC. 11, 1959	
D. H. MacDonald.	SCALE	JOB No. 837
H.G. ACRES & COMPANY LTD.	PLATE - XXIV	

FORM No. 157

H. G. ACRES & COMPANY LIMITED - CONSULTING ENGINEERS

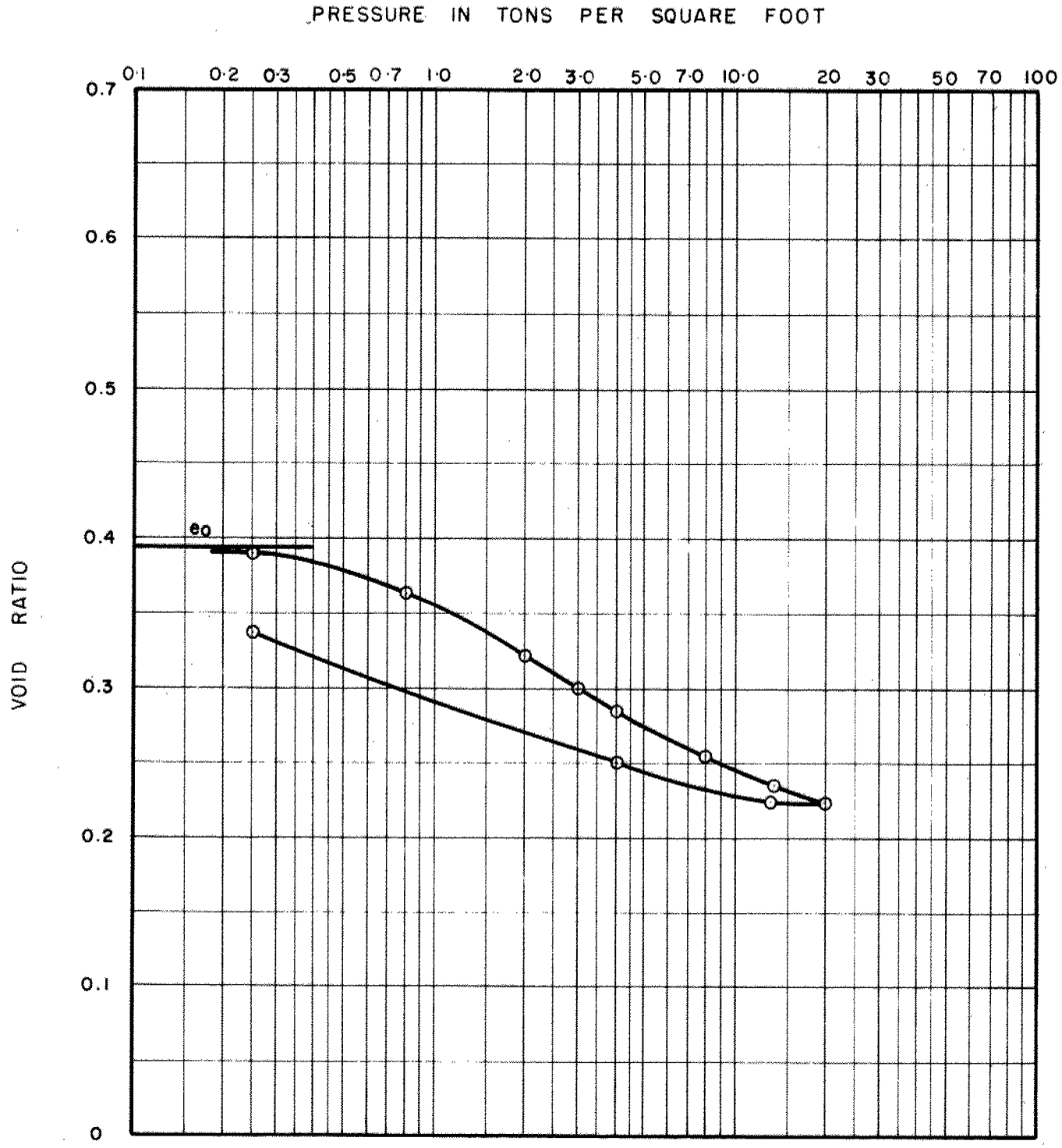
NIAGARA FALLS, CANADA

CONSOLIDATION TEST

CLIENT DEPARTMENT OF HIGHWAYS OF ONTARIO JOB NO. 837

PROJECT HWY. 401 & TILBURY CREEK CROSSING HOLE NO. ——

SAMPLE LOCATION ROCK GLEN, ARKONA, ONTARIO



P_o - OVERBURDEN PRESSURE

P_c - PRECONSOLIDATION PRESSURE

LOADING INTERVAL - 24 HOURS

APPROVED D. H. MacDonald

DATE DEC. 11, 1959

PLATE — XXV

ONTARIO DEPARTMENT OF HIGHWAYS

Toronto, Ontario

REPORT
on
SETTLEMENT STUDY
OF THE TILBURY CREEK BRIDGE
(WP 160-58)

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

40J8-21
GEOCRES No.

REPORT

on

SETTLEMENT STUDY OF THE TILBURY CREEK BRIDGE (WP 160-58)

H.G. ACRES & COMPANY LIMITED
Consulting Engineers
Niagara Falls, Canada

June, 1960

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

SETTLEMENT STUDY OF THE TILBURY CREEK BRIDGE (WP 160-58)

Table of Contents

1 - Introduction

2 - Methods of Settlement Analysis

3 - Estimated Settlements - Embankment

4 - Estimated Settlements - Bridge Piers

5 - Combined Settlements due to Embankment and Bridge Pier Loadings

6 - Time Rate of Settlement

7 - General Considerations

Appendix 1 - Consideration of Apparent Modulus of Elasticity of Soil

Appendix 2 - Embankment Settlement
A - Effect of Surface Crust
B - Elastic Settlements
C - Induced Foundation Stresses
D - Consolidation Settlements

Appendix 3 - Bridge Pier Settlement
A - Net Stresses Under Pier Base
B - Induced Foundation Stresses
C - Elastic Settlements
D - Consolidation Settlements

Appendix 4 - Combined Embankment and Pier Settlement

List of References

Table 1

Calculation Sheets - 1 to 43

List of Plates - I to VII

ONTARIO DEPARTMENT OF HIGHWAYS
Toronto, Ontario

REPORT

on

SETTLEMENT STUDY OF THE TILBURY CREEK BRIDGE (WP 160-58)

1 - Introduction

Much of southwestern Ontario is covered with deposits of clay till which in many places extend to depths greater than 100 feet. These deposits are generally capped with a stiff clay crust, but below this crust the soil is in a normally-consolidated state; that is, it appears to have a stress history which suggests that it has never been subjected to loads greater than those due to the present weight of the overlying soil. Because these soil deposits may be quite thick, and because they are normally consolidated and compressible, the imposition of structural loads may cause large settlements. In the case of bridges, the resultant settlement pattern can become complex, depending upon the bridge type

and size, the angle of skew, the approach embankment loadings, and the foundation soil properties.

The Ontario Department of Highways is planning to build a large number of bridges and approach embankments in this area, and it was decided to investigate the settlements that might be experienced by a typical bridge. The chosen structure will be on the proposed 401 Highway at Tilbury Creek (Plate I). The creek is to be spanned by two simply-supported bridges 36 feet wide, spaced 74 feet apart, and having a skew angle of 48 degrees. The perpendicular distance between the faces of the bridge piers is 50 feet. The approach embankments are 13 feet high, 122 feet wide at the top, and they abut against the bridge piers over their full height. The soil conditions at the site have been studied in detail and the results of these investigations are contained in the following reports:

(a) - Foundation Conditions at the Tilbury Creek Crossing on Highway 401 (WP 160-58). December 15, 1959. By H.G. Acres & Company Limited.

(b) - Foundation Investigation Highway 401 and Tilbury Creek Crossing. July 30, 1959. By Ontario Department of Highways.

(c) - Soils Report for Highway 401 Crossing of Tilbury Creek in Tilbury North Township No. 2. February, 1959. By E.M. Peto Associates Limited.

The general approach to the problem of estimating the settlement pattern around the bridge piers, was to consider separately the loading effects of the embankment and the bridge, and then to determine by superposition their combined effect. The embankment was considered as two strip loadings extending up to the face of the bridge piers, and the stress distributions and elastic settlements were calculated at several points. Then, separately, the stress distributions and the elastic settlements resulting from the net change in loading caused by the bridge, were determined for several points. By adding together the stress changes caused by the embankment and by the bridge, the total changes in stress were calculated and with these the total consolidation settlements were determined. Total settlements were then found by adding together the elastic and consolidation settlements. This approach was found to be necessary because the calculation of the stress distribution in the underlying soil became

extremely complex as a result of the skewed position of the bridge, the complicated loading pattern, and the layered properties of the soil.

This study presented an excellent opportunity to study the following points:

(a) - The general pattern of settlement of an embankment and bridge supported by a clay layer resting upon a rigid base.

(b) - The effect of the obliquity of the bridge on the differential settlements of the bridge piers.

(c) - The effect of a surface clay crust on the settlement magnitudes.

2 - Methods of Settlement Analysis

At the present time, the accepted method of predicting settlements is to assume that settlement is made up of two components, an elastic or immediate component which takes place at the same time as the load is applied and which is a result of soil deformation at constant volume, and a consolidation component which is the result of water being expelled from the soil as a result of stress changes in the soil (Skempton and Bjerrum, 1957). The procedure for predicting settlements is contained in the following four equations:

$$\rho_i = \int_0^z \left(\frac{\Delta\sigma_z}{E} - \frac{\mu}{E} \cdot \Delta\sigma_X - \frac{\mu}{E} \cdot \Delta\sigma_Y \right)$$

$$= q \cdot B \left(\frac{1-\mu^2}{E} \right) I_\rho \qquad \text{..... (1)}$$

$$\rho_c = \int_0^z \nu \cdot \Delta\sigma_z \cdot m_v \cdot dz$$

$$= \int_0^z \nu \cdot q \cdot I_\sigma \cdot m_v \cdot dz$$

$$= \int_0^z \nu \left(\frac{\Delta e}{1+e_o} \right) dz \qquad \text{..... (2)}$$

$$\rho_u = \rho_i + \rho_c \qquad \text{..... (3)}$$

$$\rho_t = \rho_i + U \cdot \rho_c \qquad \text{..... (4)}$$

where

ρ_i	denotes immediate or elastic settlement.
ρ_c	denotes consolidation settlement.
ρ_u	denotes net ultimate settlement.
ρ_t	denotes settlement at time "t".
z	denotes depth below foundation.
B	denotes width of foundation.
q	denotes unit surface loading.
$\Delta\sigma_z$	denotes change in unit vertical stress at depth z.
E	denotes apparent modulus of elasticity of soil.
μ	denotes Poisson's ratio of soil (generally assumed to be 0.5).
m_v	denotes modulus of vertical compressibility of soil determined from oedometer tests.
Δe	denotes change in void ratio of soil.
e_o	denotes initial void ratio of soil.
ν	denotes correction factor for consolidation settlement estimated from the results of oedometer tests.
U	denotes degree of consolidation.
I_ρ	denotes settlement influence factor.
I_σ	denotes stress influence factor.

The stress and settlement influence factors in equation No. 1 and No. 2, I_σ and I_ρ, are evaluated by the theory of elasticity. As it is an extremely difficult and time-consuming task to evaluate these factors from first principles, use is made of the various numerical solutions of problems which, fortunately, have been published, although the special boundary conditions applicable in any instances are restrictive in scope. The solutions which were found to approximate most closely the different conditions in this particular problem, and which were used in the calculations contained in this report, are as follows:

(a) - Stress distributions in a semi-infinite homogeneous elastic mass (the Boussinesq problem) beneath flexible circular and rectangular loads. The results are presented in graphical form on Plate II as vertical stress influence factors at various depths (Terzaghi, 1943). In the case of a rectangular load, the plotted stress influence factors define conditions below one corner of a loaded area, and as the law of superposition is valid within the elastic range, the vertical stress distribution beneath any point due to a uniform load of any shape, can be determined by adding and subtracting the appropriate influence factors.

(b) - Stress distribution in a three-layer system of elastic materials in which the modulus of elasticity of the

separate layers decreases with increasing depth (Acum and Fox, 1951). A solution was obtained for the case of a circular load only, and the stress influence factors at the interfaces of the three layers are listed on Plate III.

(c) - Stress distributions and settlement influence factors for two-layer rigid-base systems (Burmister, 1956). Solutions were obtained for different sizes of loaded areas and different depths to the rigid base, and the results are presented in graphical form as vertical stress influence factors and settlement influence factors under the corner of the rectangular load (Plate IV). This solution can be used in exactly the same manner as the solution of the Boussinesq problem (Plate II), to obtain the vertical stress distribution and the elastic settlement beneath any point due to a uniform load of any shape.

Values of the soil properties E, m_v, Δe and e_o in equation No. 1 and No. 2, are obtained from the results of laboratory tests on undisturbed samples. For the purpose of this study, the foundation soil has been divided into three layers and average compressibility and deformation properties have been assigned to each layer (Plate V) on the basis of the laboratory test results recorded in the report by H.G. Acres & Company Limited, dated December 15, 1959.

Values of the correction factor, ν, which is applied to consolidation settlements estimated from the

results of oedometer tests, is dependent upon the foundation stress system and the pore pressure properties of the soil. For the case of strip loadings resting on a semi-infinite homogeneous material, values can be obtained from Plate VI where ν is plotted as a function of the pore pressure parameter "A", and the depth is expressed as a fraction of the strip width (Skempton and Bjerrum, 1957; Wood, 1959).

3 - Estimated Settlements - Embankment

(a) - Loading - To simplify the calculations, the embankment section has been converted to an equivalent uniform strip load as shown on Plate I. The width of the strip has been taken to be 142 feet, the height to be 13 feet over a strip of infinite length, and for purposes of simplification the angle of skew was assumed to be 45 degrees rather than the actual value of 42 degrees. The faces of the embankment at the edge of the creek were taken to be 50 feet apart. The distance to bedrock was assumed to be constant at 118 feet, and the embankment fill was assumed to have a density of 100 pcf.*

(b) - Effect of Stiff Clay Crust - The maximum thickness of clay crust which could be considered to act as a layer of material whose stiffness is much greater than that of the underlying soil, is approximately 20 feet, and from laboratory tests it was found that the apparent modulus of elasticity of this soil is from three to five times greater than the apparent modulus of elasticity of the underlying soil. There are, however, several uncertainties which raise doubts concerning the use of the results of laboratory

* After most of the calculations had been completed it was realized that this assumed value was low. A more realistic value is 135 pcf.

undrained compression tests to evaluate the apparent elastic properties of a layered soil, and these are discussed in Appendix I. For the purpose of this report, however, it will be assumed that these laboratory results are applicable.

In Appendix 2-A, a check was made to determine the effect of the stiff crust on the distribution of stresses induced in the foundation soil. Using a circular loaded area of 140-foot diameter, a crust thickness of 35 feet, and a ratio of moduli of elasticity of five for the crust and the underlying soil, it was found that the crust reduced the stresses by less than 10 per cent. For a thinner crust and the case of a strip loading, the reduction of stresses would be even less than 10 per cent. Therefore, to simplify the calculations without affecting their accuracy appreciably, it has been assumed that the embankment rests on a layered soil which has uniform elastic properties but variable compressibility characteristics.

(c) - Location of Points of Calculated Settlement - Calculations were made for all the points indicated on Plate I so that the pattern of settlements and, in particular, the pattern of differential settlements,

could be studied. The lines of σ points are located well back from the creek and the bridge, so that there is no influence from these latter features on the settlement of the embankment foundation. The lines of Q points are located along the centreline of the creek, the lines of P points are located along the faces of the bridge piers, and the lines of P' points are located 10 feet back from the lines of P points.

(d) - Elastic or Immediate Settlements - By making the assumptions that the foundation soils have uniform elastic properties, and that a rigid layer is located at a depth of 118 feet, the elastic or immediate settlements can be calculated for the different settlement points by the use of the Burmister Settlement Charts shown on Plate IV. These calculations were laborious, as the conditions of this problem required the superposition of many loaded areas in order to obtain the final stresses. The calculations are given in Appendix 2-B and tabulated in Table 1. A value of apparent modulus of elasticity, E, equal to 200 ksf, was used. These calculations indicate that immediate settlements along the centreline of the embankment can be expected to vary from 0.13 feet in the centre of the creek to 0.45 feet well back from the bridge piers.

(e) - Consolidation Settlements - To calculate the consolidation settlements, it was first necessary to determine the stress distributions below each of the settlement points. This was done with the use of the Burmister Stress Charts, shown on Plate IV, by superimposing loaded areas of different sizes and shapes. These calculations are given in Appendix 2-C and the resulting stresses are shown graphically in Appendix 2-C, sheets 15 to 17, inclusive.

The consolidation settlements were determined by dividing the soil into five horizontal layers and considering each layer separately; the total consolidation settlement would be the sum of the settlements taking place in each of the separate layers estimated by means of equation 2. The compressibility and pore pressure characteristics of the soils given on Plate V were used. Since no laboratory compression tests with pore pressure measurements were carried out, values for the pore pressure parameter "A" had to be assumed in order to evaluate the oedometer correction factor. The calculations of consolidation settlement due to embankment loading are given in Appendix 2-D and the results are tabulated in Table 1.

4 - Estimated Settlements - Bridge Piers

(a) - Pier Loadings and Foundation Stresses - A simple spread footing type of bridge pier was assumed for the purposes of this report. The base width of the footing was taken to be 13 feet.* It was assumed that the piers would be rigid in the direction parallel to the creek and would have a uniform settlement equal to the maximum settlement of a similar flexible load. This assumption overestimates the pier settlements, but as these settlements are small in comparison with the settlements due to the embankment loading, the errors involved are sufficiently small to be neglected. It was further assumed that the piers rested on a 12-foot thick crust which was five times more rigid than the underlying soil. The values of apparent modulus of elasticity which were used for the crust and the underlying soil are equal to 1,000 and 200 ksf, respectively.

The net loading increase due to the bridge piers was found to be equivalent to a 6-foot wide strip which transfers a uniform pressure 1.6 ksf to the soil. These calculations and the determination of the stress changes below points C, D, E, and F,

* During discussions of this report with personnel from the Ontario Department of Highways and De Leuw Cather & Company of Canada Limited, it was established that the footing width for this size of pier would be in the order of 16 feet.

located on Plate I, are given in Appendices 3-A and 3-B respectively.

(b) - Elastic Settlements - The elastic settlements at points C, D, E, and F, were calculated from the changes in vertical stress by making simplifying assumptions concerning the relationships between the changes in vertical stress and the changes in the horizontal stresses (Appendix 3-C). The magnitude of these settlements is of the order of 0.02 to 0.04 feet, which is quite small in comparison with the settlements due to the embankment. These settlements are listed in Table 1.

(c) - Consolidation Settlements - The consolidation settlements at points C, D, E, and F, were calculated by dividing the foundation into five horizontal layers, and determining the compression of each layer due to the stress changes caused by the addition of the pier loads. The compressibility and pore pressure characteristics of the soils given on Plate V were used. The calculations are given in Appendix 3-D and the results are tabulated in Table 1.

5 - Combined Settlements due to Embankment and Bridge Pier Loadings

The combined settlements due to the embankment and pier loadings, were ascertained by first determining by interpolation, the settlement of the bridge pier areas due to the embankment loading. It has been assumed that the bridge piers are rigid in a direction parallel to the creek, but flexible in a direction at right angles to the creek, and therefore the pattern of settlements resulting from the embankment loading was adjusted to comply with the rigidity characteristics of the piers.

To this settlement pattern was added the settlements caused by the bridge pier loading (Appendix 4). The results of these calculations are shown on Plate VII.

The average total ultimate settlements at the centres of the bridge piers will be 1.13 feet for the pier adjacent to the 45-degree corner of the embankment and 1.29 feet for the pier adjacent to the 135-degree corner, and the differential settlements in a longitudinal direction will be 0.40 and 0.20 feet for the respective piers. The differential settlements in the direction perpendicular to the face of

the bridge piers in the case of the pier adjacent to the 45-degree corner of the embankment varies from 0.07 feet away from the creek to 0.09 feet towards the creek, and for the pier adjacent to the 135-degree corner, the differential settlements vary from 0.11 feet to 0.16 feet away from the creek. There is, therefore, an indication from the calculated differential settlements that the bridge piers will be acted upon by torsional forces, especially the piers located adjacent to the 45-degree corner of the embankment. If the piers are relatively rigid, the differential settlements may be appreciably less than those calculated.

The effective reduction in load on the foundation soils at the acute-angled corner of the embankment, causes the settlement of the end of the bridge pier nearest this corner to be less than the settlement of the end of the pier nearest to the obtuse-angled corner of the embankment, whereas the settlement of the ends of these piers nearest the centreline of the highway, are approximately equal. Therefore, bridge pier settlements will cause racking of the bridge structure.

A limited investigation was made of the effect of additional loads placed at the acute-angled corner of the embankment in order to reduce these settlements causing racking. On Sheets 39 to 43 are given the results for such an investigation; a rectangular load of 1.30 kips per square foot, 60 feet x 72 feet was placed at the acute-angled corner of the embankment as shown on Plate I. It was found that with this load the bridge pier would be subjected to very nearly the same settlement pattern as the pier across the creek and, therefore, such a load would essentially eliminate racking of the bridge structure.

6 - Time Rate of Settlement

The time rate of settlement has been estimated on the assumption that the embankment and pier loads are applied instantaneously and simultaneously, and that drainage can take place towards the ground surface and towards the bedrock surface (that is, double drainage). The equivalent total thickness of a homogeneous soil deposit was assumed to be 100 feet and the average value of C_v was determined from laboratory tests to be 29.2 feet^2 per year.

The average rate of settlement of the bridge piers is shown graphically on Plate VII. The calculations indicate that 15 to 25 per cent of the total settlement will be elastic strain, and this will occur immediately upon the imposition of the load. The consolidation settlement will occur gradually thereafter, and 50 per cent of the total settlement will be complete in approximately 9 years and 80 per cent in approximately 40 years.

This rate of settlement is probably too slow, and it is reasonable to expect that consolidation will occur more quickly than has been estimated here due to the presence of sandy layers and the resulting horizontal drainage. However, in view of the apparently

slow rate of settlement, it is likely that the final computed total and differential settlements will not be realized within the economic life of the structure.

7 - General Considerations

Throughout this report the changes in the foundation stresses have been estimated by assuming that the soil acted as an elastic material, and that the loads applied at the ground surface, were generally completely flexible. These assumptions are necessary in view of the limited amount of numerical evaluations of stress distributions in elastic or plastic media that are now available. The assumption that the soil acts as an elastic material is in all probability fairly valid because the soil is stressed to a small degree of its ultimate strength. The assumption that the applied loads are flexible is only partly true; the embankment can be rightly assumed to be flexible in view of its great width as compared with its height and its plastic nature, but the bridge piers will act essentially as rigid structures. Since the stress changes and the resulting settlements caused by the pier loadings are small in comparison with those attributable to the embankment loads, the errors involved are also small.

Uncertainties concerning the calculated stress distributions also arise as a result of the existence of the surface clay crust. There is little

doubt that this crust will aid in the distribution of applied stresses and will therefore reduce settlements, but predicting its behaviour with the mathematical solutions currently available, and our knowledge concerning the stress behaviour of a layered soil system, it is particularly questionable. In this report, the effect of the crust was ignored when the embankment loads were considered and, for this reason, the calculated settlements will be overestimated to some degree. The effect of the crust upon the settlements owing to bridge pier loadings was considered but, as these settlements were found to be small, any errors involved would similarly be small in comparison with the combined embankment and pier loadings.

The general settlement pattern of the embankment and the bridge piers will be, to a considerable extent, dependent upon the construction sequence. For instance, if the embankment is constructed initially, the elastic settlements and some portion of the ultimate consolidation settlements will take place before the piers are built and, therefore, will have no effect upon the pier movements. On the other hand, if the piers are constructed initially, their behaviour after the embankment is constructed will be

similar to those estimated in this report. The effect of construction sequence upon the settlements of the bridge piers has not been considered quantitively in this report because the number of possible time-loading conditions is very large. However, the possibility of deriving an advantage in reducing the settlements of the bridge piers by means of adjusting the construction sequence and schedule, should not be overlooked.

APPENDICES

APPENDIX I

CONSIDERATION OF APPARENT MODULUS OF ELASTICITY OF SOIL

The assigned values of apparent modulus of elasticity, E, shown on Plate V, were obtained from the results of undrained compression tests on natural samples. These values of E are, therefore, applicable to the case where loads are applied to soil which is unable to drain; that is, the construction case. In the problem at present being considered, layered system is assumed in which the surface layer is more rigid and less compressible than the soil layers at greater depth. When the loads are first applied, it is probable that the crust distributes the stresses in a manner similar to that which can be estimated by the use of elastic analysis methods, such as that published by Acum and Fox (1951). However, as consolidation progresses, the softer clay below the crust consolidates, causing further deformation of the crust which is then forced to take more and more of the load, thus reducing the load transferred to the lower strata. The action of the crust is similar to a flat plate overlying a compressible mass. Ultimately, equilibrium will be

reached and the final stress distribution will be established. Although the undrained apparent modulus of elasticity of each separate soil stratum will probably not change appreciably because of consolidation, consolidation might cause the soil strata in a drained state to act as if the crust is much stiffer than the softer clay below. Whether or not this is actually so would depend upon the following comparatively unknown considerations:

(a) - Whether the crust is capable of withstanding the tensile stresses induced by bending. Failure in tension would reduce the effective thickness of the crust.

(b) - Whether the shear stresses induced in the crust would gradually relax with time. If the creep strains were large enough, it would be possible for the soil deposit to act ultimately as a homogeneous material.

Despite these several uncertainties, the ratio of the E values of the various soil strata obtained from undrained tests, will be used to calculate the ultimate stress patterns in the foundation soil. However, it is evident from the above

discussion that, depending upon the behaviour of the crust, it is possible for the effective ratios of the E values to be much larger if the crust can withstand the tensile shear stresses, but it is also possible that the soil will act similarly to a homogeneous material (that is, the E ratios become equal to unity) if the shear stresses in the crust relax sufficiently.

APPENDIX 2

EMBANKMENT SETTLEMENT

A - Effect of Surface Crust

The stress distribution in a layered soil which has a stiff surface crust and overlying a rigid layer, has not yet been solved, at least to the degree where graphical or tabulated results are available. Therefore, to determine the effect of a surface crust upon the stresses induced in the soil due to the application of a load, a comparison was made of the stresses induced by a uniform circular load in a homogeneous material of semi-infinite extent (Terzaghi, 1943), and the stresses induced in a two-layer material having semi-infinite extent (Acum and Fox, 1951). The diameter of the loaded area was assumed to be approximately equal to the width of the assumed embankment loading; that is, 140 feet. Crust thicknesses of 35 feet and 70 feet were considered. For the purpose of calculating stresses at some point below the base of the crust, a second layer thickness of 70 feet was assumed.

To convert the solution for a three-layer system to a two-layer system, the ratio of the apparent

moduli of elasticity of the two lower layers was made equal to unity, and the vertical stress influence factors at the interfaces of the layers were plotted on calculation sheets 1 and 2 (see also Plate III).

On sheet 3 are shown the resulting influence factors for K_1 values* equal to 5, 10, and 20. It can be seen that the larger the value of K_1 and the thicker the crust, the greater is its ability to distribute stress. In the problem to be studied, the thickness of the crust is less than 35 feet and the value of K_1 is less than 5. In addition, the problem deals with a strip load rather than a circular load and, therefore, the effect of the crust will be smaller than that shown on sheet 3. For these reasons, the stress distribution curve will be somewhere between the Boussinesq case and the layered case where h_1 = 35 feet, K_1 = 5. For practical purposes, therefore, it was assumed that the crust would have very little influence on the stresses induced by the embankment loading, and the problem could be simplified to an embankment resting upon a material which has homogeneous elastic properties.

$$* \quad K_1 = \frac{\text{E of crust}}{\text{E of lower layer}}$$

B - Elastic Settlements

Using the loading conditions shown on Plate I, the Burmister Settlement Chart (Plate IV), and a value of E = 200 ksf, the elastic settlements were calculated for the different settlement points. The settlement influence factors at the different settlement points were obtained by the superposition of various sizes of strips and right-angled isosceles triangular loadings. These calculations are given on sheets 4 to 7, inclusive, and the results are summarized in Table I. The settlements are shown graphically in cross sections and longitudinal sections on sheets 20a and 20b.

C - Induced Foundation Stresses

Using the loading conditions shown on Plate I, and the Burmister Stress Charts (Plate IV), the foundation stress influence factors at five different levels for the different settlement points were calculated by superimposing various sizes of strips and right-angled isosceles triangular loadings. These calculations are given on sheets 8 to 14, inclusive, and the results are given in graphical form on sheets 15 to 17, inclusive.

On sheets 16 and 17, it is of interest to note that the most highly stressed points are not located along the axis of the embankment, but near the side which has the obtuse-angled corner. It is also interesting to note that for an angle of skew equal to 45 degrees, the magnitude of the stresses below the acute-angled corner generally is less than three quarters that of the stresses below the obtuse-angled corner.

D - Consolidation Settlements

Using the stresses determined in Appendix 2-C, and the compressibility properties shown on Plate V, the consolidation settlements were calculated for the settlement points shown on Plate I by means of equation 2. The calculations are given on sheets 18 to 20, inclusive, and the results are tabulated in Table I. The settlements are shown graphically in cross sections and longitudinal sections on sheets 20a and 20b.

APPENDIX 3

BRIDGE PIER SETTLEMENT

A - Net Stresses Under Pier Base

For the purposes of this problem, a simple type of bridge pier was assumed, namely, a spread footing type of retaining wall (sheet 21). The pier has a total height of 24 feet and a base width of 13 feet. The backfill material has been assumed to have a density of 100 pcf, and to exert a hydrostatic type of horizontal pressure against the pier corresponding to an earth pressure coefficient to 0.3. The resultant force transmitted by the soil against the pier was assumed to be inclined at 35.5 degrees to the horizontal. In the settlement calculations, it was assumed that the duration of the live loads was too short to be effective and could be ignored. The pier loading calculations are given on sheets 21 and 22, and summarized on sheet 23.

The net footing pressures were obtained by reducing these calculated pressures by an amount equivalent to the weight of soil excavated (0.7 ksf), and for the weight of the embankment in those areas where the pier and the assumed embankment limit

overlap. This is shown on sheet 23 and, fortuitously, the resulting equivalent net load is a uniform load of 1.6 ksf over a 6-foot wide strip immediately adjacent to the embankment.

B - Induced Foundation Stresses

The problem of determining the change of foundation stresses caused by the addition of the bridge pier loading, is reduced to determining the stress induced by four 6-foot by 60-foot rectangular uniform loadings of 1.6 ksf, and adding these stress changes to those caused by the embankment loading. The stress changes have been calculated along vertical axes passing through the centrelines of the strips and nine feet from the centrelines; that is, points C, D, E, and F, on Plate I. In order to do this, rectangular loads 6 feet, 12 feet, and 24 feet by 60 feet long have been superimposed to obtain the correct loading conditions. For each of these loadings, the following method has been used to take into account the effect of a 12-foot thickness of clay crust below the base of the footing.

(a) - A circular loaded area having a diameter equal to the width of the rectangular loading being considered,

was assumed. The influence curves for vertical stresses below the centre of the area were calculated for the conditions of a semi-infinite homogeneous foundation ($I_{\sigma ch}$)* and for a foundation having a 12-foot crust ($I_{\sigma cc}$)** five times more rigid than the underlying soil. For these calculations use was made of Plates II and III.

(b) - The influence curve for vertical stresses below the centre of the rectangular loading for the case of a semi-infinite homogeneous foundation soil ($I_{\sigma rh}$) was calculated by using Plate II.

(c) - The influence curve for the vertical stresses below the centre of the rectangular loading resting on a stiff crust ($I_{\sigma rc}$) was then determined by assuming that:

$$\frac{I_{\sigma rc}}{I_{\sigma rh}} = \sqrt{\frac{I_{\sigma cc}}{I_{\sigma ch}}}$$

The calculations and their results are given on sheets 24 to 32, inclusive.

C - Elastic Settlements

The elastic settlements taking place below a point can be calculated by using the following equation:

$$\rho_i = \int_0^z \left(\frac{\Delta\sigma_z}{E} - \frac{\mu}{E} \cdot \Delta\sigma_x - \frac{\mu}{E} \cdot \Delta\sigma_y \right) \quad \ldots\ldots (1)$$

* $I_{\sigma ch}$ denotes vertical stress influence factor for the case of a circular load resting on a homogeneous material.

** $I_{\sigma cc}$ denotes vertical stress influence factor for the case of a circular load resting on a layered material.

To simplify the calculations it has been assumed that:

$$\mu = 0.5 \quad \text{and} \quad \Delta\sigma_X = \Delta\sigma_Y = 0.5\ \Delta\sigma_Z$$

Substituting these values into equation (1), the following expression is obtained:

$$\rho_i = 0.5 \int_0^z \frac{\Delta\sigma_Z}{E}$$

The elastic settlements at points C, D, E, and F, due to the pier loadings, were calculated from the values of vertical stress change determined in Appendix 3-B. These calculations are given on sheets 27, 28, and 32, and tabulated in Table I.

D - Consolidation Settlements

To calculate the consolidation settlements below points C, D, E, and F, it was first necessary to determine the stress distribution below these points caused by embankment and bridge pier loadings. This was accomplished by first obtaining the stress distributions due to the embankment loading by interpolation between points P_5, P_5' and P_6, P_6', and then adding the increase in stresses due to the pier loading. These calculations are given on sheet 27. The consolidation settlements due to the addition of pier loads were calculated by equation (2) as summarized on sheets 27, 28, and 32, and tabulated in Table I.

APPENDIX 4

Combined Embankment and Pier Settlement

The combined immediate settlements caused by the embankment and pier loadings were determined, and the calculations are given on sheets Nos. 33 to 36 inclusive. Initially, the immediate settlements caused by the embankment loading were adjusted to comply with the condition of rigidity of the bridge piers in a direction parallel to the creek, and then the combined settlements were obtained by adding the immediate settlements caused by the net pier loads.

The total combined settlements caused by the embankment and pier loadings, were determined and the calculations are given on sheets Nos. 37 and 38. These were obtained by the procedure used for the immediate settlements; that is, the total settlements caused by the embankment loading were adjusted to account for the rigidity characteristics of the piers, and then the total settlements caused by the net pier loadings were added.

LIST OF REFERENCES

TABLE I

LIST OF REFERENCES

(1) - Acum, W.E.A., and Fox, L., 1951
"Computation of Load Stresses in a Three-Layer Elastic System."
Geotechnique, Vol. II, pp. 293-300.

(2) - Burmister, D.M., 1956
"Stress and Displacement Characteristics of a Two-Layer Rigid Base Soil System: Influence Diagrams and Practical Applications."
Highway Research Board Proceedings, Vol. 35.

(3) - Skempton, A.W., and Bjerrum, L., 1957
"A Contribution to the Settlement Analysis of Foundations on Clay."
Geotechnique, Vol. VII, pp. 1-11.

(4) - Terzaghi, K., 1943
"Theoretical Soil Mechanics."
John Wiley and Sons, Inc., P. 484-490

(5) - Wood, M., 1959
"Discussion of Paper by Skempton and Bjerrum, 1957."
Geotechnique, Vol. IX, p.

TABLE I

Results of Settlement Calculations

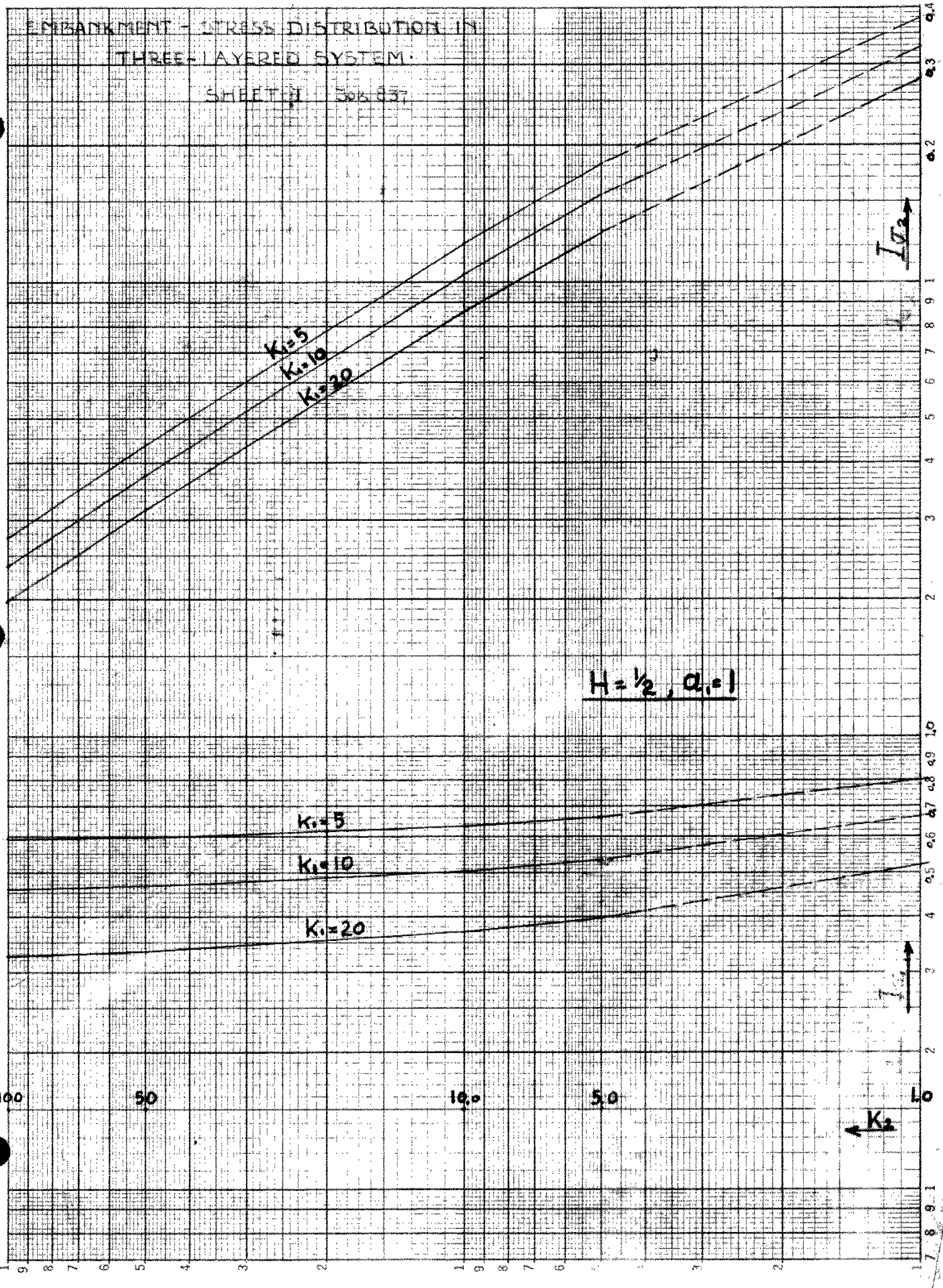
Settlement Point	Embankment Load			Net Pier Load		
	ρ_i	ρ_c	ρ_u	ρ_i'	ρ_c	ρ_u
Q3	0.06	0.40	0.46			
Q4	0.09	0.56	0.65			
Q2	0.13	0.56	0.69			
P4	0.06	0.44	0.50			
P5	0.20	0.80	1.00			
P2	0.24	0.91	1.15			
P6	0.23	0.90	1.13			
P3	0.16	0.70	0.86			
P'_4	0.12	0.55	0.67			
P'_5	0.29	0.96	1.23			
P'_2	0.33	1.07	1.40			
P'_6	0.32	1.08	1.40			
P'_3	0.20	0.77	0.97			
σ_1	0.22	0.89	1.11			
σ_2	0.42	1.32	1.74			
σ	0.45	1.39	1.84			
C				0.03	0.19	0.22
D				0.02	0.09	0.11
E				0.03	0.19	0.22
F				0.02	0.09	0.11

NOTE: Settlements measured in feet.

CALCULATION SHEETS

EMBANKMENT - STRESS DISTRIBUTION IN THREE-LAYERED SYSTEM.

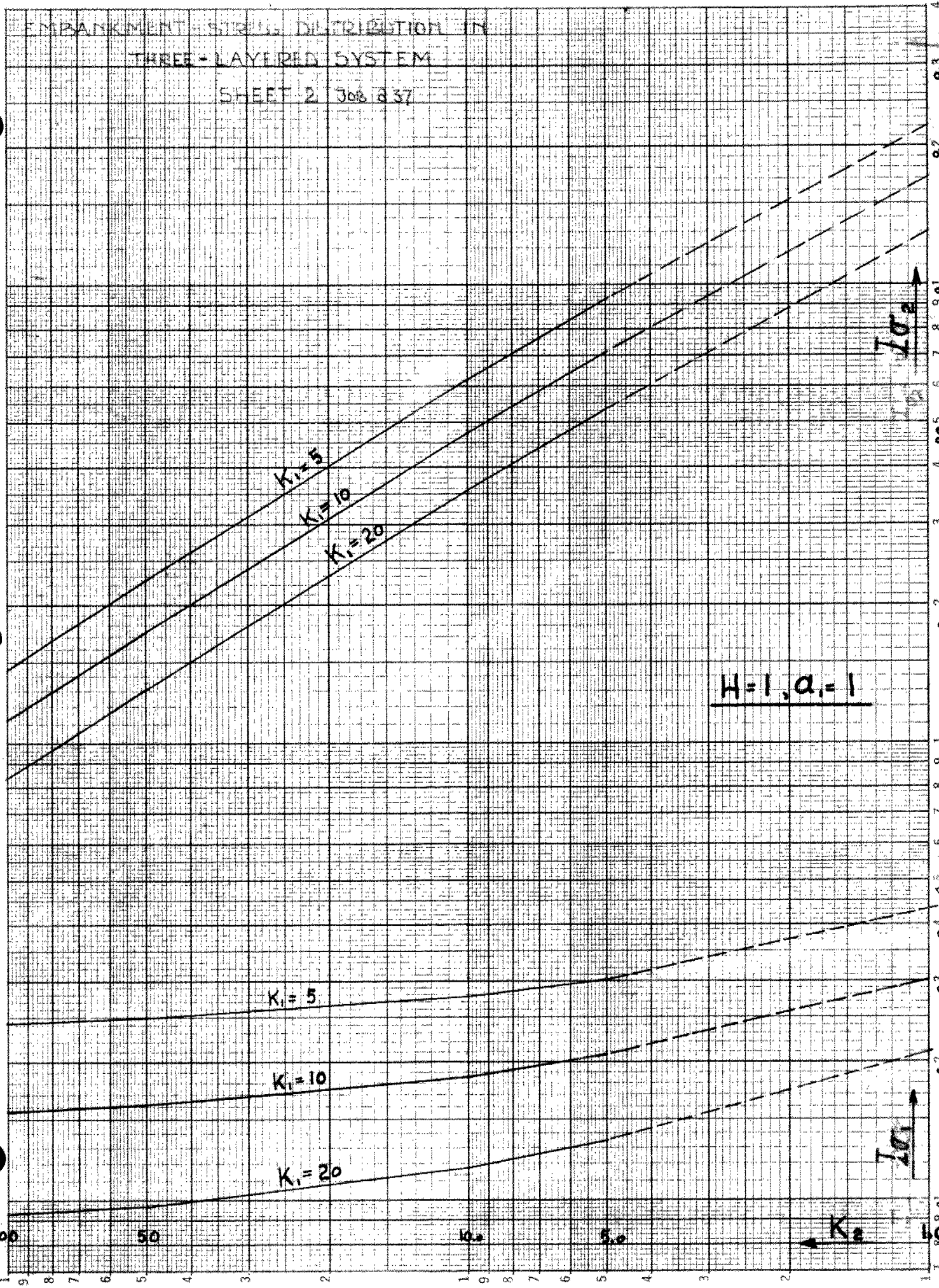
SHEET II SOIL 83T



G 9 - 111 G
Logarithmic, 2 1/2 X 2 Cycles.
MADE IN CANADA

MICROGRAPH

EMBANKMENT STRESS DISTRIBUTION IN
THREE-LAYERED SYSTEM
SHEET 2 Job 837



G 9-111 G
Logarithmic, 2½ X 2 Cycles.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

JOB 837 FILE
SHEET 3 OF
BY DATE
APP DATE

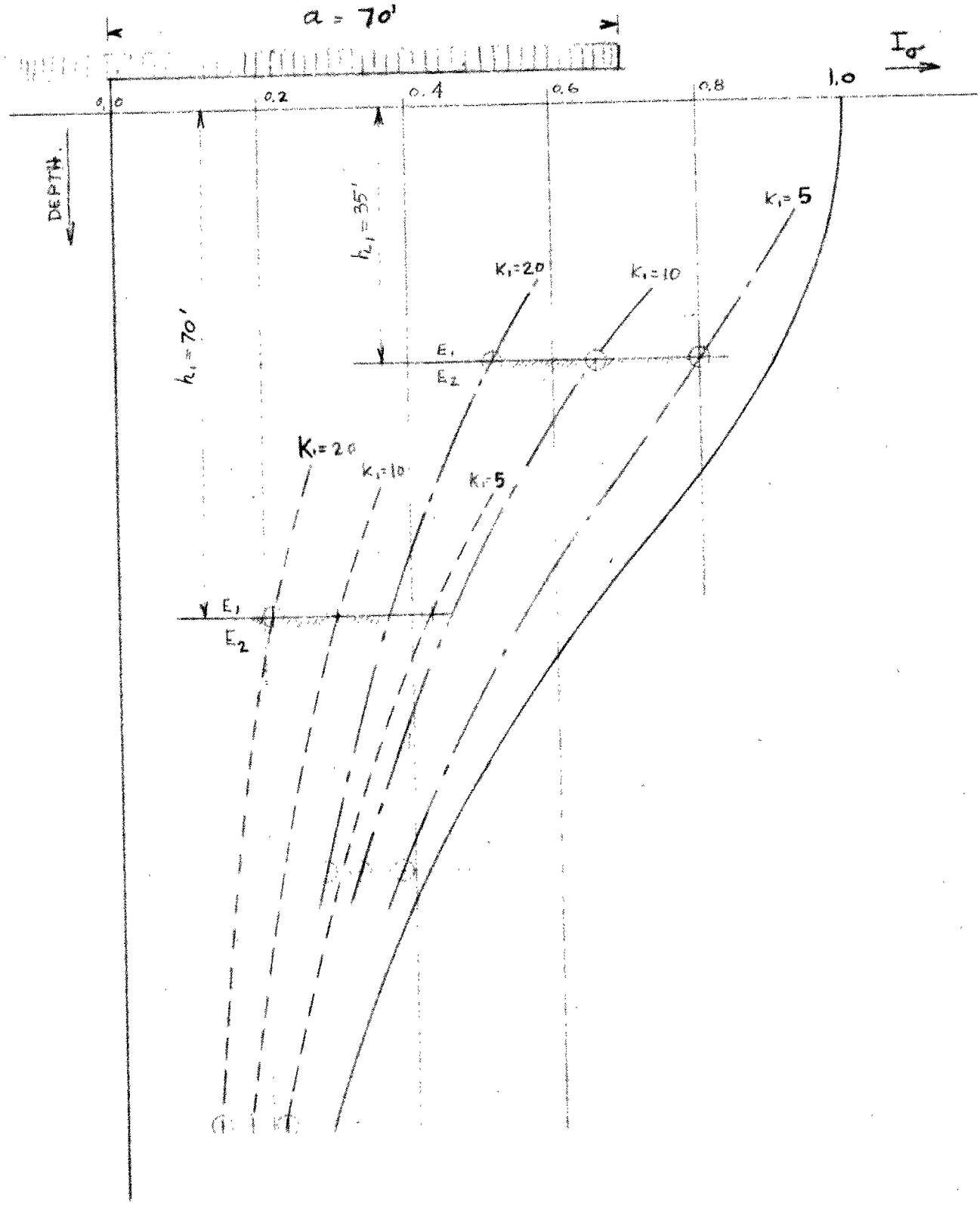
SUBJECT: EMBANKMENT - STRESS DISTRIBUTION IN THREE - LAYERED SYSTEM

INFLUENCE FACTORS FOR THE VERT. STRESS UNDER THE CENTRE OF A CIRCULAR LOADED AREA.

——— BOUSSINESQ. ($K_1 = 1$, $K_2 = 1$)

— — — — ACUM AND FOX $H = 1$, $a_1 = 1$, $K_2 = 1$

——— - ——— ACUM AND FOX $H = \frac{1}{2}$, $a_1 = 1$, $K_2 = 1$



FORM NO. 152 5M R.P.

ELASTIC SETTLEMENTS. USE "D.M. ISUR MISTER" CHART No 7 ($\mu = 0.4$)
SEE STRESS CALCULATION FOR "AREA COMBINATION" Pg 10

POINT σ $\quad \frac{b}{H} = \frac{71}{118} = 0.6 \quad \frac{b}{a} = 0.1 \qquad I_z = 4 \times 0.1475 = 0.59$

$H = 118' \quad E = 200 \text{ tons/ft}^2, \; q = 1.3 \qquad \delta_{\sigma} i = \frac{118}{200} 1.3 \times 0.59$

$= 0.77 \times 0.59 = \mathbf{0.45'}$

$\sigma_1 \quad \frac{b}{H} = 1.2 \qquad I_z = 2 \times 0.143 = .286$

$\delta_{\sigma_1} i = 1.77 \times 0.286 = \mathbf{0.22'}$

$\sigma_2 \quad \frac{b}{H} = 0.3 \qquad I_z = 2 \times .1293 = .2466$

$\frac{b}{H} = 0.9 \qquad I_z = 2 \times 0.147 = .2940$

0.5406

$\delta_{\sigma_2} i = 0.77 \times 0.5406 = \mathbf{0.42'}$

$P_2 \quad (\sigma - D_1 - D_2) \qquad I_z : \quad \sigma = 0.59$

$D_1 : \frac{b}{a} = 0.666 \quad \frac{b}{H} = 0.425 \quad : \quad -0.139$

$D_2 : \quad 0.9 \quad 0.425 \quad : \quad -0.143$

$I_{z\,P_2} \quad 0.308$

$\delta_{P_2} i = 0.77 \times 0.308 = \mathbf{0.24'}$

$P_3 \quad (\sigma_1 - A_1 + E) \qquad \sigma_1 = 0.286$

$E : \frac{b}{a} = 1 \quad \frac{b}{H} = 0.425 \qquad 0.065$

0.351

$A_1 : \frac{b}{a} = 0.222 \quad \frac{b}{H} = 0.425 \qquad -0.140$

0.211

$\delta_{P_3} i = 0.77 \times 0.211 = \mathbf{0.16'}$

$P_4 : (P_3 - 2E) \qquad 0.211$

$-2E \quad : \quad 0.130$

0.081

$\delta_{P_4} i : 0.77 \times 0.081 = \mathbf{0.06'}$

FORM NO. 152 5M R.P

Q_2 $(\sigma - 4B)$

B $\frac{b}{a} = 0.25$ $\frac{b}{H} = 0.212$, $4B$: 4×0.107

σ	0.59
	− 0.428
	0.162

$\delta_{Q_2} i = 0.77 \times 0.162 = 0.125'$

Q_3 $(\sigma_1 - 2C)$

$C = \frac{b}{a} = 0.25$ $\frac{b}{H} = 0.212$ $2C$: 2×0.107

σ_1	0.286
	− 0.214
	0.072

$\delta_{Q_3} i = 0.77 \times 0.072 = 0.055'$

P_2' : $(P_2 + 2 \times F_2)$

F_2: $\frac{b}{a} = 0.1$, $\frac{b}{H} = 0.085$, $2F_2 = 2 \times 0.063$

P_2 :	0.308
	0.126
	0.434

$\delta_{P_2'} i = 0.77 \times 0.434 = 0.33'$

P_3' : $(P_3 + F_2 - L_2)$

F_2 $\frac{b}{a} = 0.1$, $\frac{b}{H} = 0.085$,

L_2 $\frac{b}{a} = 0.1$ $\frac{b}{H} = 0.085$ $\left(\frac{F_2}{2}\right)$

P_3 :	0.211
	0.063
	0.274
	− 0.019
	0.255

$\delta_{P_3'} i = 0.77 \times 0.255 = 0.20'$

P_4' $(P_4 \quad F_2 - L_2)$

P_4	0.081
F_2	0.063
L_2	0.019
	0.163

$\delta_{P_4'} i = 0.77 \times 0.163 = 0.12'$

P_5 $(O_2 - D_1 - D_2)$

I_5	0.541
D_1	− 0.14
D_2	− 0.144
	0.257

$\delta_{P_5 i} = 0.77 \times 0.257 = 0.198 \sim 0.20'$

P_6 $(O_2 - A_2 - E')$

I_6	0.541
A_2	− 0.143
E'	− 0.105
	.293

$\delta_{P_6 i} = 0.77 \times 0.293 = 0.225'$

P'_5 $(P_5 + F_1 + F_2)$

I_{P_5}	0.257
F_1	0.061
F_2	0.064
	0382

$\delta_{P'_5 i} = 0.77 \times 0.382 = 0.29'$

P'_6 $(P_6 + F_1 + F_2)$

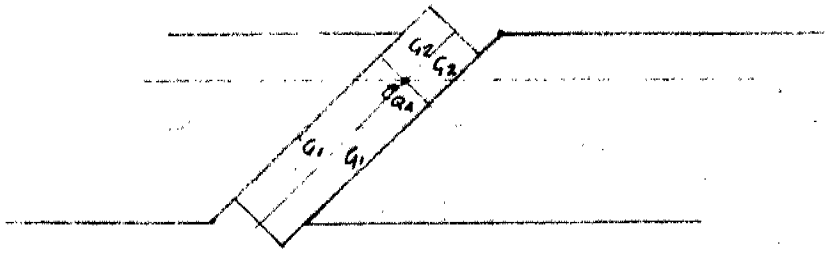
I_{P_6}	0.293
	0.061
	0.064
	0.418

$\delta_{P'_6 i} = 0.77 \times 0.415 = 0.32'$

FORM NO. 152 5M 8-5

SUBJECT:

EMBANKMENT ELASTIC SETTLEMENTS



POINT Q_A: $(\sigma_2 - 2G_1 - 2G_2)$ I_{σ_2} : 0.5406

G_1 $a = 150$, $b = 25$, $\frac{b}{a} = 0.17$, $\frac{b}{H} = 0.212$:

$2G_1 = 2 \times 0.11 = -0.22$

G_2 $a = 50$ $b = 25$ $\frac{b}{a} = 0.5$, $\frac{b}{H} = 0.212$

$2G_2 = 2 \times 0.10 = -0.20$

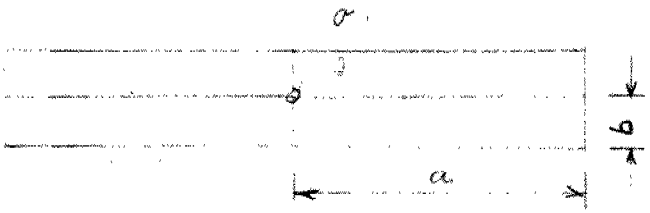
0.1206.

$\delta_{Q_A} = 0.77 \times 0.1206 = 0.093$

FORM NO. 152 5M R.P.

STRESS DISTRIBUTION UNDER POINTS IN THE INTERIOR SECTION OF A STRIP LOAD

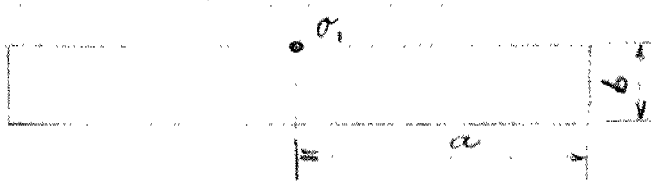
POINT σ AT ℄.



$\frac{b}{a} = 0.1 \sim 0$, $\frac{b}{H} = \frac{71}{118} = 0.6$

DEPTH RATIO $\frac{z}{H}$:	0.2	0.4	0.6	0.8	1.0
$\frac{1}{4} I_1$	0.251	0.243	0.225	0.21	0.186
I_1	1.004	0.972	0.900	0.840	0.744
$\Delta\sigma_z = I.q = I_1 \times 1.3$ kips/SF.	1.3	1.26	1.17	1.09	0.97

POINT σ_1 AT EDGE.



$\frac{b}{a} = 0.1 \sim 0$, $\frac{b}{H} = \frac{142}{118} = 1.2$

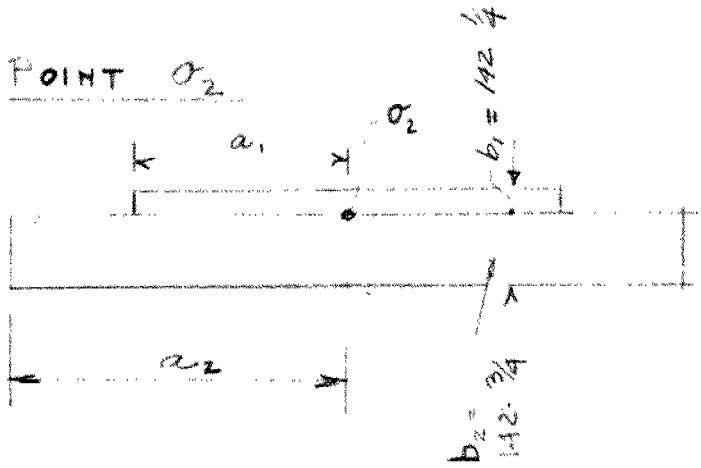
$\frac{z}{H}$	0.2	0.4	0.6	0.8	1.0
$\frac{1}{2} I_1$	0.2505	0.255	0.2515	0.244	0.24
I_1	0.505	0.510	0.503	0.488	0.48
$\Delta\sigma_z = I_1 q = I_1 \times 1.3$ K/SF	0.656	0.663	0.652	0.635	0.625

FORM NO. 152 5M R.P.

SUBJECT:

EMBANKMENT STRESS DISTRIBUTION

POINT σ_2



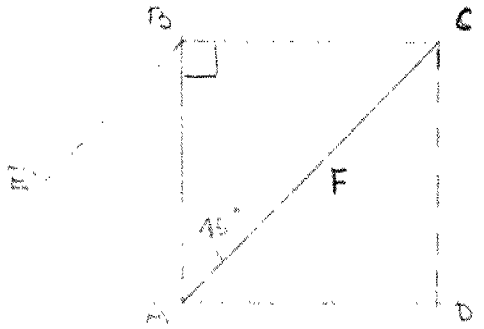
A) $\frac{a_1}{b_1} = 0.1$; $\frac{b_1}{H} = \frac{142}{118 \times 4} = 0.3$

B) $\frac{a_2}{b_2} = 0.1$; $\frac{b_2}{H} = \frac{142 \cdot 3}{118 \cdot 4} = 0.9$

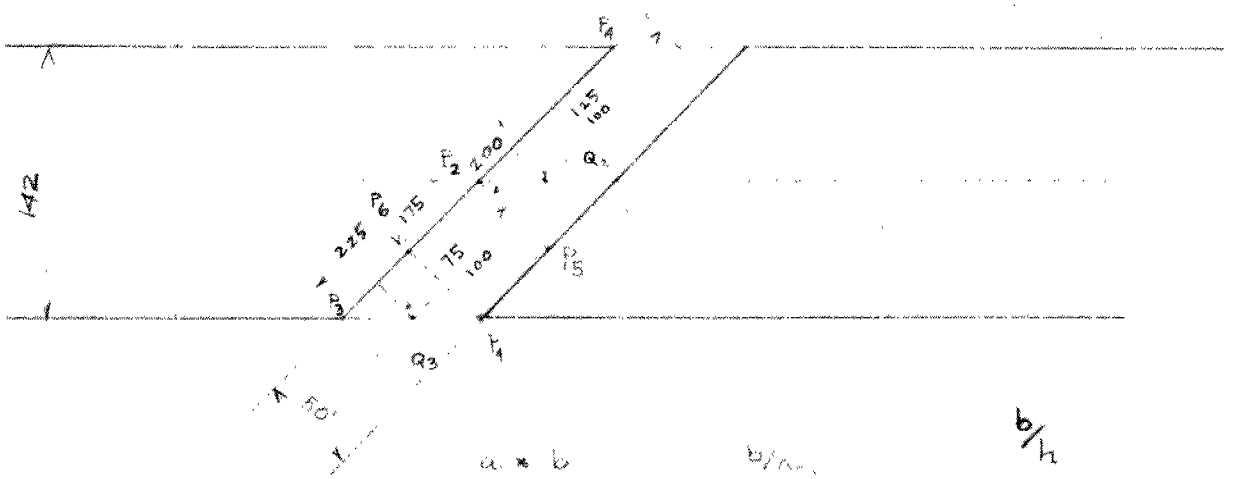
		$\frac{z}{H}$	0.2	0.4	0.6	0.8	1.0
AREAS "A"	$\frac{b}{H} = 0.3$	$\frac{I_{1A}}{2}$	0.232	0.186	0.152	0.130	0.113
AREAS "B"	$\frac{b}{H} = 0.9$	$\frac{I_{1B}}{2}$	0.253	0.254	0.248	0.238	0.229
		$\frac{1}{2}(I_{1A} + I_{1B})$	0.485	0.440	0.400	0.368	0.342
		$I_1 = I_{1A} + I_{1B}$	0.970	0.880	0.800	0.736	0.684
		$\Delta\sigma_z = I_1 \times q = I_1 \times 1.3$ k/sf	1.26	1.14	1.04	0.957	0.888

FORM NO. 152 5M R.P.

STRESS DISTRIBUTION UNDER POINTS AT THE BENDING LOCATION.



FOR THE LOADED AREA BEING THE RIGHT-ANGLED, ISOSCELES TRIANGLE ABC IT IS READILY SEEN THAT THE STRESSES UNDER CORNERS A & C ARE HALF OF THE STRESSES PRODUCED BY THE SQUARE ABCD, AND THAT THE STRESSES UNDER B IS EQUAL TO THE STRESSES PRODUCED BY THE SQUARE AEBF.



	a × b	b/a	b/h
COMPONENTS.	A_1 = 225 × 50	0.222	0.425
	A_2 = 175 × 50	0.285	0.425
	B = 100 × 25	0.25	0.212
	C = 200 × 25	0.125	0.212
	D_1 = 15 × 50	0.066	0.425
	D_2 = 125 × 50	0.4	0.425
△	E = 50 × 50	1.000	0.425

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SUBJECT: EMBANKMENT - STRESS DISTRIBUTION

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	b/a	b/h	0.2 h	0.4 h	0.6 h	0.8 h	h
A_1	0.222	0.425	0.245	0.222	0.192	0.170	0.148
A_2	0.285	0.425	0.245	0.222	0.192	0.170	0.148
B	0.250	0.212	0.211	0.146	0.112	0.092	0.072
C	0.125	0.212	0.212	0.148	0.116	0.096	0.081
D_1	0.666	0.425	0.243	0.215	0.177	0.148	0.120
D_2	0.400	0.425	0.245	0.221	0.190	0.166	0.140
E	1.000	0.425	(0.238)	(0.194)	(0.146)	(0.118)	(0.094)
			0.119	0.097	0.073	0.059	0.047

P_3 $(\sigma_1 - A_1 + E)$	σ_1	0.505	0.510	0.503	0.488	0.480
	E	0.119	0.097	0.073	0.059	0.047
		0.624	0.607	0.576	0.547	0.527
	A_1	−0.245	−0.222	−0.192	−0.170	−0.148
	I_1	0.379	0.385	0.384	0.377	0.379
	$\Delta\sigma = I_1 \times 1.3$ K/SF	0.492	0.500	0.500	0.490	0.492

P_4 $(\sigma_1 - A_2 - E,\ A_2 = A_1 \rightarrow P_3 - 2E)$		0.379	0.385	0.384	0.377	0.379
	−2E	−0.238	−0.194	−0.146	−0.118	−0.094
		0.141	0.191	0.238	0.259	0.285
	$\Delta\sigma = I_1 \times 1.3$ K/SF	0.183	0.248	0.310	0.337	0.370

Q_3 $(\sigma_1 - 2c)$	σ_1	0.505	0.510	0.503	0.488	0.480
	2c	−0.424	−0.296	−0.232	−0.184	−0.162
		0.081	0.214	0.271	0.304	0.328
	$\Delta\sigma = I_1 \times 1.3$ K/SF	0.105	0.278	0.352	0.395	0.427

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SUBJECT:

EMBANKMENT - STRESS DISTRIBUTION

		0.2h	0.4h	0.6h	0.8h	h
P_2 $(\sigma - D_1 - D_2)$	σ:	1.004	0.972	0.900	0.840	0.744
	D_1	−0.243	−0.215	−0.177	−0.148	−0.120
	D_2	−0.245	−0.221	−0.190	−0.166	−0.140
		0.516	0.536	0.533	0.526	0.484
	$\Delta\sigma = I_1 \times 1.3$ K/SF	0.67	0.697	[illegible]	0.685	[illegible]
Q_2 $\sigma - 4B$	σ	1.004	0.972	0.900	0.840	0.744
	4B	−0.844	−0.584	−0.448	−0.368	−0.288
		0.160	0.388	0.452	0.472	0.456
	$\Delta\sigma = I \times 1.3$ K/SF	0.208	0.505	0.593	0.613	0.594
P_5 $(\sigma_2 - D_1 - D_2)$	σ_2	0.970	0.880	0.800	0.736	0.684
	D_1	−0.243	−0.215	−0.177	−0.148	−0.120
	D_2	−0.245	−0.221	−0.190	−0.166	−0.140
		0.482	0.444	0.433	0.422	0.424
	$\Delta\sigma$ $I_1 \times 1.3$ K/SF	0.626	0.577	0.564	0.550	0.550
P_6 $(\sigma_2 - A_2 - E')$	σ_2	0.970	0.880	0.800	0.736	0.684
	A_2	−0.245	−0.222	−0.192	−0.170	−0.148
E': 35 in, $\frac{6}{2} \cdot 1 \cdot \frac{b}{h} = \frac{35}{118} = 0.212$	E'	−0.184	−0.093	−0.055	−0.040	−0.030
		0.541	0.565	0.553	0.526	0.496
	$\Delta\sigma = I_1 \times 1.3$ K/SF	0.704	0.735	0.720	[illegible]	0.645

FORM NO. 152 5M R.P.

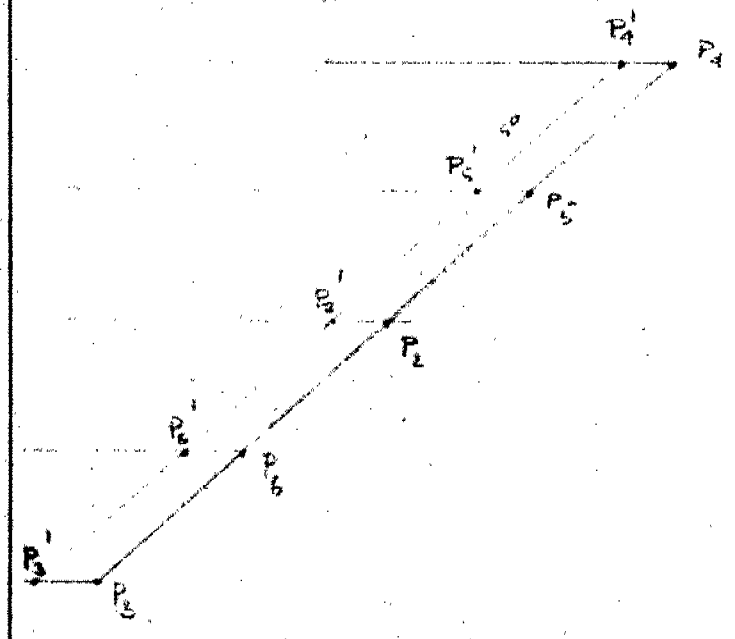
SUBJECT:

EMBANKMENT - STRESS DISTRIBUTION

STRESS DISTRIBUTION 10' FROM LINE $P_3 - P_4$

FOR THIS CASE A NEW CALCULATION SHOULD BE MADE WITH A 60' WIDE UNLOADED STRIP AND A 10' WIDE STRIP SHOULD THEN BE ADDED.

HOWEVER, THE STRESSES FOR A 60' STRIP ARE ONLY SLIGHTLY DIFFERENT FROM THOSE FOR A 50' STRIP UNLOADED (ABT. 5% LESS) FOR WHICH REASON THE CALCULATION CAN BE BASED ON THE RESULTS ALREADY OBTAINED FOR THE CASE ABOVE.



LOADING COMPONENTS.

				0.2 H	0.4 H	0.6 H	0.8 H	1.0 H
F_1 50 × 10	$\frac{b}{a} = 0.2$	$\frac{b}{H} = \frac{10}{118} = 0.085$		0.121	0.060	0.038	0.028	0.02
F_2 100 × 10	$\frac{b}{a} = 0.1$	$\frac{b}{H} = 0.085$		0.121	0.066	0.047	0.038	0.030
E_2 10 × 10	$\frac{b}{a} = 1.0$	$\frac{b}{H} = 0.085$	$\frac{T_1}{2}$	0.034	0.021	0.005	0.004	0.002
ADD P_4 :	$F_2 + E_2$			0.155	0.087	0.052	0.042	0.032
— — P_3	$F_2 - E_2$			0.087	0.045	0.042	0.034	0.028
— — P_2	$2 \times F_2$			0.242	0.132	0.094	0.076	0.060
— — $P_5 + P_6$	$F_2 + F_1$			0.242	0.126	0.085	0.066	0.052

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H. G. ACRES & COMPANY LIMITED — CALCULATIONS

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SUBJECT:

EMBANKMENT - STRESS DISTRIBUTION

STRESSES UNDER P_1', P_5'

			0.2	0.4	0.6	0.8	1.0
P_1'	P_1	:	0.141	0.191	0.238	0.259	0.285
			0.155	0.087	0.052	0.042	0.032
			0.296	0.278	0.290	0.301	0.317
	$\Delta\sigma$	K/SF	0.385	0.361	0.364	0.391	0.412
P_5'	P_5	:	0.482	0.444	0.433	0.422	0.424
			0.242	0.126	0.085	0.065	0.052
			0.724	0.570	0.518	0.488	0.476
	$\Delta\sigma$	K/SF	0.940	0.741	0.675	0.635	0.620
P_2'	P_2	:	0.516	0.536	0.533	0.524	0.484
			0.242	0.132	0.094	0.076	0.060
			0.758	0.668	0.627	0.600	0.544
	$\Delta\sigma$	K/SF	0.985	0.869	0.815	0.783	0.706
P_6'	P_6	:	0.541	0.565	0.554	0.526	0.493
			0.242	0.126	0.080	0.066	0.052
			0.783	0.691	0.634	0.592	0.545
	$\Delta\sigma$	K/SF	1.020	0.900	0.830	0.770	0.713
P_3'	P_3	:	0.371	0.385	0.354	0.337	0.279
			0.087	0.145	0.072	0.074	0.128
			0.466	0.430	0.426	0.411	0.407
	$\Delta\sigma$	K/SF	0.606	0.559	0.554	0.525	0.528

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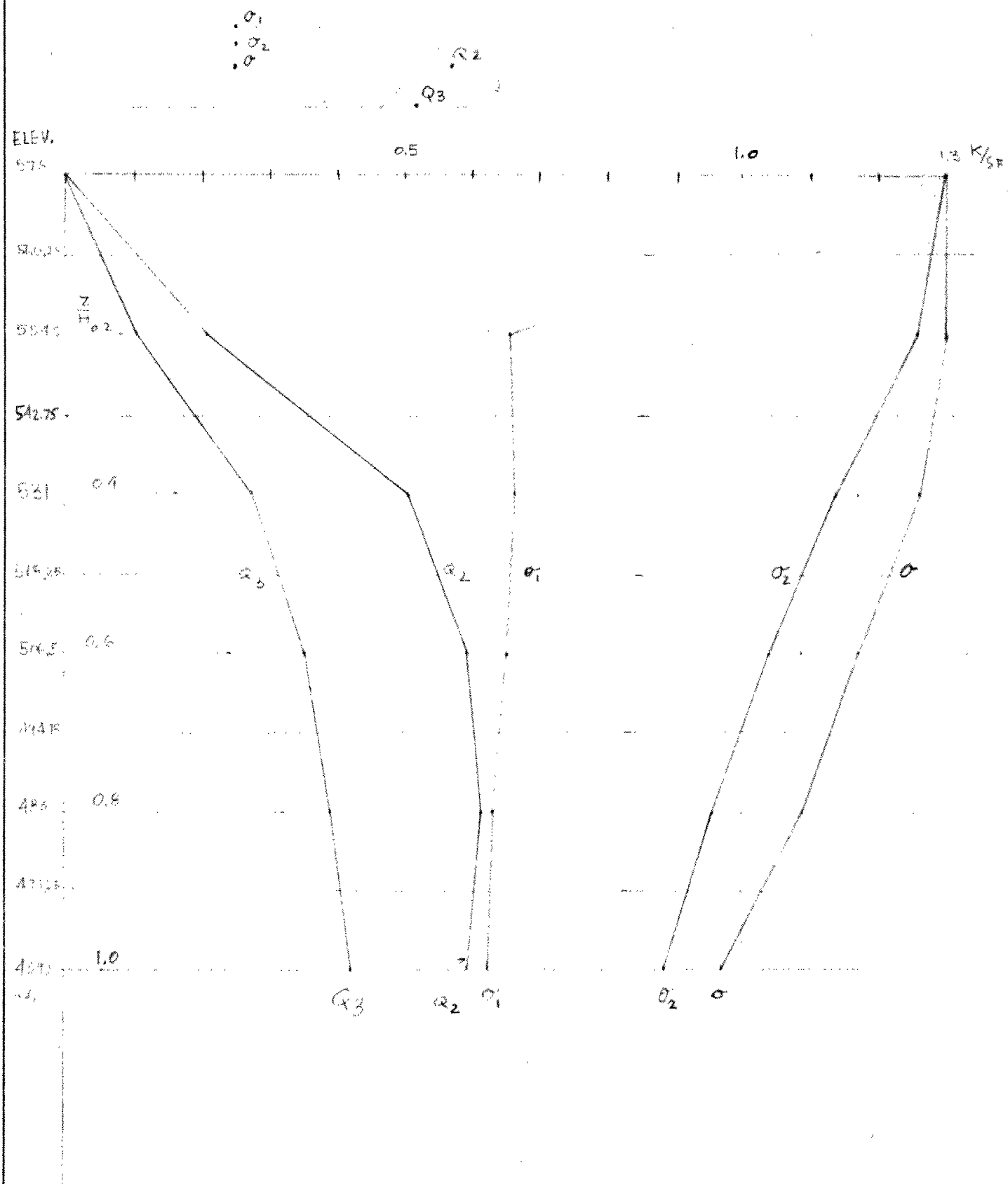
SUBJECT:

EMBANKMENT - STRESS DISTRIBUTION



FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT:

EMBANKMENT - STRESS DISTRIBUTION

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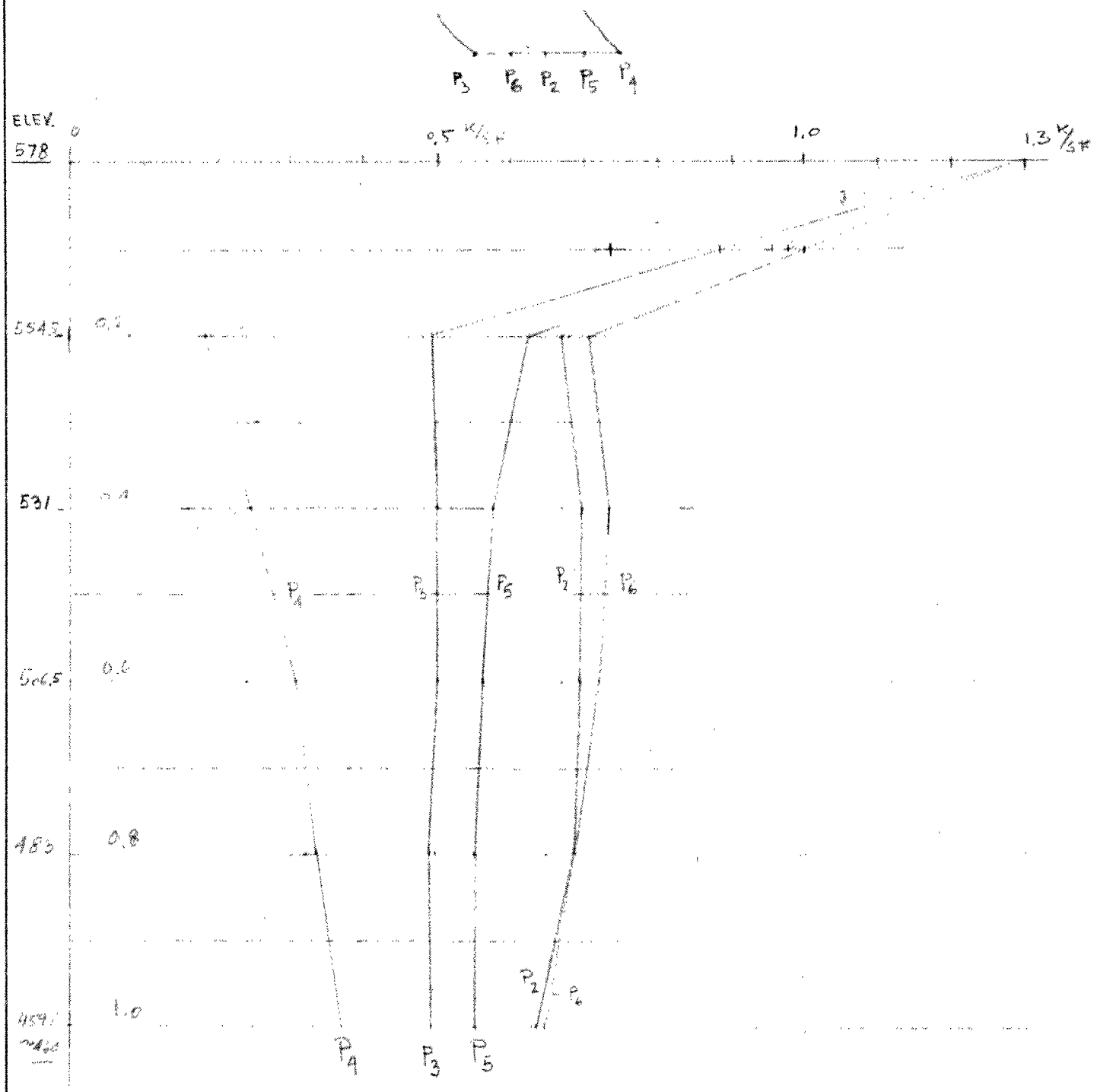

FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT: EMBANKMENT - STRESS DISTRIBUTION IN HOMOGENEOUS FOUNDATION

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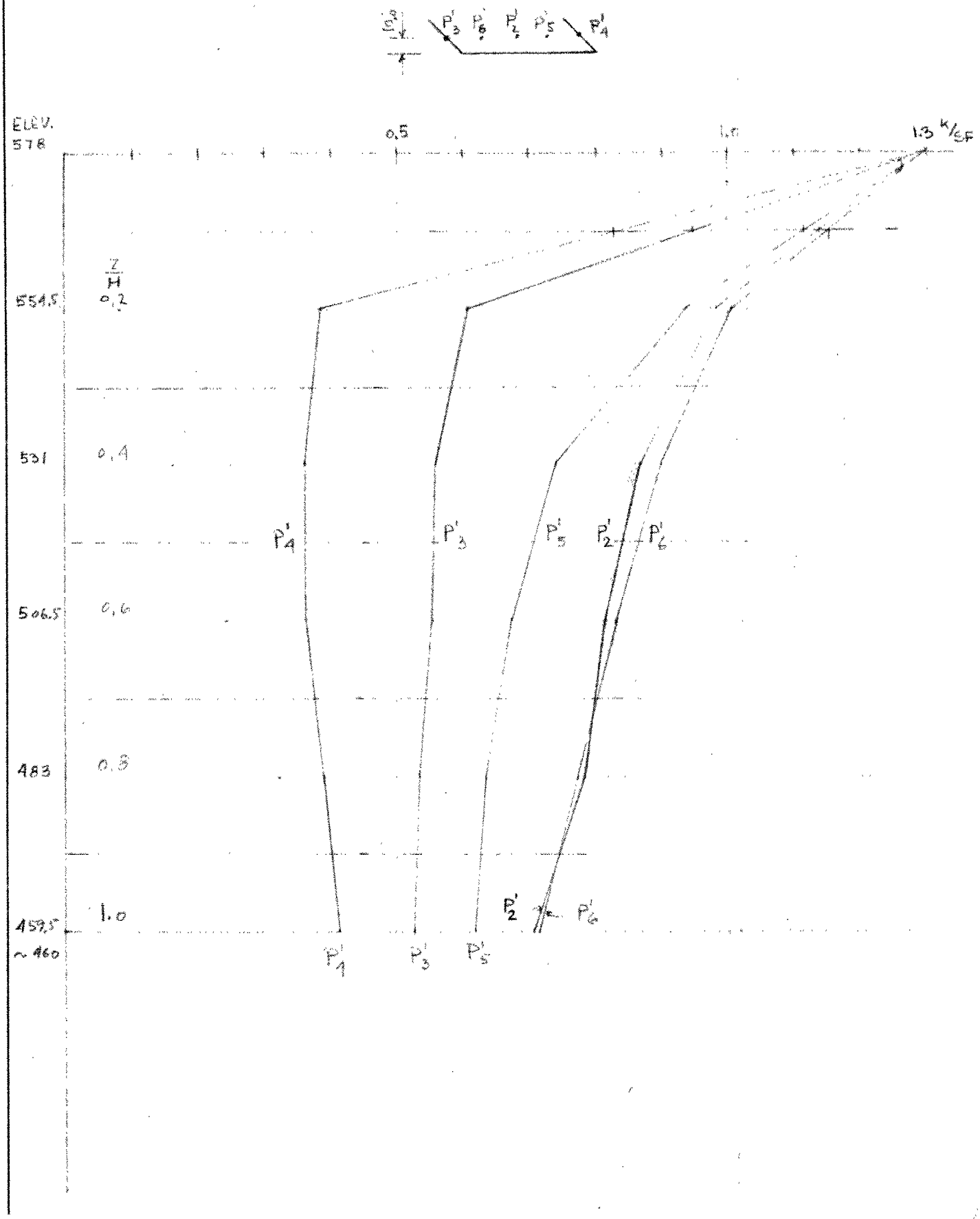

FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

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SHEET 18 OF
BY DATE
APP DATE

SUBJECT:

EMBANKMENT - CONSOLIDATION SETTLEMENTS

POINT	LAYER	P_o t/sf	$\Delta\sigma$ t/sf	$P_o+\Delta\sigma$	e_o	Δe	$\frac{\Delta e}{1+e_o}\times 100$	$\Delta h \times \gamma$	$\delta_c = \gamma \sum \frac{\Delta e}{1+e_o}\Delta h$
σ	1	0.7	0.65	1.35	0.61	0.015	0.93	23.5×0.7	$\frac{0.93}{100}\times 23.5\times 0.7 = 0.15$
	2	1.42	0.64	2.06	0.72	0.037	2.15	23.5×0.9	
	3	2.16	0.61	2.77	0.765	0.030	1.70	23.5×0.9	
	4	2.98	0.57	3.55	0.725	0.019	1.10	23.5×0.9	$\frac{5.89}{100}\times 23.5\times 0.9 = 1.24$
	5	3.70	0.52	4.22	0.700	0.016	0.99	23.5×0.9	$\delta_{\sigma c} = 1.39'$
							5.89		
σ_2	1	0.7	0.64	1.34	0.61	0.015	0.93	23.5×0.7	$\frac{0.93}{100}\times 23.5\times 0.7 = 0.15'$
	2	1.42	0.60	2.02	0.72	0.035	2.04	23.5×0.9	
	3	2.16	0.55	2.71	0.765	0.026	1.59	23.5×0.9	$\frac{5.55}{100}\times 23.5\times 0.9 = 1.17$
	4	2.98	0.50	3.48	0.725	0.018	1.04	23.5×0.9	$\delta_{\sigma_2 c} = 1.32$
	5	3.70	0.46	4.16	0.700	0.015	0.88	23.5×0.9	
							5.55		
σ_1	1	0.7	0.49	1.19	0.61	0.012	0.75	23.5×0.7	$\frac{0.75}{100}\times 23.5\times 0.7 = 0.12$
	2	1.42	0.33	1.75	0.72	0.023	1.34	23.5×0.9	
	3	2.16	0.33	2.49	0.765	0.019	1.08		
	4	2.98	0.32	3.30	0.725	0.011	0.64		$\frac{3.65}{100}\times 23.5\times 0.9 = 0.77$
	5	3.70	0.32	4.02	0.700	0.010	0.59		$\delta_{\sigma_1} = 0.89'$
							3.65		
Q_2	1	0.7	0.05	75	0.61	~ 0			
	2	1.42	0.18	1.60	0.72	0.012	0.70		
	3	2.16	0.28	2.44	0.765	0.014	0.82		
	4	2.98	0.30	3.28	0.725	0.012	0.68	23.5×0.9	$\delta_{Q_2 c} = \frac{2.67}{100}\times 23.5\times 0.9 = 0.56'$
	5	3.70	0.30	4.00	0.700	0.010	0.59		
							2.67		
Q_3	1	0.7	0.03	0.73	0.61	~ 0			
	2	1.42	0.10	1.52	0.72	0.008	0.46		
	3	2.16	0.16	2.32	0.765	0.010	0.57		$\delta_{Q_3 c} = \frac{1.90}{100}\times 23.5\times 0.9 = 0.40'$
	4	2.98	[illegible]	3.17	0.725	[illegible]	0.46	23.5×0.9	
	5	3.70	[illegible]	3.90	0.700	0.007	0.41		
							1.90		

FORM NO. 152 5M R.F

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

JOB 837 FILE
SHEET 19 OF
BY DATE
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SUBJECT:

EMBANKMENT- CONSOLIDATION SETTLEMENTS

POINT	LAYER	P_o t/sq	$\Delta\sigma$ t/sq	$P_o+\Delta\sigma$	e_o	Δe	$\frac{\Delta e}{1+e_o}\times 100$	ΔH ft	$S_c = \frac{\Delta e}{1+e_o}\times\Delta H$
P_4	1	0.7	0.37	1.07	0.61	0.010	0.62	23.5×0.7	$\frac{0.62}{100}\times 23.5\times 0.7 = 0.10$
	2	1.42	0.11	1.53	0.72	0.007	0.52	23.5×0.9	
	3	2.16	0.14	2.30	0.765	0.0175	0.42	—	$\frac{1.61}{100}\times 23.5\times 0.9 = 0.34$
	4	2.98	0.16	3.14	0.700	0.0055	0.35		
	5	3.70	0.18	3.88	0.700	0.0055	0.32		$S_{P_4 C} = 2.44'$
							1.61		
P_5	1	0.7	0.48	1.18	0.61	0.012	0.75	23.5×0.7	$\frac{0.75}{100}\times 23.5\times 0.7 = 0.12$
	2	1.42	0.30	1.72	0.72	0.017	1.10	23.5×0.9	
	3	2.16	0.28	2.44	0.765	0.0155	0.85	—	$\frac{2.12}{100}\times 23.5\times 0.9$
	4	2.98	0.28	3.26	0.700	0.0105	0.61	—	$S_{P_5 C} = 0.78'$
	5	3.70	0.28	3.98	0.700	0.009	0.53	—	
							3.12		
P_2	1	0.7	0.49	1.19	0.61	0.012	0.75	23.5×0.7	$\frac{0.75}{100}\times 23.5\times 0.7 = 0.12$
	2	1.42	0.34	1.76	0.72	0.021	1.22	23.5×0.9	
	3	2.16	0.35	2.51	0.700	0.019	1.07	—	$\frac{2.72}{100}\times 23.5\times 0.9 = 0.79$
	4	2.98	0.35	3.33	0.700	0.014	0.81		
	5	3.70	0.33	4.03	0.700	0.0105	0.62	—	$S_{P_2 C} = 2.91'$
							3.72		
P_6	1	0.7	0.50	1.20	0.61	0.0125	0.78	23.5×0.7	$\frac{0.78}{100}\times 23.5\times 0.7 = 0.12$
	2	1.42	0.36	1.78	0.72	0.022	1.39	23.5×0.9	
	3	2.16	0.36	2.52	0.765	0.0195	1.10	—	
	4	2.98	0.35	3.33	0.700	0.014	0.81	—	$\frac{3.81}{100}\times 23.5\times 0.9 = 0.81$
	5	3.70	0.33	4.03	0.700	0.0105	0.62	—	$S_{P_6 C} = 2.73'$
							3.81		
P_3	1	0.7	0.44	1.14	0.61	0.0110	0.68	23.5×0.7	$\frac{0.68}{100}\times 23.5\times 0.7 = 0.11$
	2	1.42	0.25	1.67	0.72	0.016	0.93	23.5×0.9	
	3	2.16	0.25	2.41	0.765	0.014	0.86	—	
	4	2.98	0.25	2.23	0.700	0.010	0.58	—	$\frac{2.70}{100}\times 23.5\times 0.9 = 0.59$
	5	3.70	0.25	3.95	0.700	0.008	0.47	—	$S_{P_3 C} = 0.70'$
							2.78		

FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT: EMBANKMENT - CONSOLIDATION SETTLEMENTS

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POINT	LAYER	P_o t/SF	$\Delta\sigma$ t/SF	$P_o+\Delta\sigma$	e_o	Δe	$\frac{\Delta e}{1+e_o}\cdot 100$	$\Delta h \times \gamma$	$\delta_c = \gamma \sum \frac{\Delta e}{1+e_o}\Delta h$
P_4'	1	0.7	0.42	1.12	0.61	0.011	0.66	23.5×0.7	$\frac{0.66}{100}$ 23.5×0.7 = 0.11
	2	1.42	0.19	1.61	0.72	0.125	0.73	23.5×0.9	
	3	2.16	0.18	2.34	0.65	0.010	0.57		$\frac{2.06}{100}$ 23.5×0.9 = 0.44
	4	2.95	0.19	3.17	0.70	.001	0.41		
	5	3.70	0.20	3.90	0.70	0.006	0.35		$\delta_{P_4'c}$ = 0.55'
							2.06		
P_5'	1	0.7	0.56	1.26	0.61	0.0135	0.84	23.5×0.7	$\frac{0.84}{100}$ 23.5×0.7 = 0.14
	2	1.42	0.42	1.84	0.72	0.025	1.45	23.5×0.9	
	3	2.16	0.35	2.51	0.70	0.019	1.03		$\frac{3.84}{100}$ 23.5×0.9 = 0.81
	4	2.95	0.33	3.31	0.70	0.013	0.75		
	5	3.70	0.31	4.01	0.70	0.0095	0.56		$\delta_{P_5'c}$ = 0.95'
							3.84		
P_2'	1	0.7	0.57	1.27	0.610	0.0140	0.87	23.5×0.7	$\frac{0.87}{100}$ 23.5×0.7 = 0.14
	2	1.42	0.46	1.88	0.720	0.028	1.62	23.5×0.9	
	3	2.16	0.42	2.58	0.665	0.022	1.19		$\frac{4.41}{100}$ 23.5×0.9 = 0.93
	4	2.95	0.40	3.35	0.720	0.015	0.87		
	5	3.70	0.37	4.07	0.700	0.0115	0.68		$\delta_{P_2'c}$ = 1.07'
							4.41		
P_6'	1	0.7	0.58	1.28	0.61	0.014	0.87	23.5×0.7	$\frac{0.87}{100}$ 23.5×0.7 = 0.14
	2	1.42	0.49	1.91	0.70	0.0209	1.66	23.5×0.9	
	3	2.16	0.49	2.65	0.665	0.020	1.24		$\frac{4.45}{100}$ 23.5×0.9 = 0.94
	4	2.95	0.40	3.35	0.720	0.015	0.87		
	5	3.70	0.37	4.07	0.700	0.0115	0.68		$\delta_{P_6'c}$ = 1.18'
							4.45		
P_3'	1	0.7	0.48	1.18	0.61	0.012	0.74	23.5×0.7	$\frac{0.75}{100}$ 23.5×0.7 = 0.12
	2	1.42	0.29	1.71	0.720	0.019	1.10	23.5×0.9	
	3	2.16	0.28	2.44	0.665	0.0155	0.88		
	4	2.95	0.27	3.25	0.720	0.010	0.58		$\frac{3.09}{100}$ 23.5×0.9 = 0.65
	5	3.70	0.27	3.97	0.700	0.009	0.53		$\delta_{P_3'c}$ = 0.77'
							3.09		

FORM NO. 152 5M R.F.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT:

EMBANKMENT SETTLEMENT

JOB 837 FILE
SHEET 20 - A OF
BY DATE
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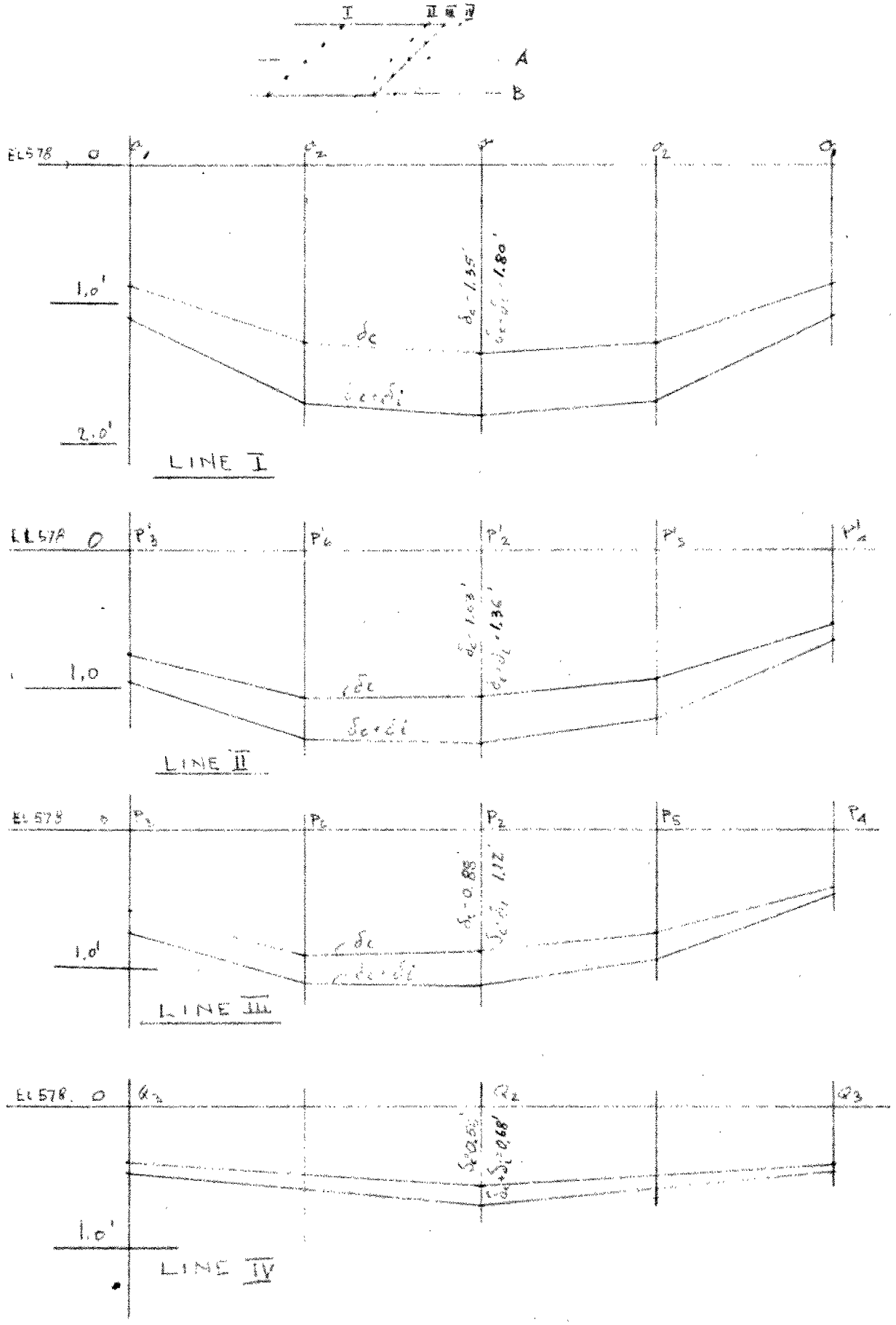


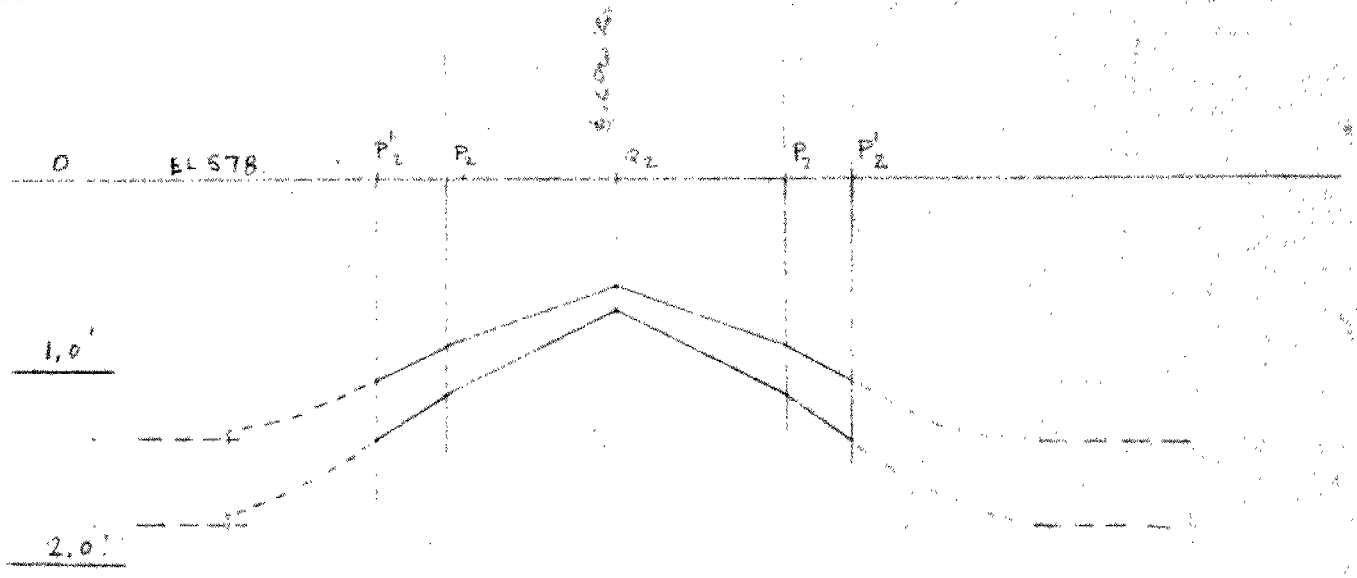
FORM NO. 182 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

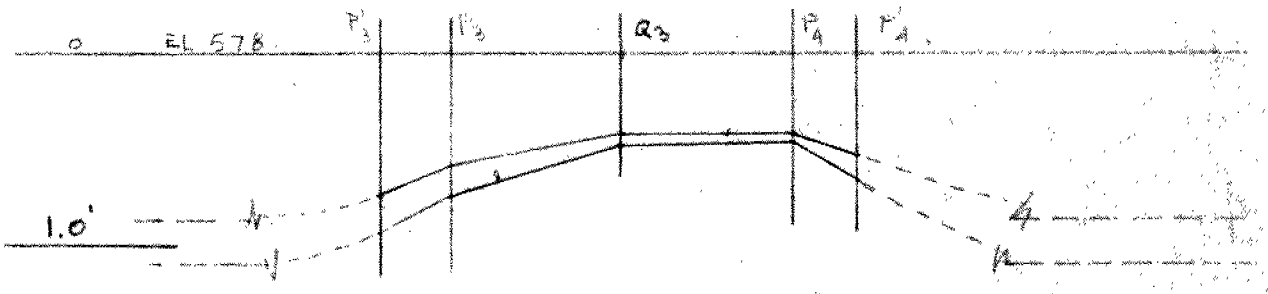
JOB 837 FILE
SHEET 20-B OF
BY DATE
APP DATE

SUBJECT:

EMBANKMENT SETTLEMENTS



LINE A



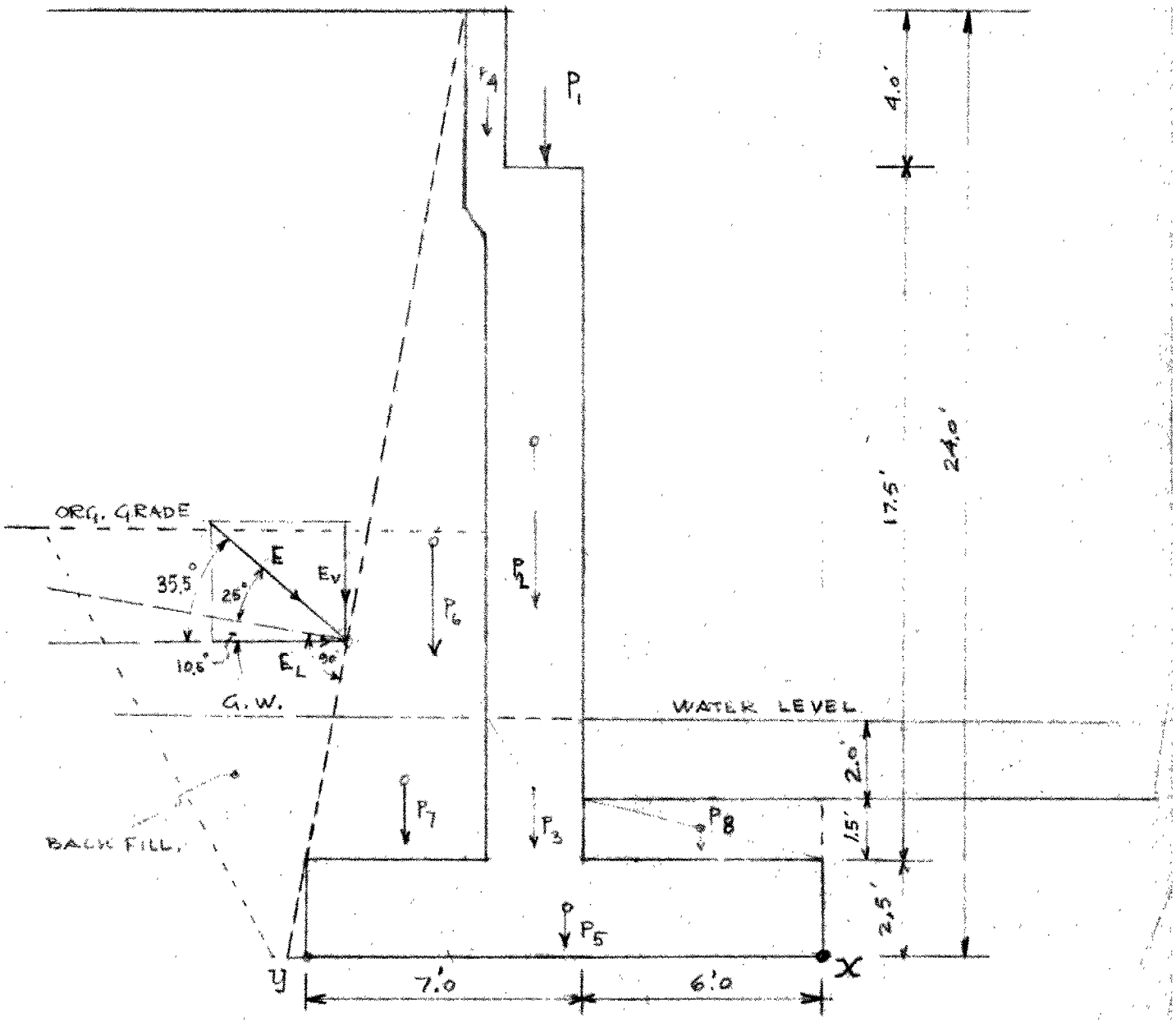
LINE B

FORM NO. 152 5M R.P.

SUBJECT:

BRIDGE PIERS - ASSUMED SECTION

ASSUMED CROSS SECTION
OF BRIDGE ABUTMENT WALL.
SCALE 1/4" = 1'-0



BACK FILL $\varphi = 32.5$ $K = 0.3$

$$E_L = \frac{1}{2}\gamma h^2 K = \frac{1}{2} \times 0.1 \times 24^2 \times 0.3 = 8.65 \text{ K/LF}$$

$$E_V = E_L \times tg\, 35.5^\circ = 8.65 \times tg\, 35.5 = 6.15 \text{ K/LF}$$

FORM NO. 152 5M R.P

DEAD LOAD RESULTANT.

		LOAD IN KIPS.	MOM. ARM.	MOM. AT X
P_1	(REACTION FROM BRIDGE DECK)	4.5	7.0'	31.5 'K
P_2	2.5 × 14.0 × 0.15	5.25	7.25'	38.0 "
P_3	2.5 × 3.5 × 0.09	0.80	7.25'	5.8 "
P_4	5 × 1.0 × 0.15	0.75	8.5'	6.4 "
P_5	2.5 × 13 × 0.09	2.90	6.5'	19.0 "
P_6	½ × 4 × 18 × 0.1	3.6	9.83	35.4
P_7	4.25 × 3.5 × 0.07	1.1	10.6	11.7
P_8	6 × 1.5 × 0.07	0.65	3.0	2.0
E_V		6.15	12.0	74.0
		25.70		223.8
E_L	8.65		- 8	69.2
		25.70		154.6

$$X = \frac{154.6}{25.7} = 6.0'$$

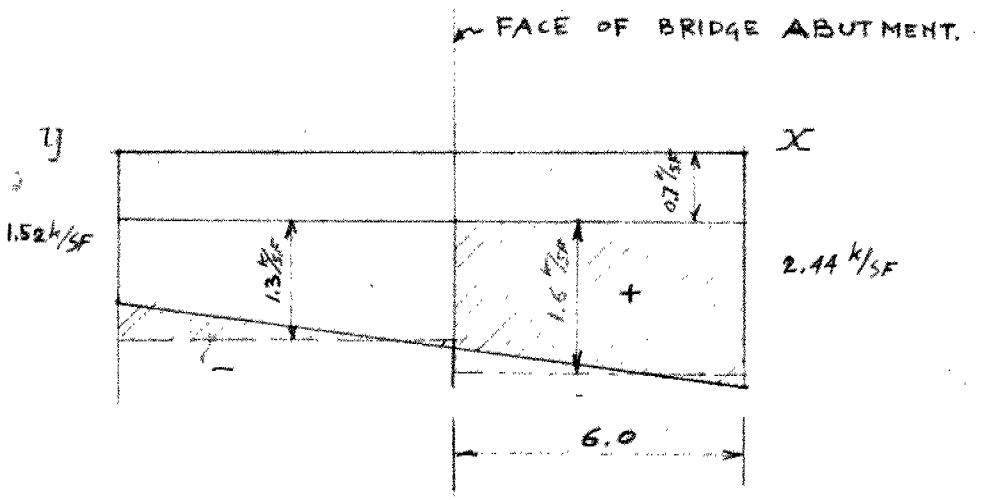
$$e = 6.0 - 6.0 = 0.5'$$

$$f = \frac{25.70}{13} \pm \frac{25.70 \times 0.5'}{\frac{1}{6} \times 13^2}$$

$$= 1.98 \pm 0.46 = \begin{matrix} 2.44 \\ 1.52 \end{matrix} \ \text{T/SQ.FT.}$$

SUBJECT

BRIDGE PIERS - CONTACT STRESSES



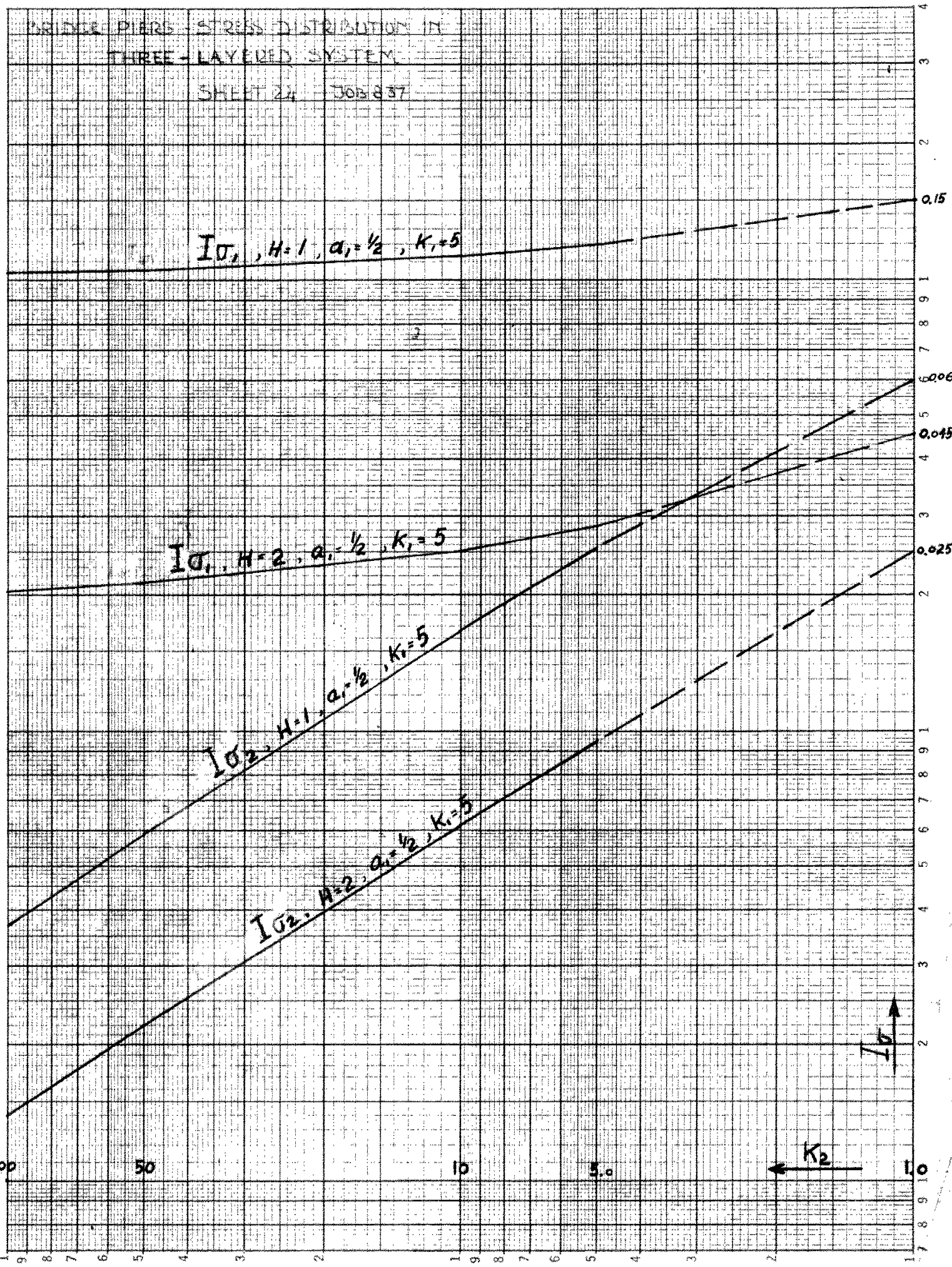
THE AVERAGE EXISTING PRESSURE IN THE AREA OF THE FOOTING IS ESTIMATED TO ABOUT 0.7 KIPS/S.F.

IN THE SETTLEMENT ANALYSIS FOR THE EMBANKMENT THE LIMIT OF THE LOADED AREA WAS ASSUMED TO BE AT THE FACE OF THE BRIDGE ABUTMENT.

THE STRESSES WHICH WILL PRODUCE ADDITIONAL SETTLEMENTS ARE SEEN TO BE IN THE TWO HATCHED AREAS ON THE STRESS DIAGRAM; A LARGE AREA AT X WITH POSITIVE STRESSES AND A SMALL AREA AT Y WITH NEGATIVE STRESSES. FOR THE PURPOSE OF CALCULATING THE ADDITIONAL SETTLEMENTS IT WILL BE ACCURATE ENOUGH TO SUBTITUTE THIS DISTRIBUTION OF ADDITIONAL STRESSES WITH A CONSTANT STRESS OF 1.6 K/S.F. OVER THE 6.0' OF WIDTH AT X

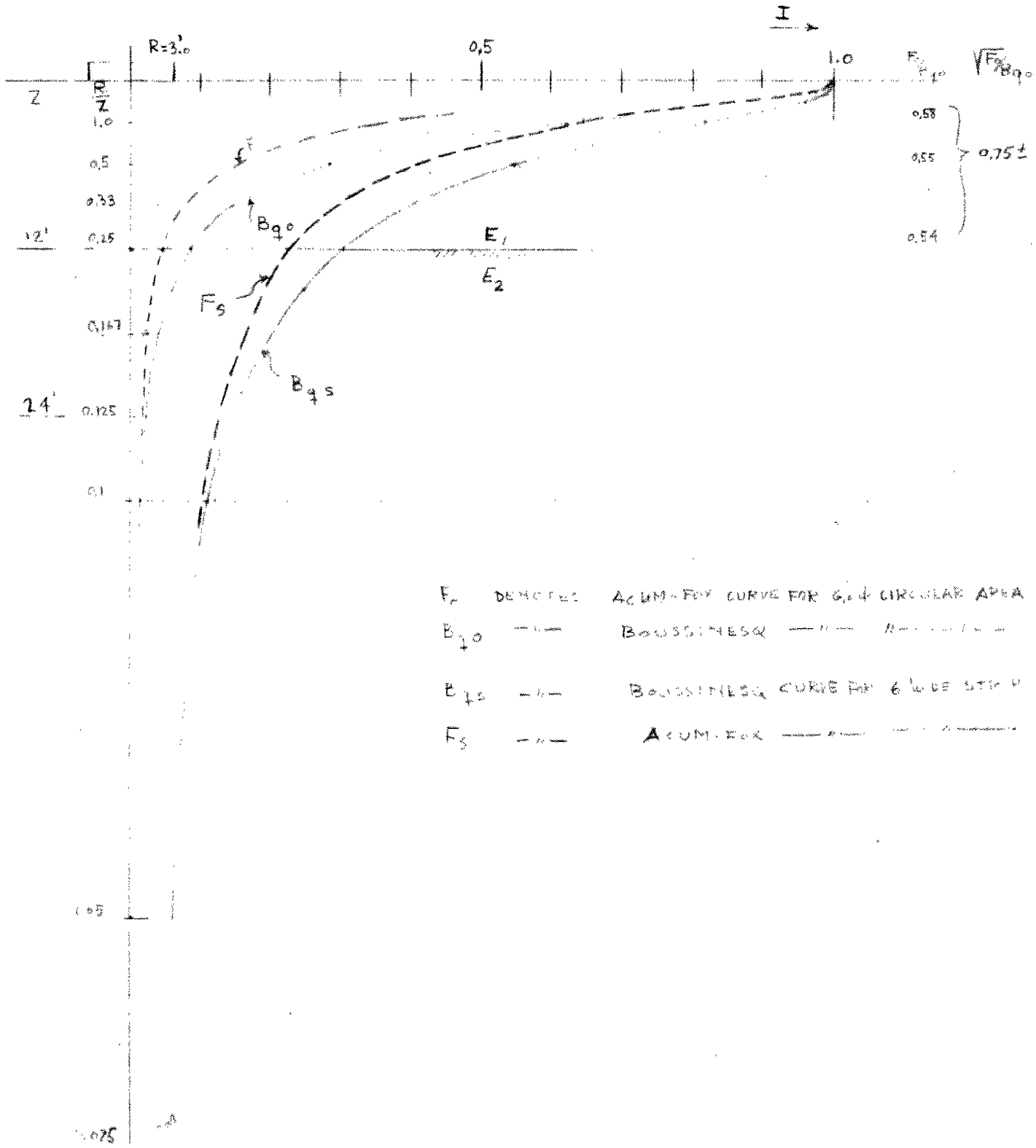
FORM NO. 152 5M R.P.

BRIDGE PIERS - STRESS DISTRIBUTION IN
THREE-LAYERED SYSTEM
SHEET 24 JOB 837

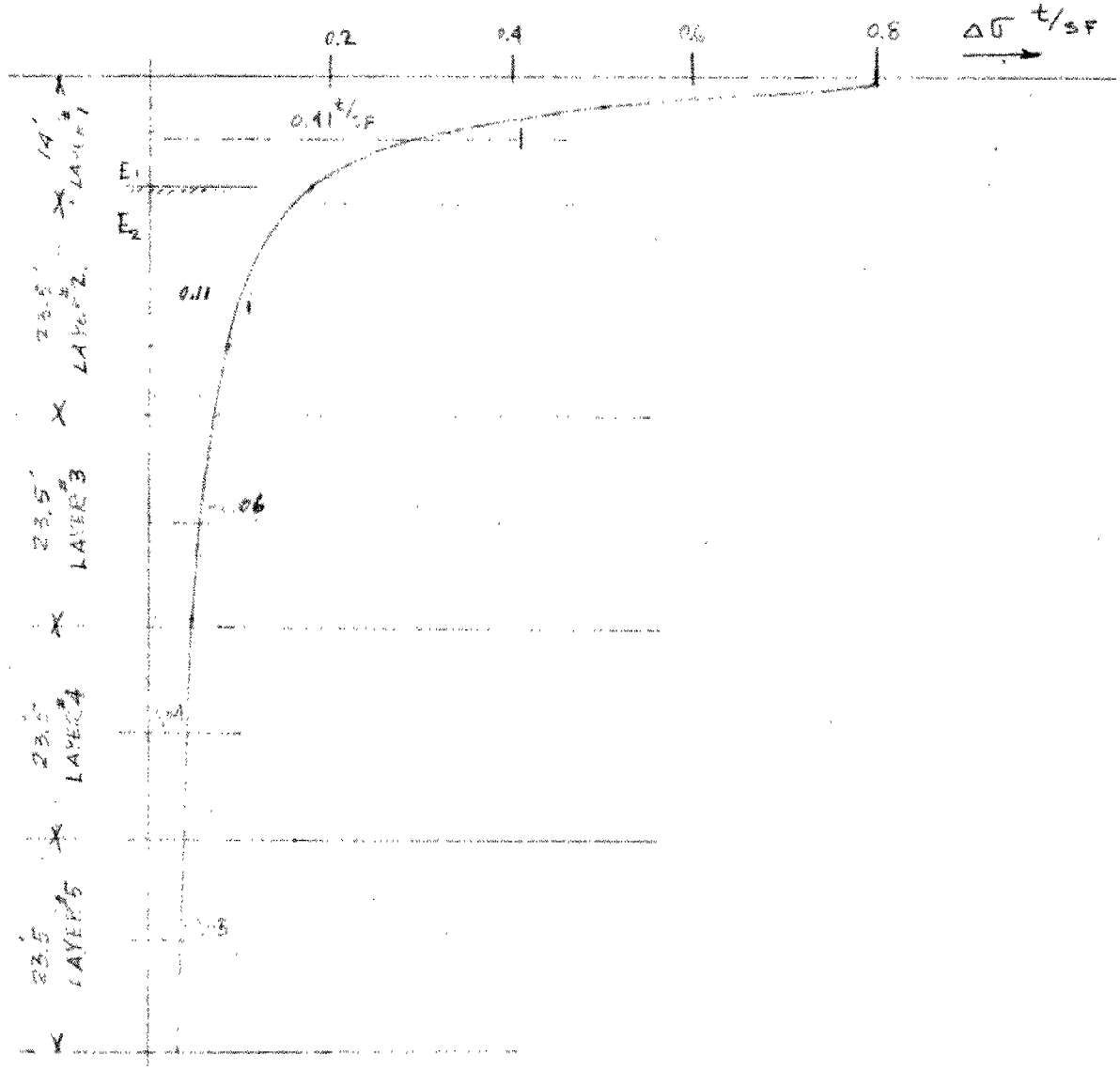


G 9-111 G
Logarithmic, 2¾ X 2 Cycles.
MICROGRAPH

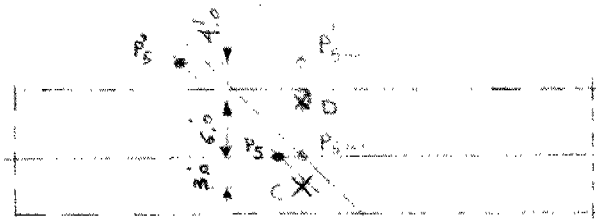
STRESS - INFLUENCE VALUES FOR 6'-0 WIDE STRIP.



STRESS DISTRIBUTION UNDER CENTRE OF 6' WIDE STRIP, @ 0.8 t/sf



SUBJECT: BRIDGE PIERS - STRESS DISTRIBUTION
ELASTIC SETTLEMENTS
CONSOLIDATION SETTLEMENTS



LAYER	P_o	$\Delta\sigma_{P_5'}$	$\Delta\sigma_{P_5}$	$\Delta\sigma_D$	$\Delta\sigma_C$	$P_o' + \Delta\sigma_D$	$P_o' + \Delta\sigma_C$
1	0.7	0.56	0.48	0.55	0.46	1.23	1.16
2	1.42	0.42	0.30	0.37	0.26	1.79	1.68
3	2.16	[illegible]	[illegible]	0.32	0.26	2.48	2.42
4	2.99	[illegible]	[illegible]	0.31	0.26	3.29	3.24
5	3.70	0.21	[illegible]	0.30	0.27	4.00	3.97

SETTLEMENT UNDER POINT "C" DUE TO PIER LOADING.

LAYER	Δh	P_o'	$\Delta\sigma$	$P_o' + \Delta\sigma$	e_o	Δe	$\frac{\Delta e}{1+e_o} \times 100$	y
1	14.0	1.16	0.41	1.57	0.599	0.0080	0.50	0.6
2	23.5	1.66	0.11	1.79	0.700	0.0062	0.364	0.55
3	23.5	2.42	0.06	2.48	0.755	0.0038	0.216	—
4	23.5	3.24	0.04	3.28	0.712	0.0018	0.105	—
5	23.5	3.97	0.03	4.00	0.663	0.0012	0.071	—
			0.24				0.746	

$\frac{0.50}{100} \times 14 \times 0.6 = 0.042$

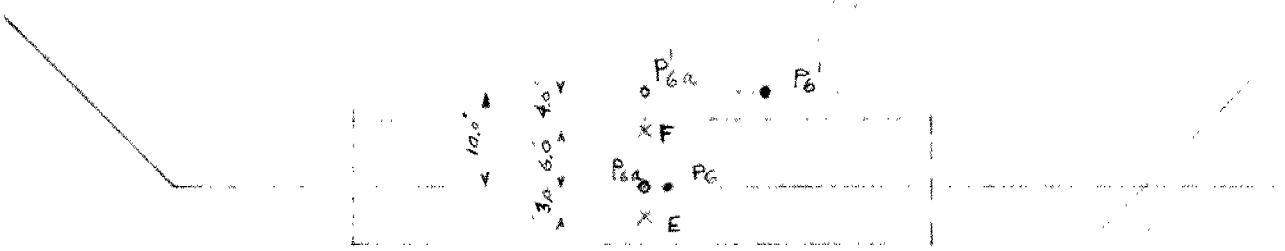
$\frac{0.746}{100} \times 23.5 \times 0.85 = 0.149$

$\delta_c = 0.191'$

$$\delta_i \sim 0.5 \sum \frac{\Delta\sigma}{E} \Delta h = 0.5 \left(\frac{0.41}{500} \times 14 + \frac{0.24}{100} \times 23.5 \right) = (0.011 + 0.055)\, 0.5 = 0.033$$

TOTAL SETTLEMENT $\delta_i + \delta_c = 0.224$

FORM NO. 152 5M R.P



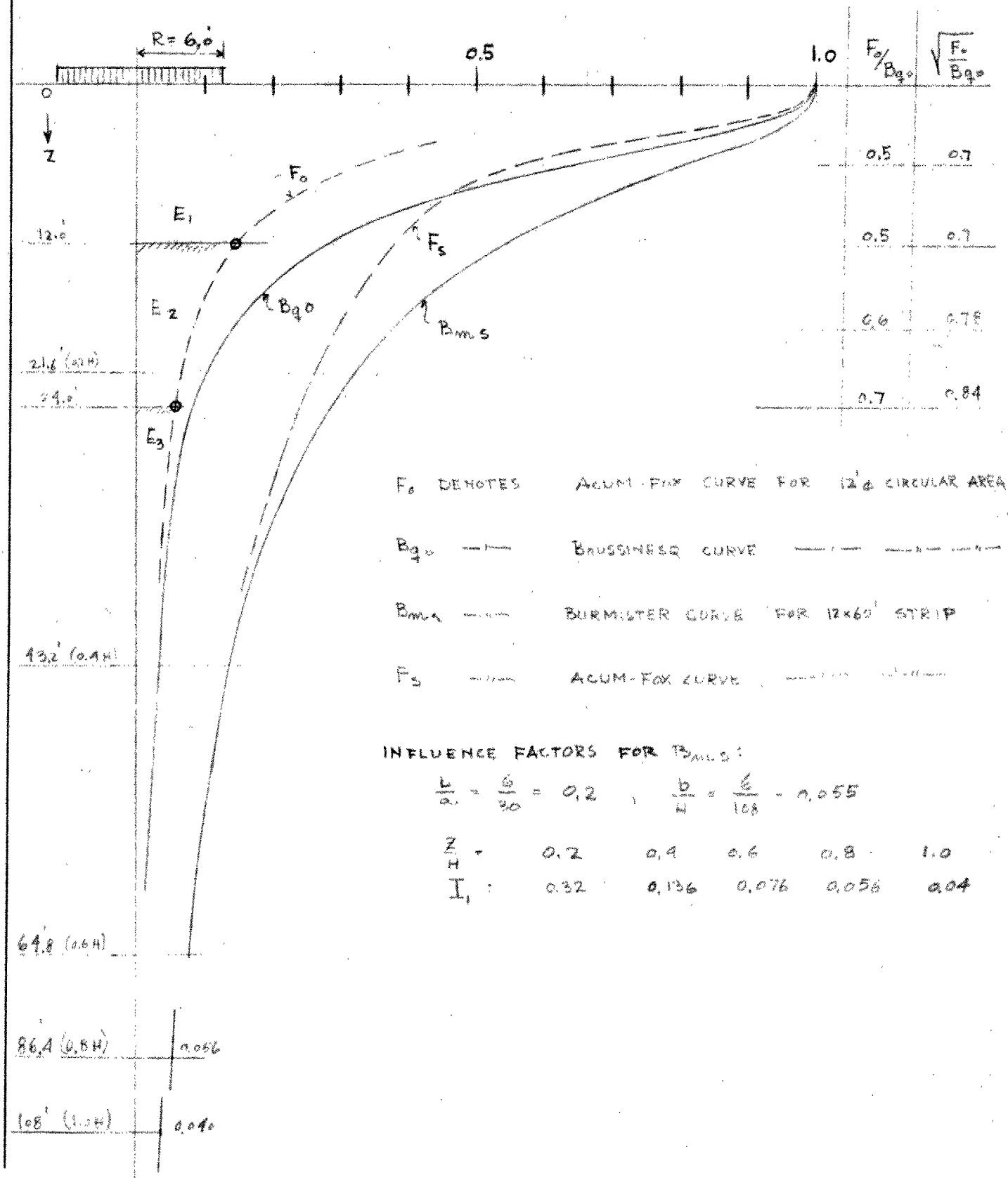
LAYER	P_o	$\Delta\sigma_{P'_o}$	$\Delta\sigma_{P_6}$	$\Delta\sigma_F$	$\Delta\sigma_E$	P'_o $P_o+\Delta\sigma_F$	P'_o $P_o+\Delta\sigma_E$
1	0.7	0.58	0.50	0.55	0.48	1.25	1.18
2	1.42	0.48	0.36	0.43	0.32	1.85	1.74
3	2.16	0.43	0.34	0.40	0.34	2.56	2.50
4	2.98	0.40	0.35	0.38	0.33	3.36	3.31
5	3.70	0.37	0.33	0.35	0.32	4.05	4.02

SETTLEMENT UNDER POINT E DUE TO PIER LOADING.

SINCE THE OVERBURDEN PRESSURE P'_o DIFFERS VERY LITTLE FROM THE VALUES FOUND FOR POINT 'C' THE SETTLEMENT UNDER POINT E WILL BE PRACTICALLY THE SAME AS UNDER POINT C.

FORM NO. 152 5M R.P.

STRESS - INFLUENCE FACTORS FOR 12'×60' STRIP.



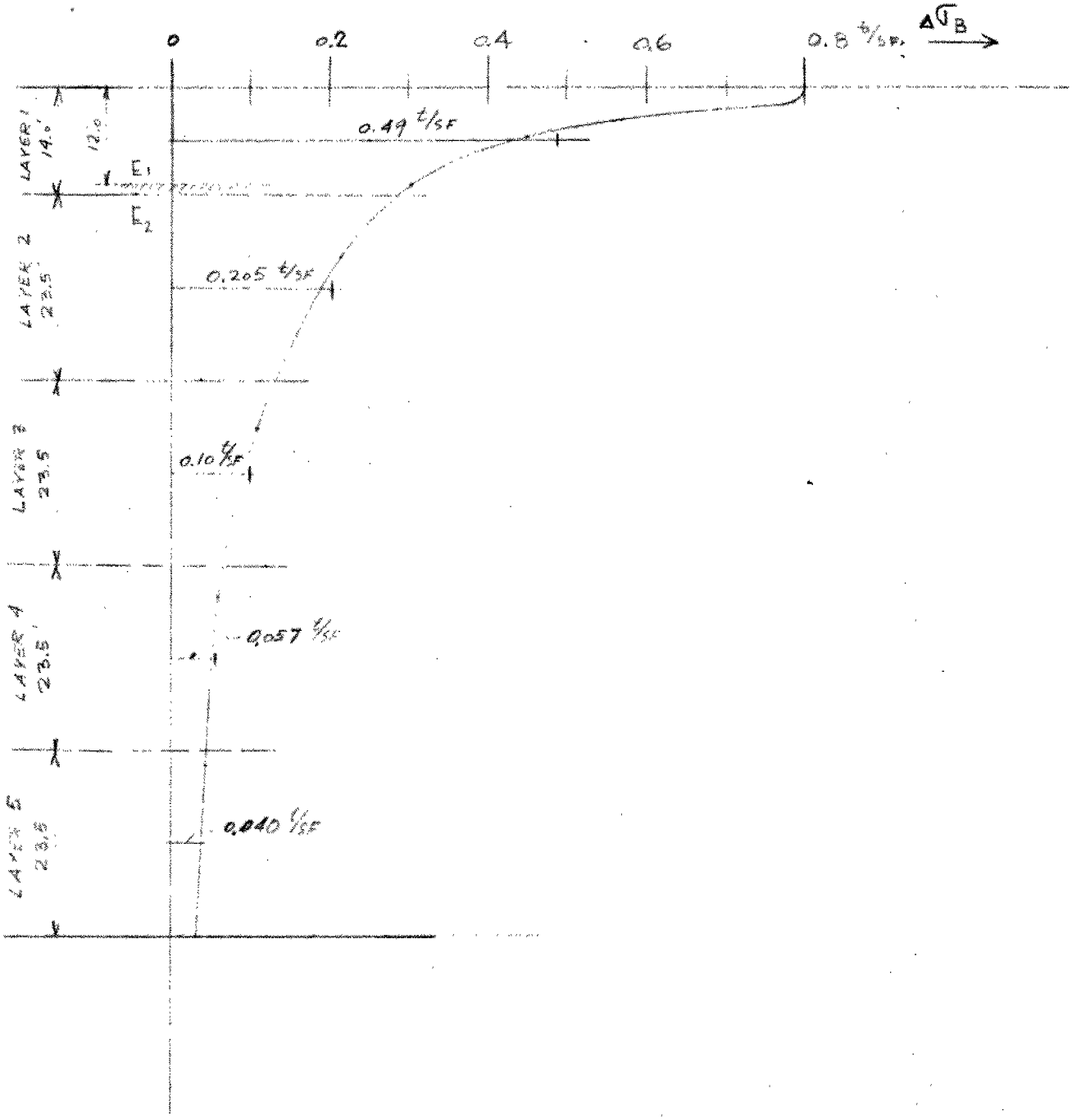
INFLUENCE FACTORS FOR B_{mS}:

$\frac{b}{a} = \frac{6}{30} = 0.2$, $\frac{b}{H} = \frac{6}{108} = 0.055$

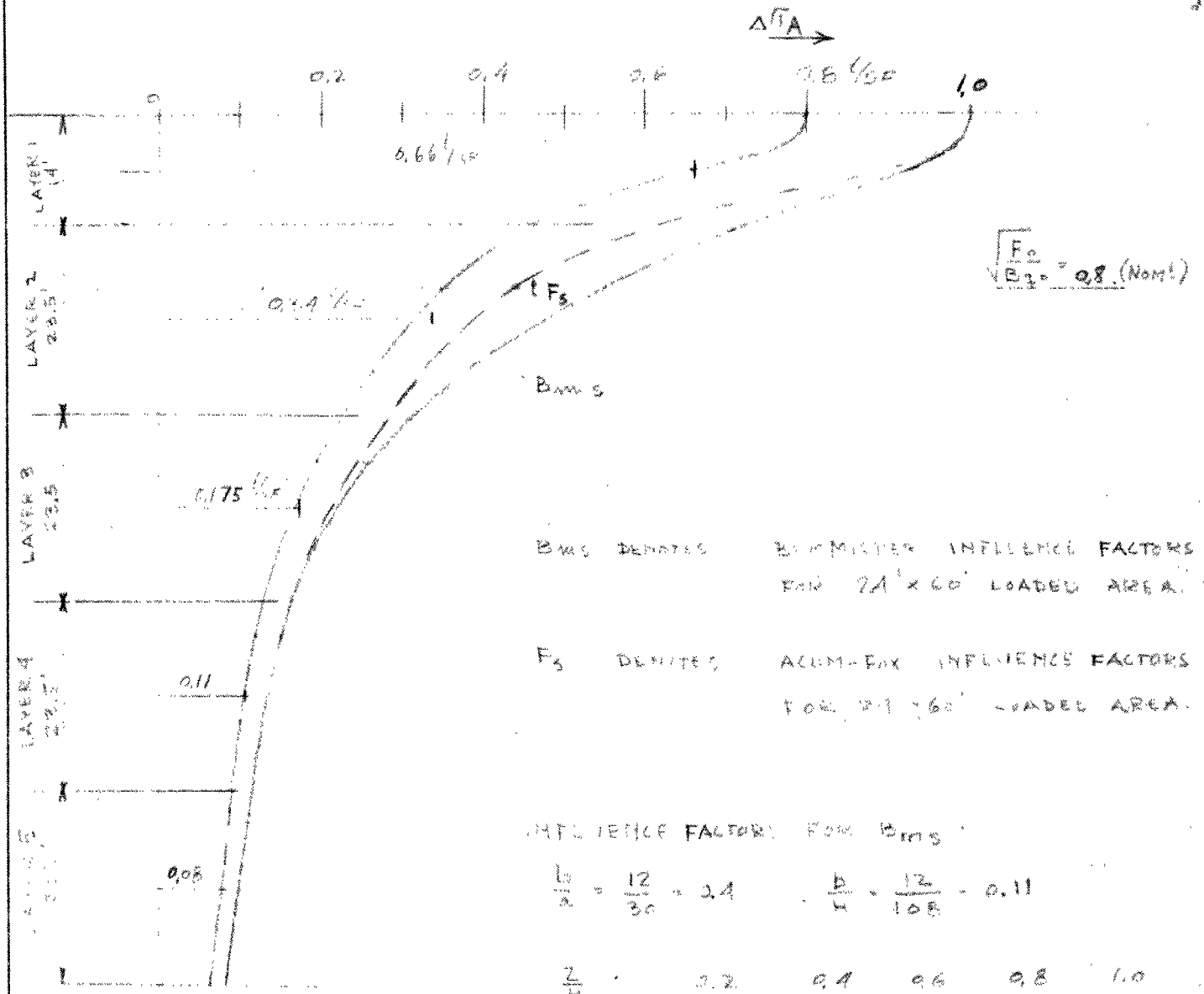
$\frac{Z}{H}$ =	0.2	0.4	0.6	0.8	1.0
I_1 :	0.32	0.136	0.076	0.056	0.04

FORM NO. 152 5M R.P.

STRESS DISTRIBUTION UNDER CENTRE OF 12'0 × 66'-0 STRIP, q = 0.8 t/SF



STRESS INFLUENCE FACTORS AND STRESS DISTRIBUTION UNDER CENTRE OF 24' x 60' LOADED AREA, q = 0.8 t/SF.



B_{ms} DENOTES BOUSSINESQ INFLUENCE FACTORS FOR 24' x 60' LOADED AREA.

F_s DENOTES ACUM-FOX INFLUENCE FACTORS FOR 24' x 60' LOADED AREA.

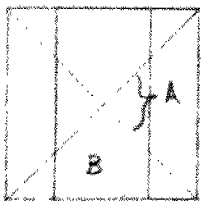
INFLUENCE FACTORS FOR B_{ms}:

$\frac{L}{B} = \frac{12}{30} = 2.4$ $\frac{b}{h} = \frac{12}{108} = 0.11$

$\frac{Z}{H}$:	0.2	0.4	0.6	0.8	1.0
I_1 :	0.552	0.260	0.152	0.112	0.084

FORM NO. 152 5M R.P.

SUBJECT: BRIDGE PIERS - STRESS DISTRIBUTION
ELASTIC SETTLEMENTS
CONSOLIDATION SETTLEMENTS



LAYER	$\Delta\sigma_A$	$\Delta\sigma_B$	$\Delta\sigma = \frac{1}{2}(\Delta\sigma_A - \Delta\sigma_B)$
1	0.66	0.49	0.085 t/SF
2	0.34	0.205	0.067
3	0.175	0.10	0.037
4	0.11	0.057	0.026
5	0.08	0.040	0.020

SETTLEMENT UNDER POINT 'D' DUE TO PIER LOADING.

LAYER	Δh	P_o'	$\Delta\sigma$	$P_o' + \Delta\sigma$	e_o	Δe	$\frac{\Delta e}{1+e_o} 100$	γ
1	14.0	1.23	0.085	1.315	0.597	0.0015	0.094	0.6
2	23.5	1.79	0.067	1.857	0.684	0.0035	0.206	0.85
3	23.5	2.48	0.037	2.517	0.750	0.0020	0.114	- -
4	23.5	3.27	0.026	3.316	0.710	0.0012	0.070	- -
5	23.5	4.00	0.020	4.020	0.682	0.0007	0.042	- -
							0.432	

$\frac{0.094}{100} \times 14 \times 0.6 = 0.0079'$

$\frac{0.432}{100} \times 23.5 \times 0.85 = 0.0863$

$\delta_c = 0.0942'$

$\delta_i = 0.020$

$\delta_i + \delta_c = 0.114'$

SETTLEMENT UNDER POINT 'F' DUE TO PIER LOADING.
SINCE THE OVERBURDEN PRESSURE, P_o', IS PRACTICALLY THE SAME FOR POINT 'D' & 'F' THE SETTLEMENTS WILL ALSO BE THE SAME

FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT: COMBINED SETTLEMENT

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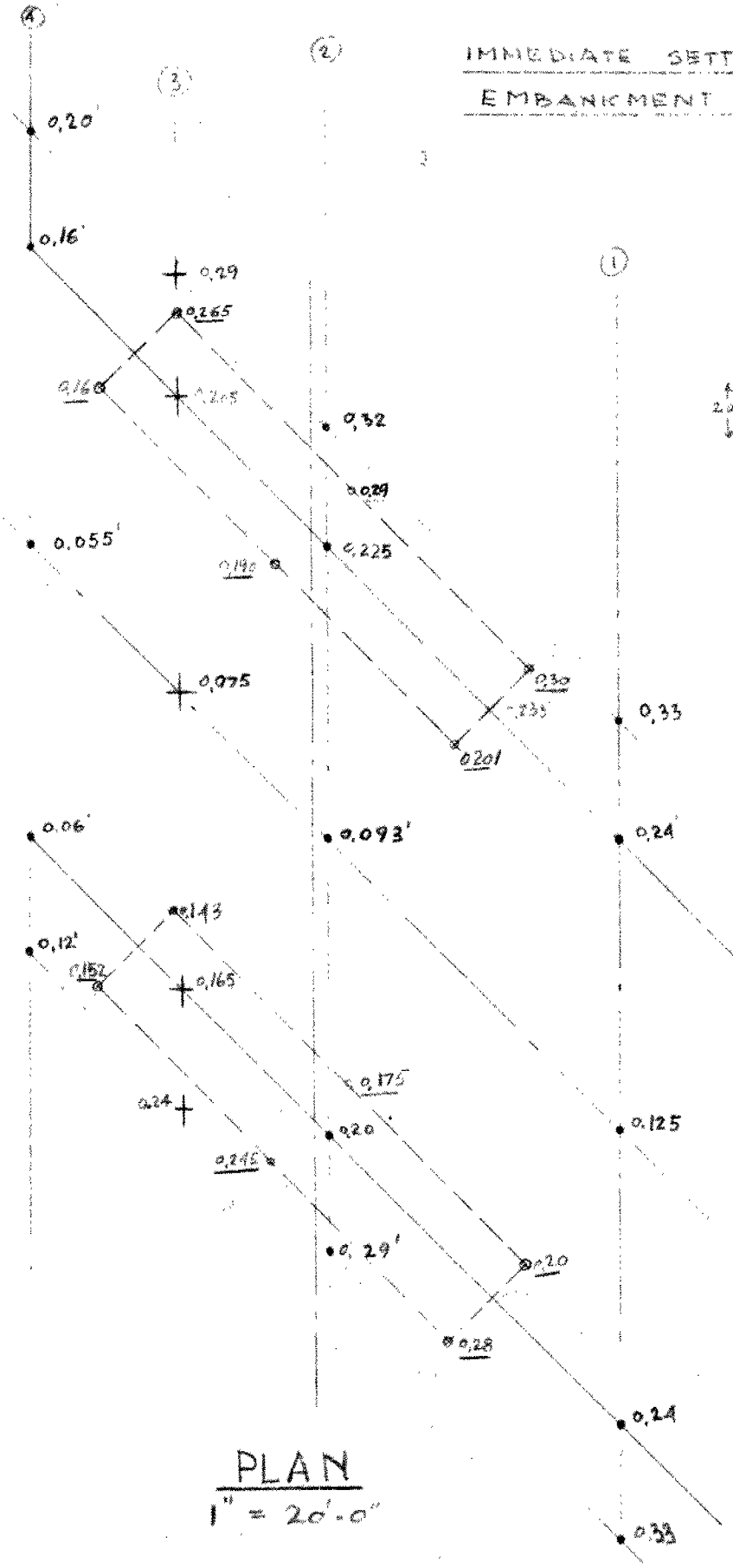
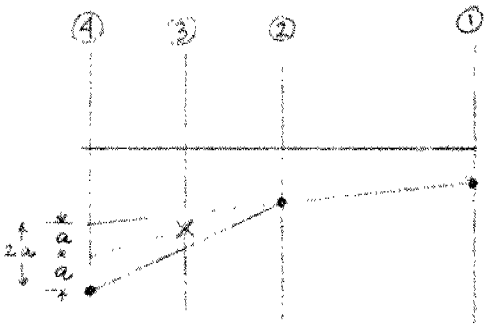
SHEET 33 OF
BY DATE
APP DATE

IMMEDIATE SETTLEMENTS DUE TO EMBANKMENT LOAD



PLAN

1" = 20'-0"



METHOD OF INTERPOLATING SETTLEMENTS ON LINE 3

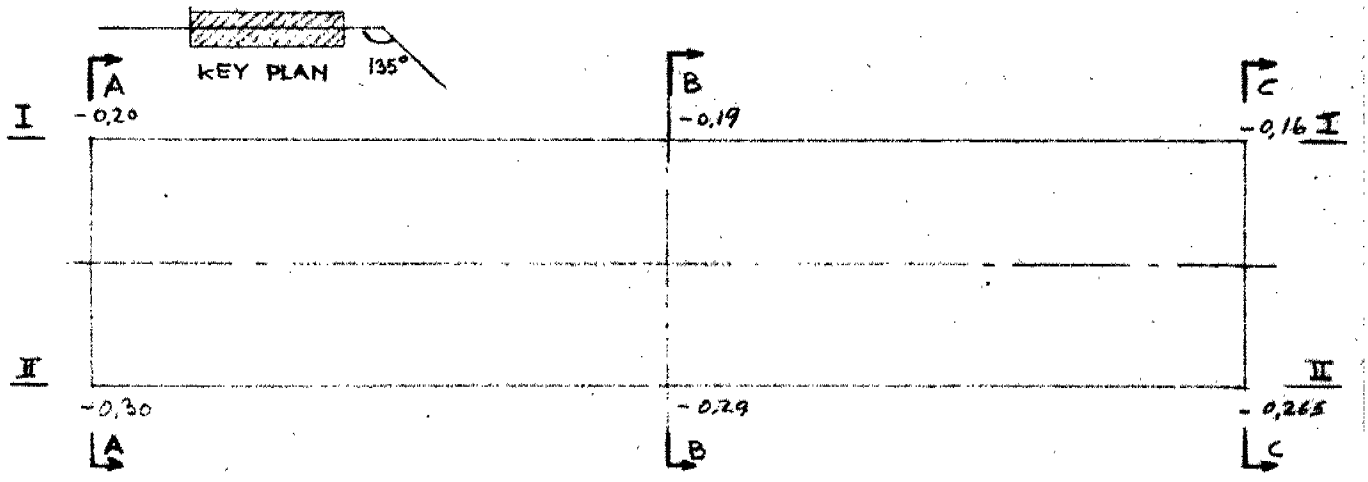
• DENOTES CALCULATED SETTLEMENTS

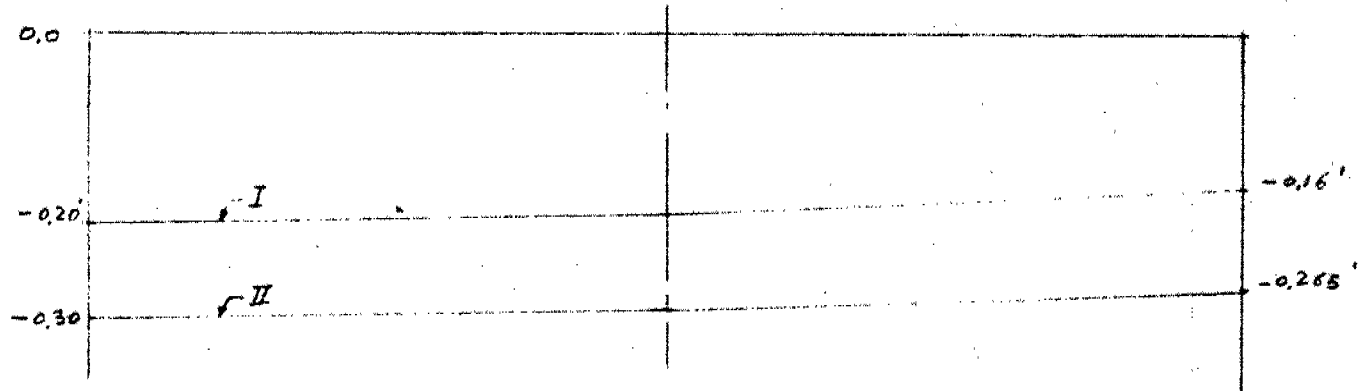
X —"— SETTLEMENTS ON LINE 3

O —"— SETTLEMENTS DETERMINED BY LINEAR INTERPOLATION BETWEEN POINTS ON LINE 1 TO 4

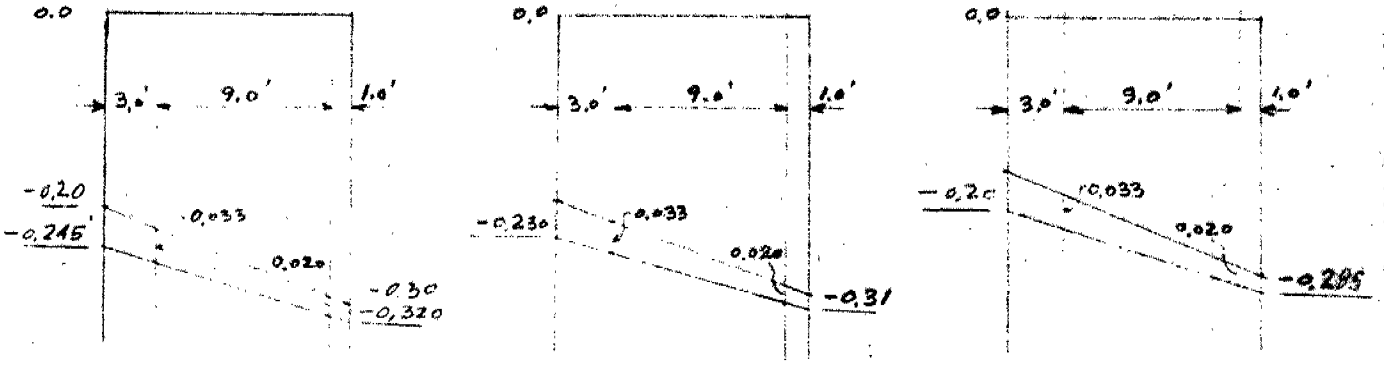
FORM NO. 152 5M P.F.

SUBJECT: COMBINED SETTLEMENT



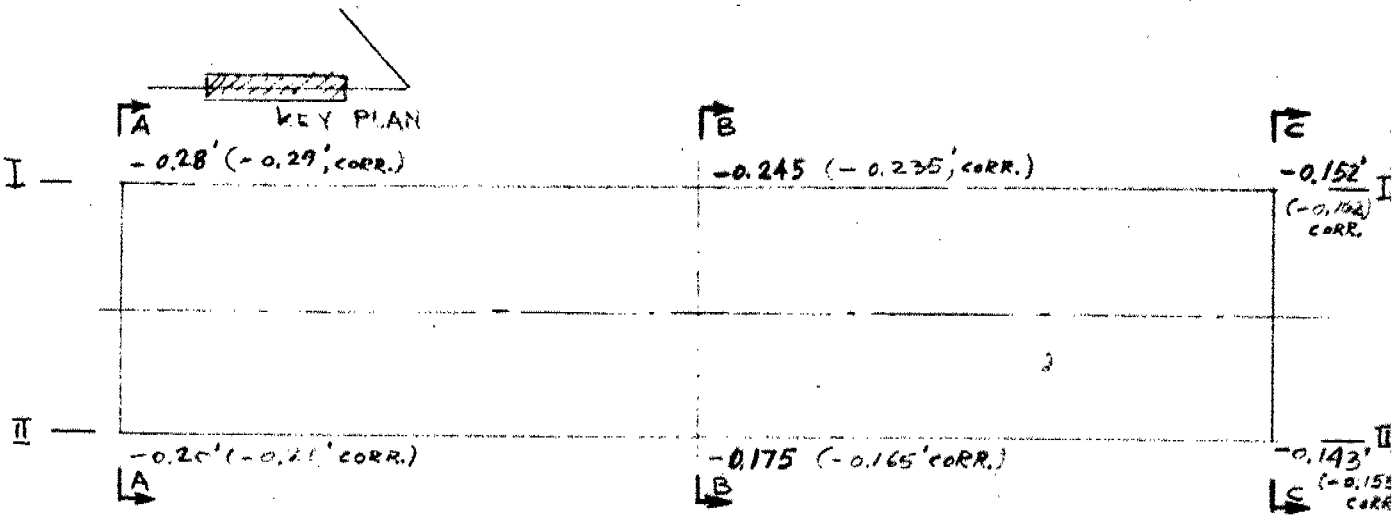


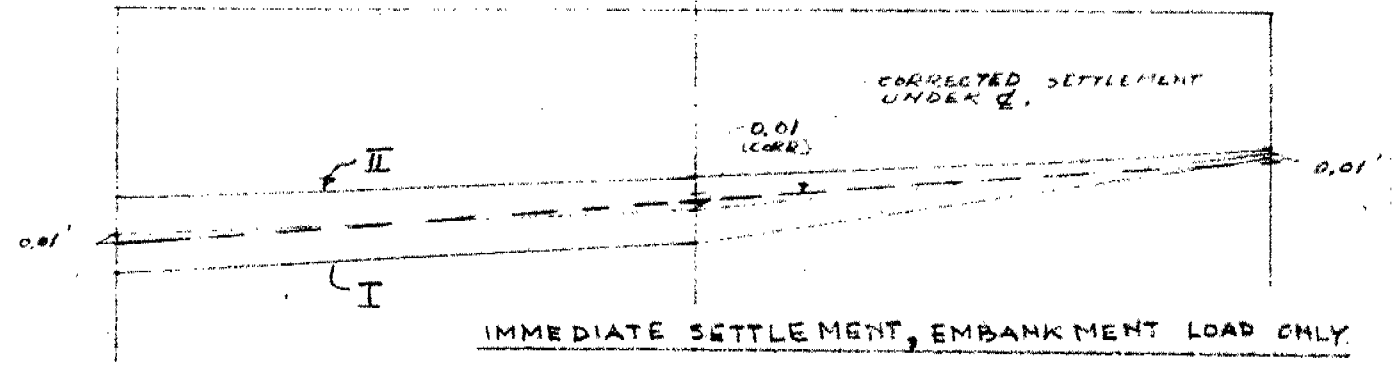
IMMEDIATE SETTLEMENT, EMBANKMENT LOAD ONLY



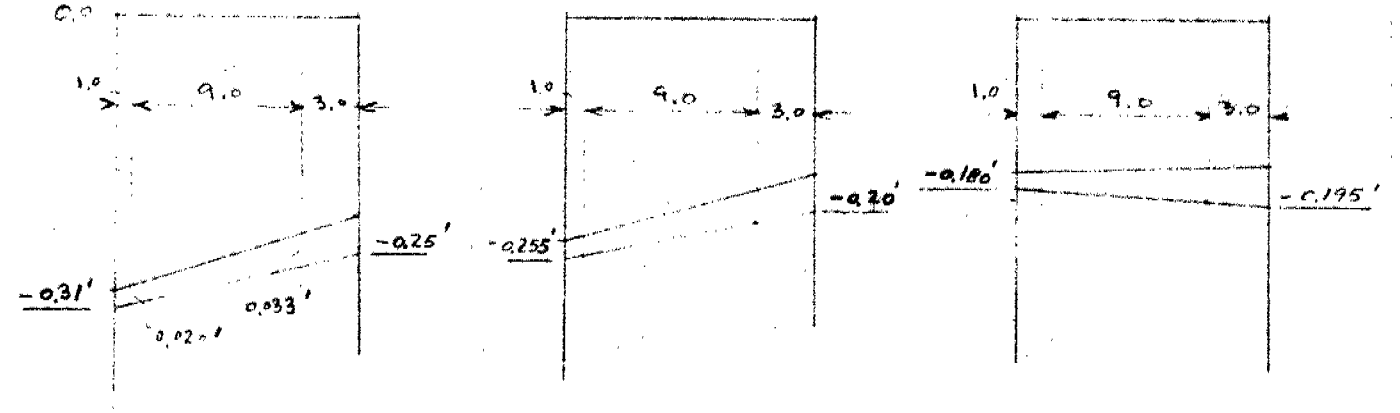
SECTION A-A SECTION B-B SECTION C-C

TOTAL IMMEDIATE SETTLEMENT OF FOOTING AREA.





IMMEDIATE SETTLEMENT, EMBANKMENT LOAD ONLY



SEC. A-A SEC. B-B SEC. C-C

TOTAL IMMEDIATE SETTLEMENT OF FOOTING AREA.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT: COMBINED SETTLEMENT

JOB 837 FILE
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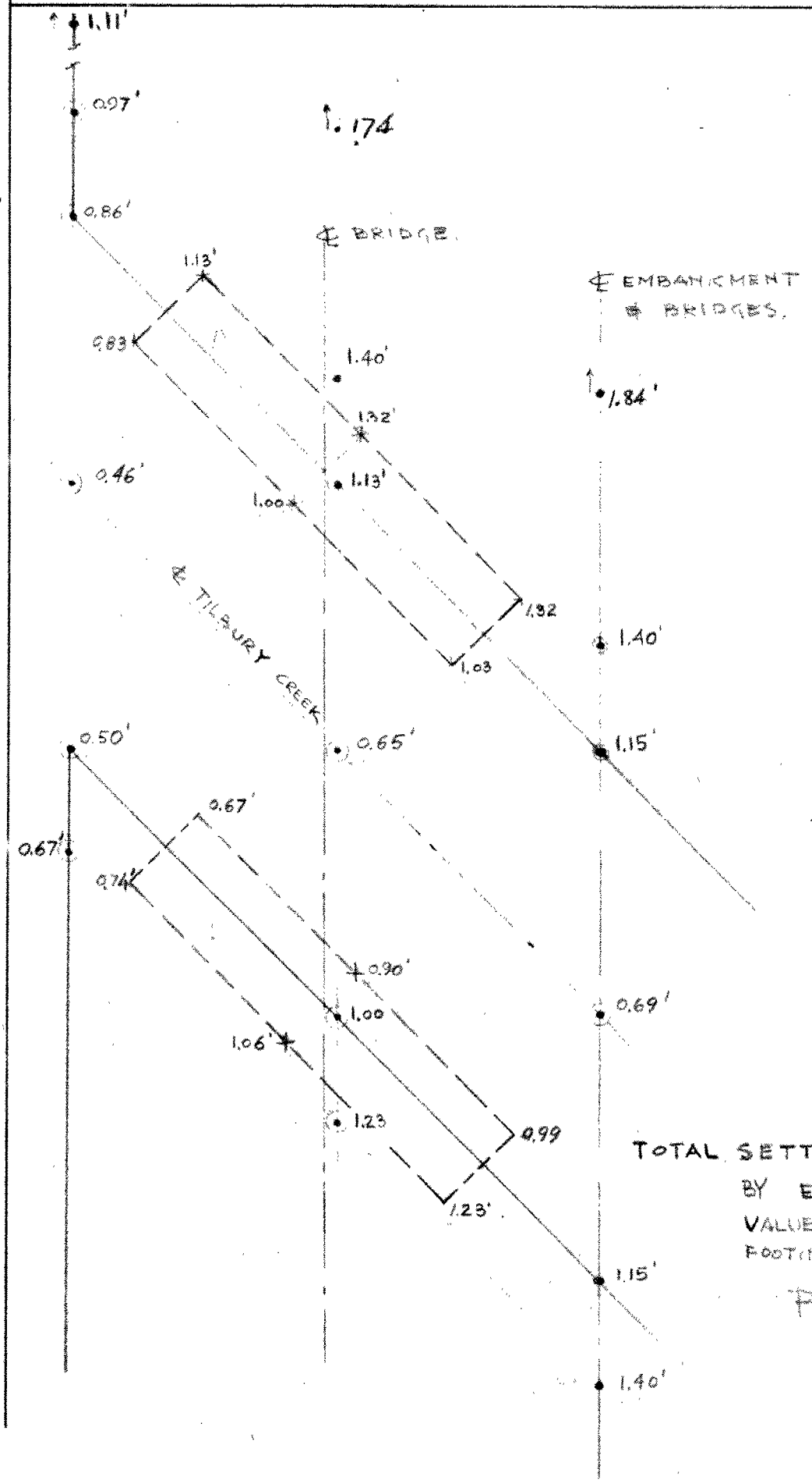
BY DATE
APP DATE



TOTAL SETTLEMENTS PRODUCED BY EMBANKMENT LOAD
VALUES AT OUTLINE OF FOOTINGS INTERPOLATED.

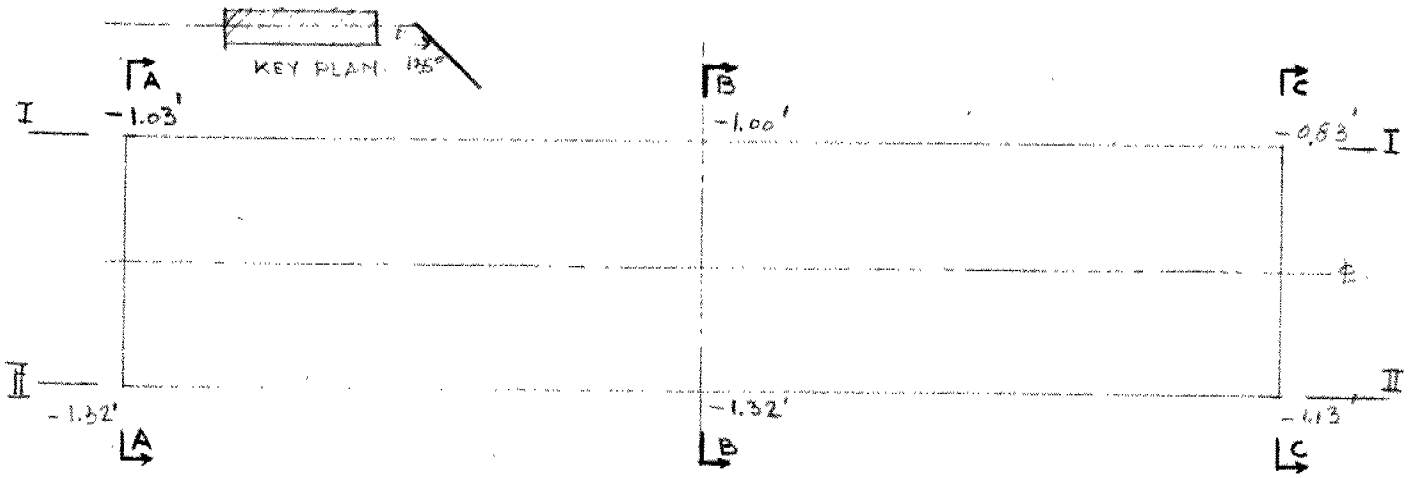
PLAN.
1" = 20'-0

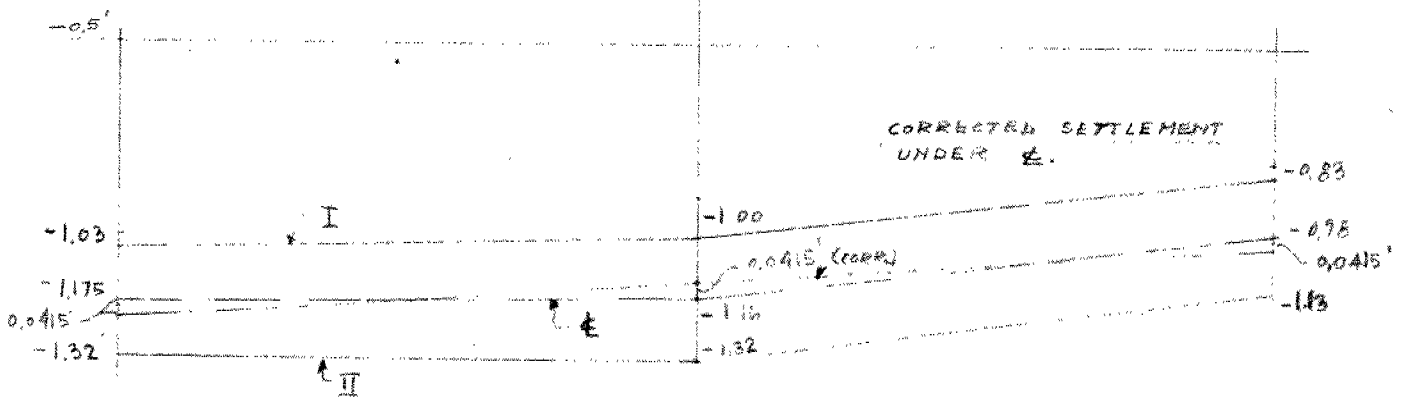
FORM NO. 152 5M R.P

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

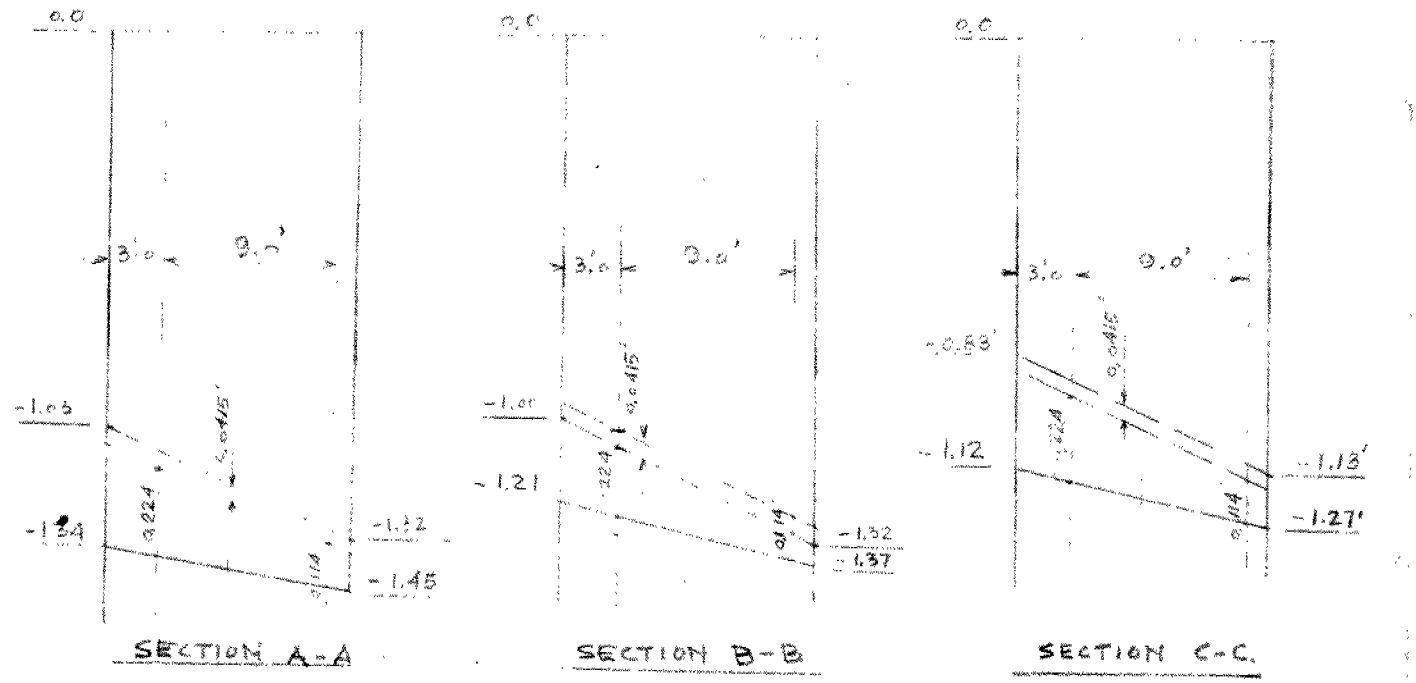
SUBJECT: COMBINED SETTLEMENT

JOB 837 FILE
SHEET 37 OF
BY DATE
APP DATE





TOTAL SETTLEMENT, EMBANKMENT LOAD ONLY



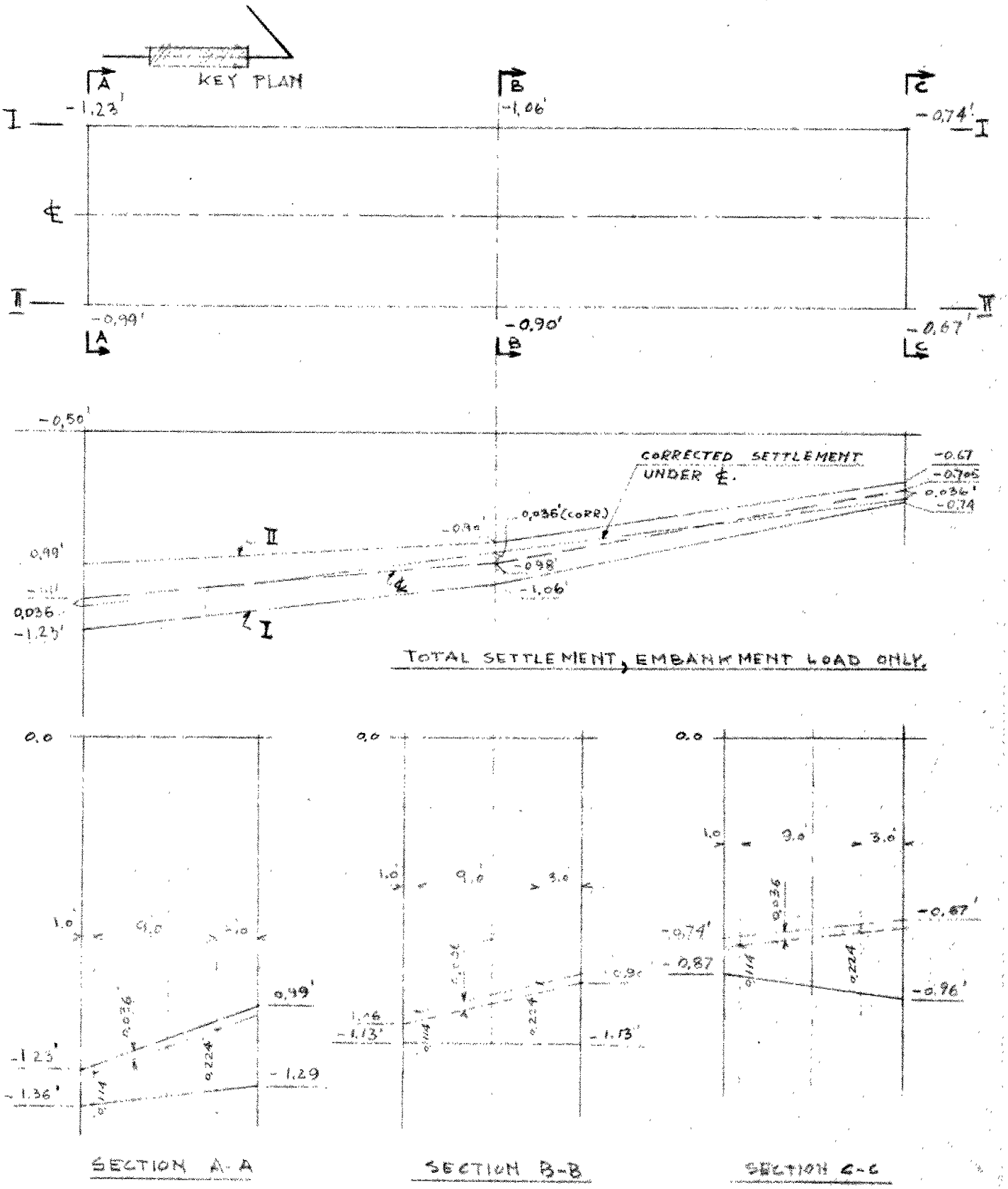
TOTAL SETTLEMENT OF FOOTING AREA

FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT: COMBINED SETTLEMENT

JOB 837 FILE
SHEET 38 OF
BY DATE
APP DATE



TOTAL SETTLEMENT OF FOOTING AREA

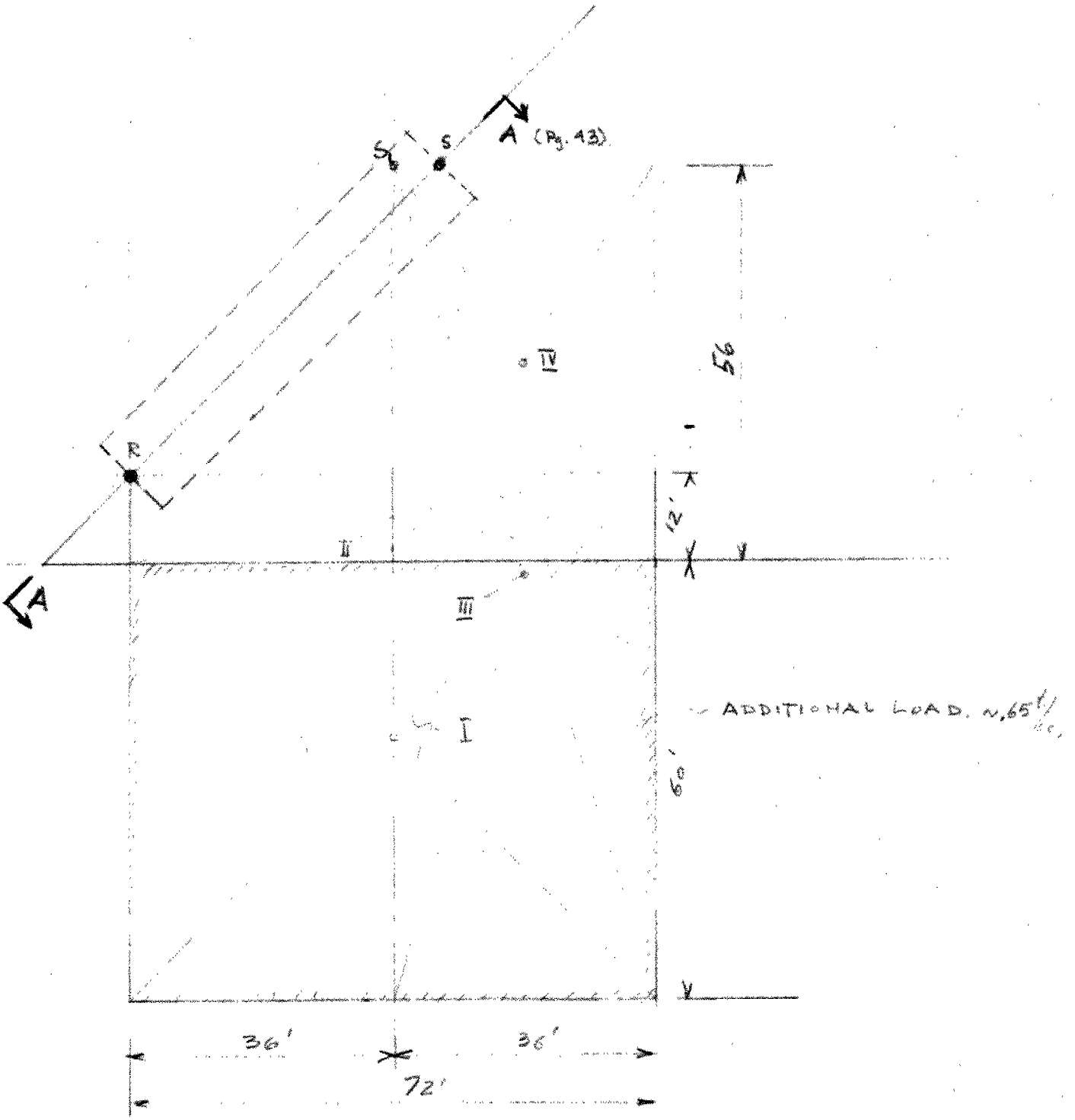
FORM NO. 152 5M P.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

SUBJECT: TILBURY CREEK BRIDGE SETTLEMENT STUDY.

JOB 837 FILE
SHEET 39 OF
BY DATE
APP DATE

EXTRA FILL TO ELIMINATE DIFFERENTIAL SETTLEMENTS OF PIERS.



FORM NO. 152 5M R.P

POINT R.

ELASTIC SETTLEMENTS.

$$\rho_i = \frac{H}{E} I_z \times p.$$

$H = 118' \sim 120$

$E = 200$ t/SF

$p = 0.65$ t/SF

I_z = BURHEISTER'S INFLUENCE FACTOR ($\mu = 0.4$).

I: $\frac{b}{a} = \frac{72}{72} = 1$, $\frac{b}{H} = \frac{72}{120} = 0.6$, I_z : 0.152

II $\frac{b}{a} = \frac{72}{12} = 6$, $\frac{b}{H} = \frac{72}{120} = 0.6$ I_z : 0.070

$I_{z\,(I-II)} = 0.082$

$$\rho_i = \frac{120}{200} \times 0.082 \times 0.65 = 0.032'$$

CONSOLIDATION SETTLEMENTS.

STRESS DISTRIBUTION:

	0×H	0.2 H	0.4 H	0.6 H	0.8 H	1.0 H.
AREA I	0.25	0.249	0.231	0.203	0.177	0.146
AREA II.	0.25	0.138	0.076	0.052	0.040	0.031
$I_{z\,(I-II)}$	0.00	0.111	0.155	0.151	0.137	0.115
$\Delta\sigma = 0.65\,I$.	0.00	0.072	0.101	0.098	0.089	0.075
$\Delta\sigma$	0.036	0.087	0.100	0.094	0.089	t/SF

THE STRAIGHT PART OF THE CONSOLIDATION CURVE:

$$\log p = a - b \cdot e$$

$$\log p' - \log p_o = b\,(e_o - e) = b\,\Delta e$$

LAYER	ΔH	P_o t/sf	$\Delta\sigma$	$P' = P_o + \Delta\sigma$	e_o	b	Δe	$\frac{\Delta e}{1+e_o} 100$	$\frac{\Delta e}{1+e_o} \times \Delta h \times \gamma$
1	23.5'	0.70	0.036	0.736	0.61	20.0	0.0011	0.0684	0.011
2	—"—	1.42	0.087	1.507	0.72	4.25	0.0066	0.326	
3	—"—	2.16	0.100	2.260	0.765	3.58	0.0055	0.312	0.209
4	—"—	2.98	0.094	3.074	0.725	3.58	0.0032	0.185	
5	—"—	3.70	0.089	3.789	0.700	3.58	0.0028	0.165	
								0.988	$\rho_c = 0.22'$

SETTLEMENT AT POINT R. $\rho_i = 0.032'$

$\rho_c = 0.22'$

TOTAL. $\rho_i + \rho_c = 0.252'$

POINT S

SETTLEMENTS ASSUMED THE SAME AS FOR POINT "S_1"

ELASTIC SETTLEMENTS.

AREA III. $\frac{b}{a} = \frac{36}{116} = 0.31$; $\frac{b}{H} = 0.3$

AREA IV. $\frac{b}{a} = \frac{36}{56} = 0.64$, $\frac{b}{H} = 0.3$.

$I_{z\,III} = 0.127$

$I_{z\,IV} = 0.118$

$I_{z(III-IV)} = 0.009$.

$I_z = 2 \cdot I_{z(III-IV)} = 0.018$

$$\rho_i = \frac{120}{200} \cdot 0.018 \cdot 0.65 = 0.007'$$

CONSOLIDATION SETTLEMENTS.

STRESS DISTRIBUTION:

	0.0	0.2 H	0.4 H	0.6 H	0.8 H	1.0 H
AREA III	0.25	1.234	0.185	0.150	0.127	0.106
AREA IV	0.25	0.230	0.167	0.130	0.098	0.077
I	0.00	0.004	0.018	0.020	0.029	0.029
2 I	0.00	0.008	0.036	0.040	0.058	0.058
$\Delta\sigma = I \times 0.65$	1.00	0.0052	0.0244	0.026	0.0377	0.0377
$\Delta\sigma$	0.003	0.0148	0.025	0.032	0.038	t/SF

LAYER	ΔH	P_o t/s=	$\Delta\sigma$	$P' = P_o + \Delta\sigma$	e_o	b	Δe	$\frac{\Delta e}{1+e_o} 100$	$\frac{\Delta e}{1+e_o} \times \Delta H \times \nu$
1	235	0.70	0.003	0.703	0.61	20.10	0.00009	0.0056	$\nu = 0.7$ 0.0009
2	-"-	1.42	0.015	1.435	0.72	4.52	0.0010	0.058	
3	-"-	2.16	0.025	2.185	0.765	3.58	0.0014	0.078	$\nu = 0.7$ 0.066
4	-"-	2.98	0.032	3.012	0.725	3.58	0.0013	0.075	
5	-"-	3.70	0.038	3.738	0.700	3.58	0.0013	0.076	
								0.287	0.067'

SETTLEMENT AT POINT 'S' = (POINT 'S')

ρ_i = 0.007'

ρ_c = 0.067'

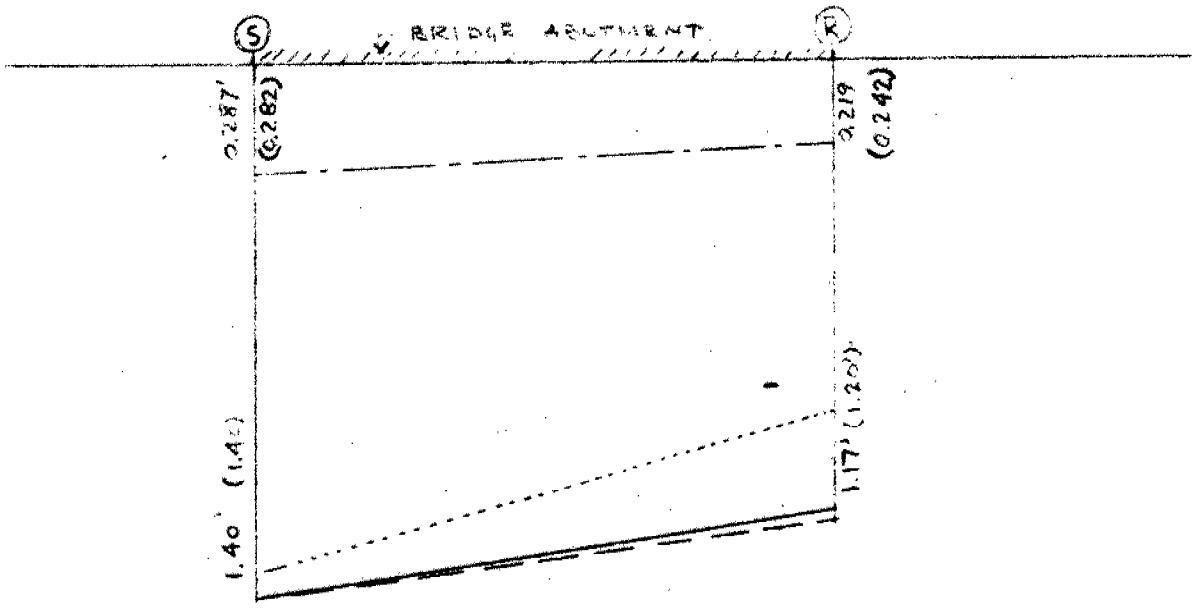
TOTAL. $\rho_i + \rho_c$ = 0.074'

FORM NO. 152 5M R.P.

H. G. ACRES & COMPANY LIMITED — CALCULATIONS

JOB 837 FILE
SHEET 43 OF
BY DATE
APP DATE

SUBJECT: SUMMARY OF EFFECT OF ADDITIONAL FILL PLACED AT ACUTE ANGLE OF EMBANKMENT BOUNDARY.



SECTION A-A.

FOR LOCATION SEE Pg 39

LEGEND:

— — — — — IMMEDIATE SETTLEMENTS WITH ADDITIONAL FILL.

——— ULTIMATE — · — · — · — · —

. . . . . ULTIMATE SETTLEMENTS WITHOUT ADDITIONAL FILL

— — — — — — — ULTIMATE SETTLEMENTS OF OPPOSITE ABUTMENT

1.23' SETTLEMENTS, SECTION A-A

(1.17) SETTLEMENTS, OPPOSITE ABUTMENT.

REFERENCE DWG: SETTLEMENTS OF BRIDGE PIERS AND EMBANKMENT, Job No 837, PLATE VII.

FORM NO. 152 5M R.P.

LIST OF PLATES

LIST OF PLATES

Plate I - General Plan and Sections
Location of Assumed Loadings
and Chosen Settlement Points

Plate II - Semi-Infinite Homogeneous Elastic System
Stress Influence Factors for Uniform
Rectangular and Circular Loads

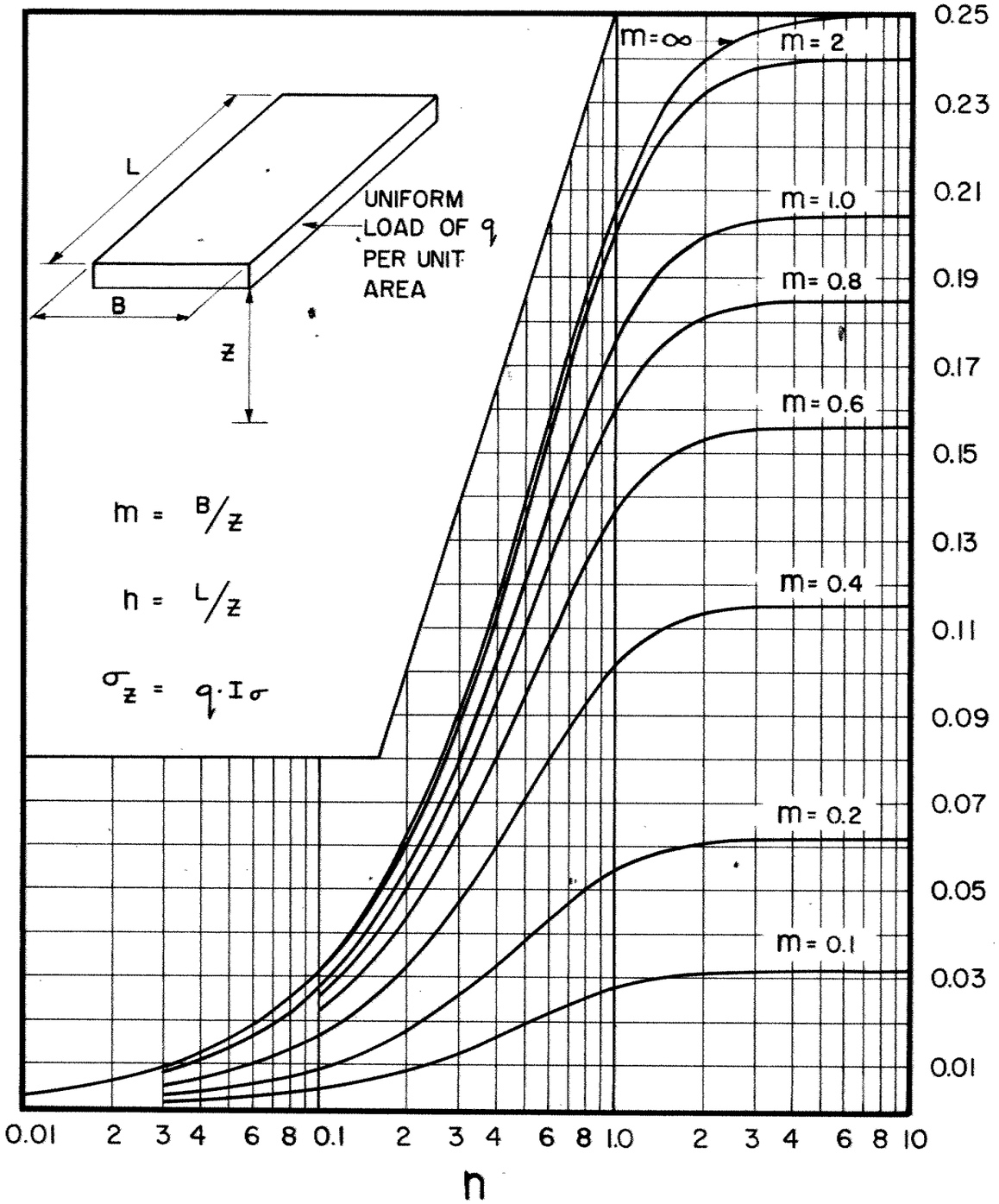
Plate III - Three-Layer Elastic System
Vertical Stress Influence Factors
for Uniform Circular Loads

Plate IV - Two-Layer Rigid Base System
Influence Factors for Vertical Stress
Changes and Vertical Settlements

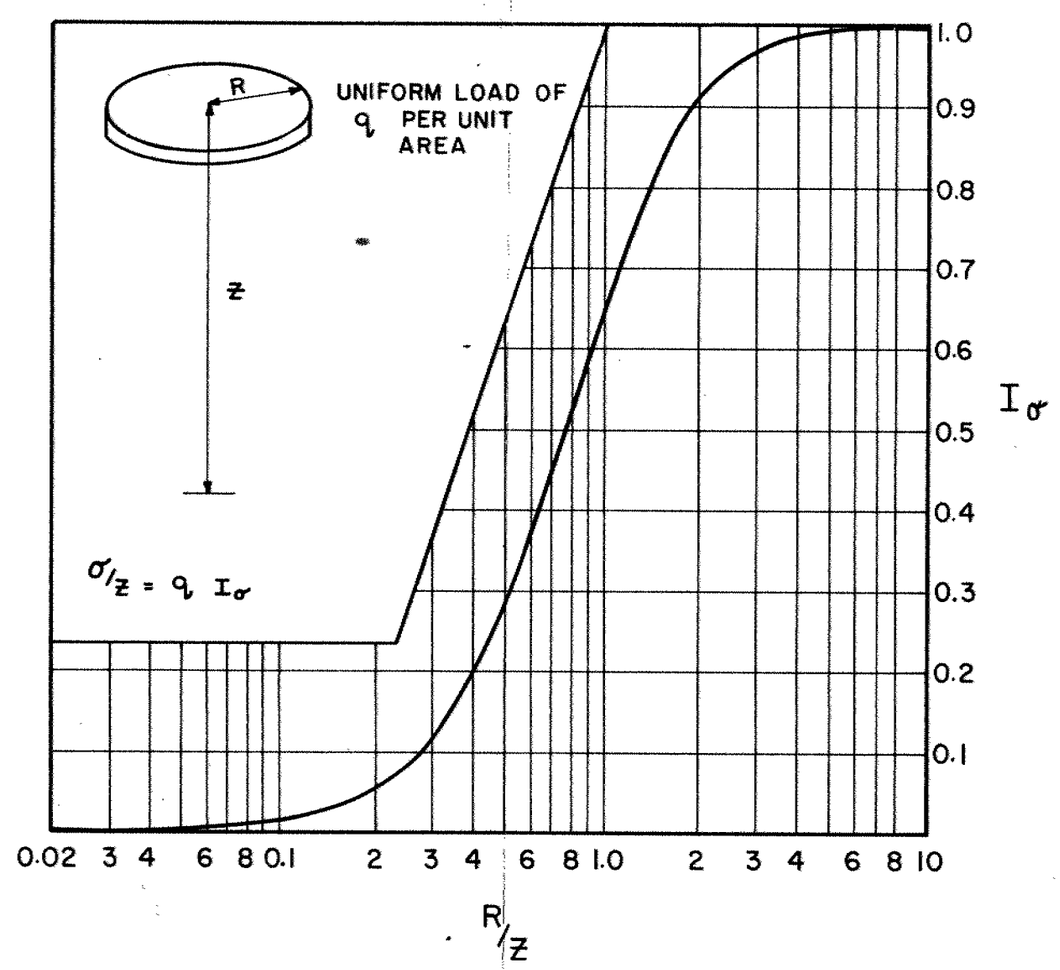
Plate V - Foundation Soil Properties

Plate VI - Oedometer Settlement Correction
Factor, ν for Strip Footings

Plate VII - Settlement of Bridge Piers and
Embankment



STRESS INFLUENCE FACTORS BENEATH CORNER OF FLEXIBLE RECTANGULAR LOAD



STRESS INFLUENCE FACTORS BENEATH CENTRE OF FLEXIBLE CIRCULAR LOAD

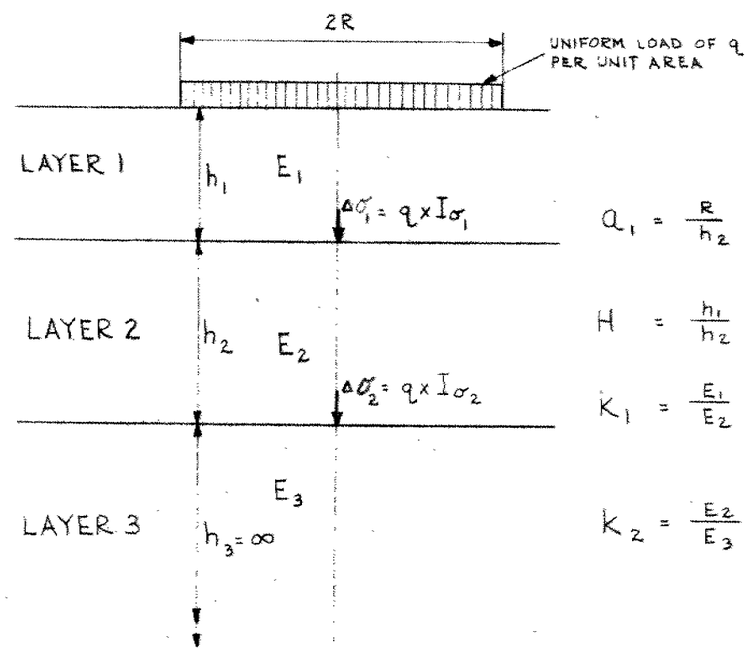
REFERENCE:
TERZAGHI, K, 1948
"THEORETICAL SOIL MECHANICS"
NEW YORK, JOHN WILEY AND SONS INC.

H. G. ACRES & COMPANY, LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY. 401 & TILBURY CREEK CROSSING	
SEMI-INFINITE HOMOGENEOUS ELASTIC SYSTEM. STRESS INFLUENCE FACTORS FOR UNIFORM RECTANGULAR AND CIRCULAR LOADS	
APPROVED	DATE: MARCH, 1960
D. H. MacDonald	SCALE / JOB No. 837
H.G. ACRES & COMPANY LIMITED	PLATE - II

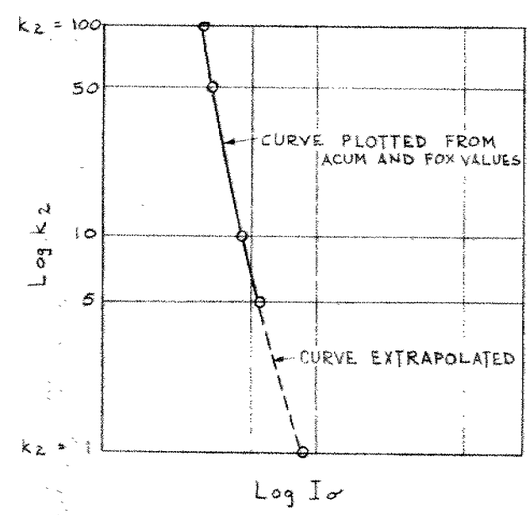
SK-837-B-28

	$H = 0.5$, $a_1 = 1$						$H = 1$, $a_1 = 1$						$H = 1$, $a_1 = 0.5$		$H = 2$, $a_1 = 0.5$	
k_1	5		10		20		5		10		20		5		5	
k_2	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}	I_{σ_1}	I_{σ_2}
100	0.591	0.0277	0.460	0.0237	0.328	0.0200	0.243	0.0142	0.156	0.0111	0.0935	0.0084	0.103	0.0037	0.0202	0.0014
50	0.598	0.0436	0.467	0.0374	0.335	0.0314	0.250	0.0223	0.161	0.0174	0.0976	0.0130	0.105	0.0059	0.0212	0.0022
10	0.634	0.1210	0.504	0.1040	0.370	0.0859	0.279	0.0611	0.187	0.0473	0.1180	0.0353	0.113	0.0166	0.0251	0.0062
5	0.664	0.1810	0.534	0.1560	0.399	0.1290	0.303	0.0922	0.208	0.0714	0.1350	0.0533	0.120	0.0255	0.0285	0.0095
EXTRAPOLATED VALUES 1	0.800	0.380	0.660	0.330	0.520	0.280	0.430	0.220	0.300	0.170	0.210	0.130	0.150	0.060	0.0450	0.0250

INFLUENCE FACTORS FOR VERTICAL STRESSES AT FIRST AND SECOND INTERFACES OF A THREE-LAYER SYSTEM



NOTATION – KEY DIAGRAM



EXTRAPOLATION METHOD TO DETERMINE INFLUENCE FACTORS FOR $k_2 = 1$

REFERENCE:

ACUM, W.E.A. AND FOX, L. 1951. COMPUTATION OF LOAD STRESSES IN A THREE-LAYER ELASTIC SYSTEM. GEOTECHNIQUE, VOL. II, pp. 293-300

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

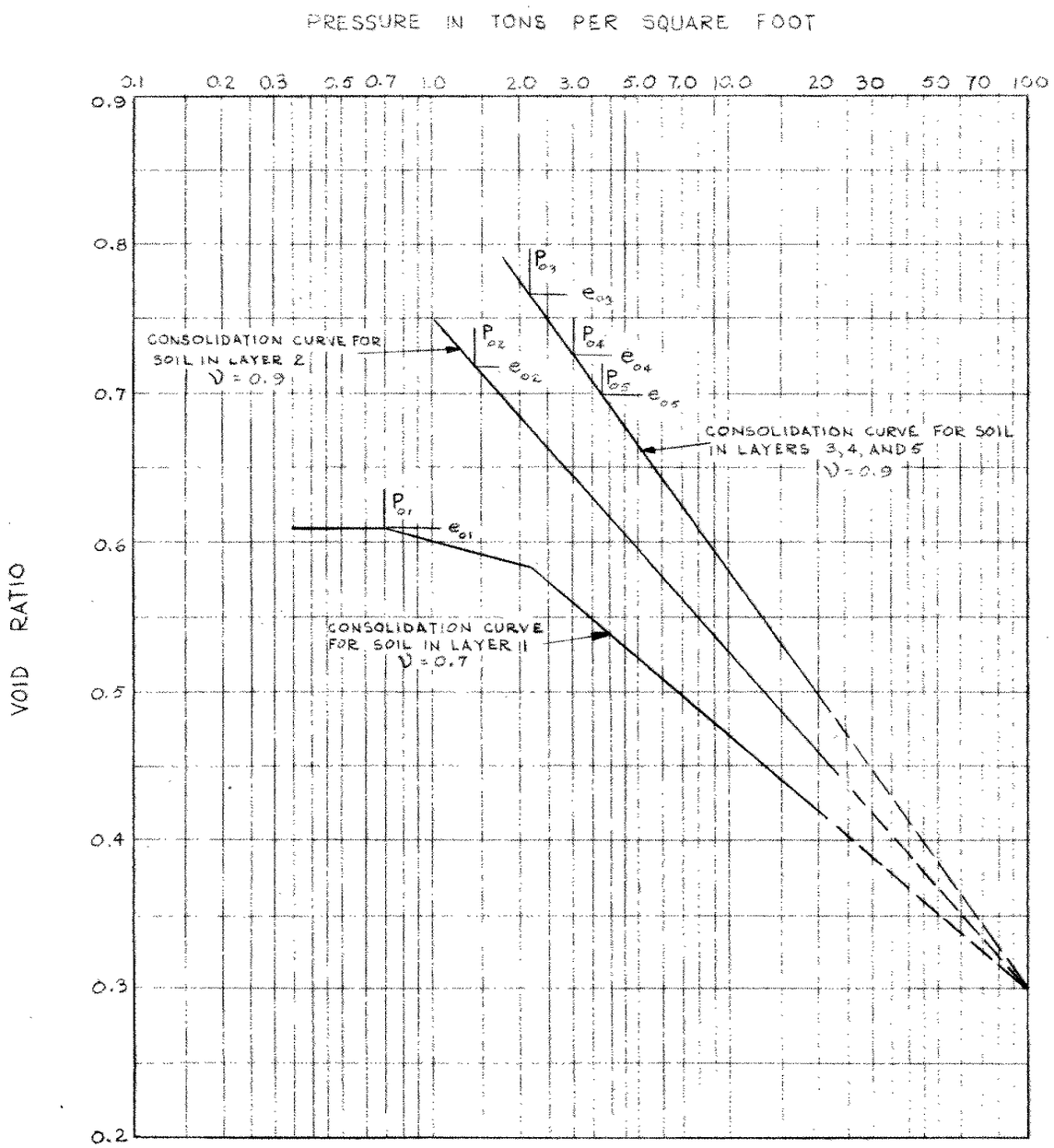
DEPARTMENT OF HIGHWAYS OF ONTARIO

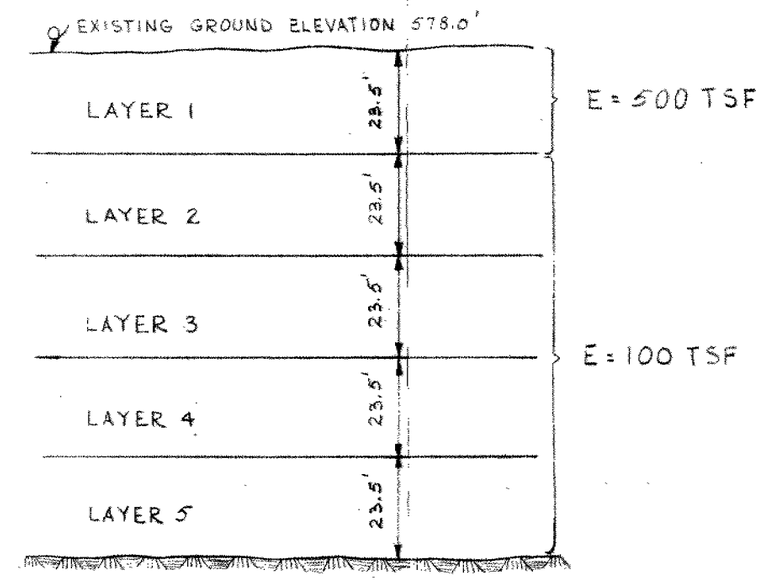
HWY. 401 & TILBURY CREEK CROSSING

THREE-LAYER ELASTIC SYSTEM.
VERTICAL STRESS INFLUENCE FACTORS FOR UNIFORM CIRCULAR LOADS

APPROVED	DATE: MARCH, 1960	
D. H. MacDonald	SCALE	JOB No. 837
H.G. ACRES & COMPANY LIMITED	PLATE - III	

SK-837-A-29





DIVISION OF LAYERS

P_{on} - OVERBURDEN PRESSURE IN CENTRE OF LAYER n
e_{on} - NATURAL VOID RATIO IN CENTRE OF LAYER n
ν - OEDOMETER SETTLEMENT CORRECTION FACTOR
E - APPARENT MODULUS OF ELASTICITY

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

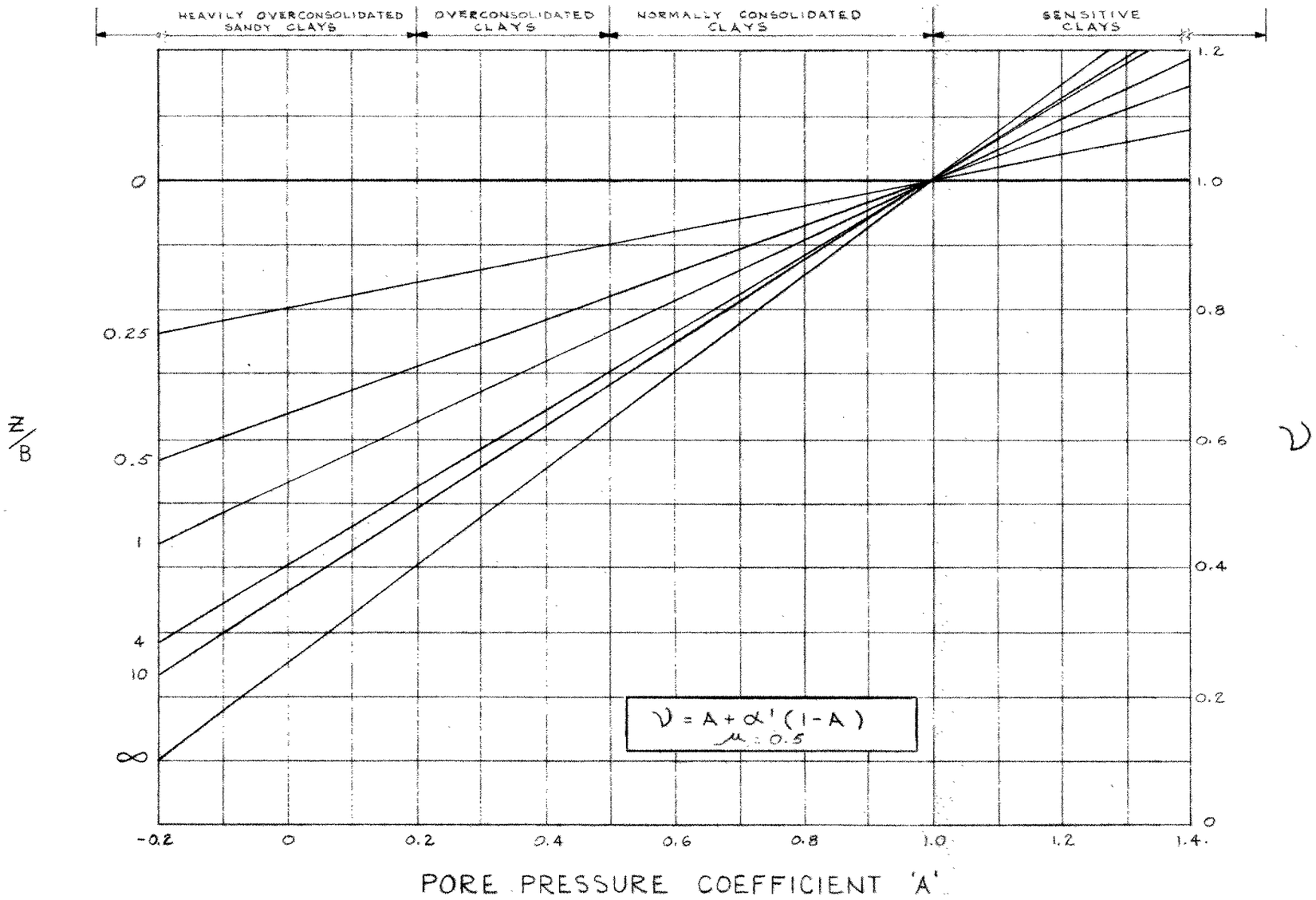
HWY. 401 & TILBURY CREEK CROSSING

FOUNDATION SOIL PROPERTIES

APPROVED	DATE: MARCH, 1960	
D. H. MacDonald	SCALE	JOB No. 837
H. G. ACRES & COMPANY LIMITED	PLATE - V	

SK-837-A-31

STRESS HISTORY & STRUCTURE OF CLAY



(a) SKEMPTON A.W. AND BJERRUM L., 1957.
"A CONTRIBUTION TO THE SETTLEMENT ANALYSIS OF FOUNDATIONS ON CLAY".
GEOTECHNIQUE, VOL. VII, PP. 1-11
(b) WOOD, M., 1959.
DISCUSSION OF SKEMPTON AND BJERRUM, 1957.
GEOTECHNIQUE, VOL. IX, PP.

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

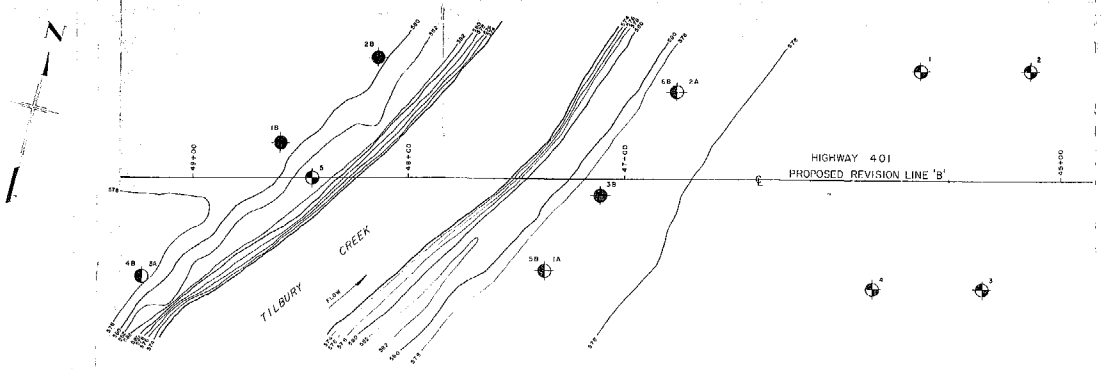
HWY. 401 & TILBURY CREEK CROSSING

OEDOMETER SETTLEMENT CORRECTION FACTOR, ν, FOR STRIP FOOTINGS

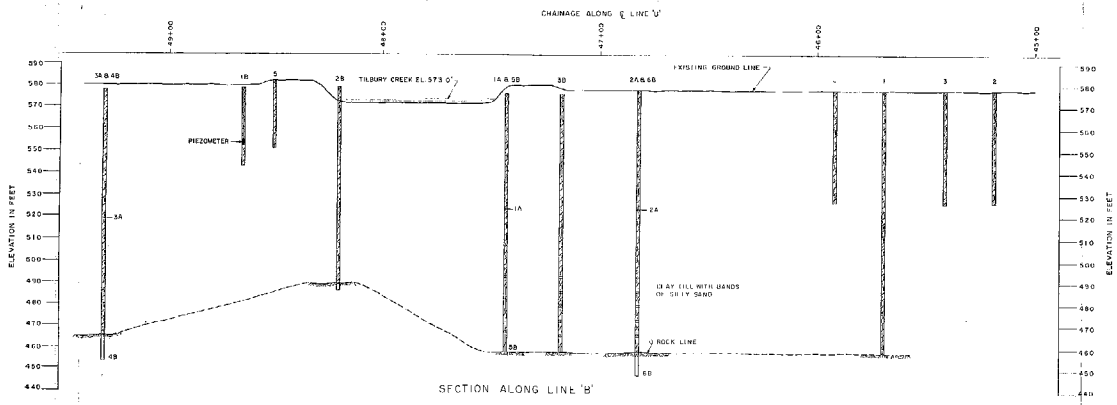
APPROVED	DATE: MARCH, 1960	
D. H. Macdonald	SCALE	JOB No. 837
H. G. ACRES & COMPANY LIMITED	PLATE - VI	

SK-837-B-32

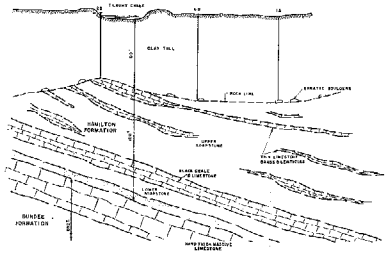
OVERSIZE DRAWING



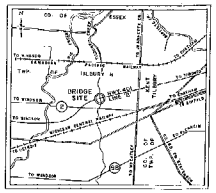
PLAN



SECTION ALONG LINE 'B'



GENERALIZED PROFILE
(NOT TO SCALE)



KEY PLAN
SCALE: 1IN = 1MI

LEGEND:

- EXPLORATORY HOLE, H. G. ACRES, SEPTEMBER 1959
- EXPLORATORY HOLE, DEPARTMENT OF HIGHWAYS, JUNE 1959
- EXPLORATORY HOLE, E.M. PETO, JANUARY 1959
- SOIL TYPE - CLAY TILL
- SOIL TYPE - SAND

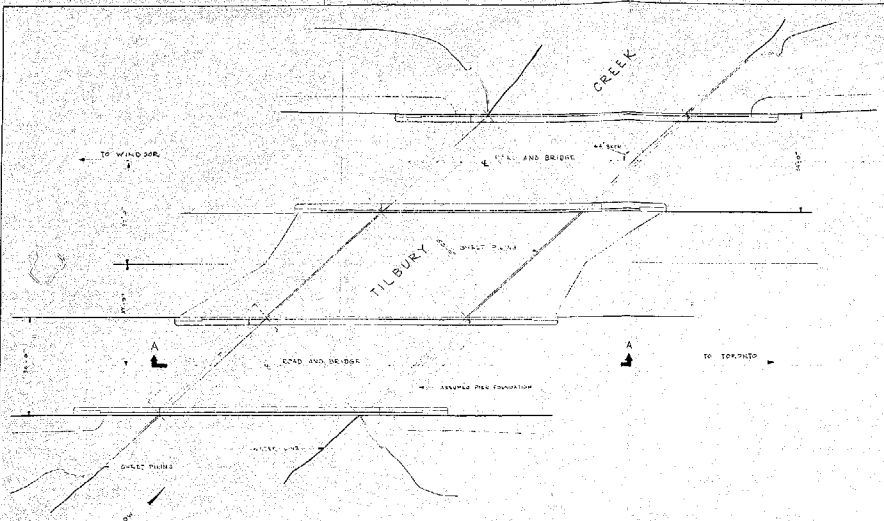
H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

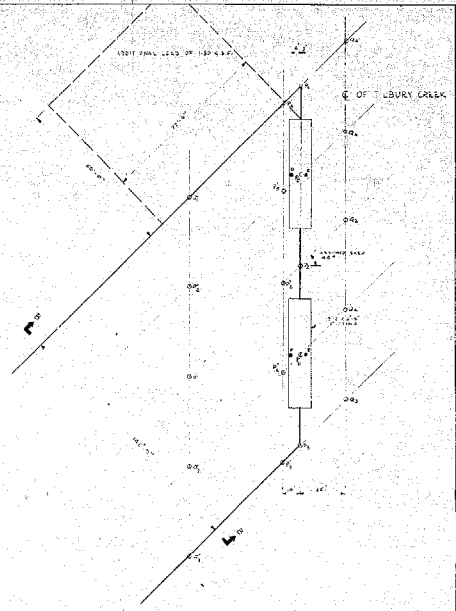
HWY. 401 & TILBURY CREEK CROSSING

EXPLORATORY HOLES
PLAN AND SECTION

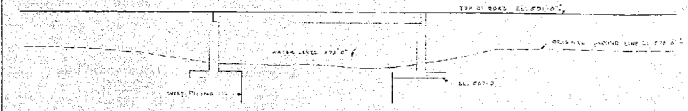
APPROVED	DATE - DEC. 11, 1959	
D. H. Brackenridge	SCALE 1 IN = 20 FT	JOB NO. 837
H. G. ACRES & COMPANY LIMITED	PLATE - 1	

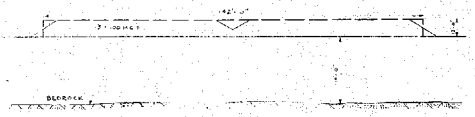


PLAN



PLAN SHOWING LOCATION OF ASSUMED LOADINGS AND SETTLEMENT POINTS



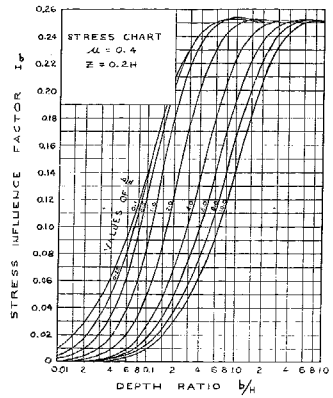


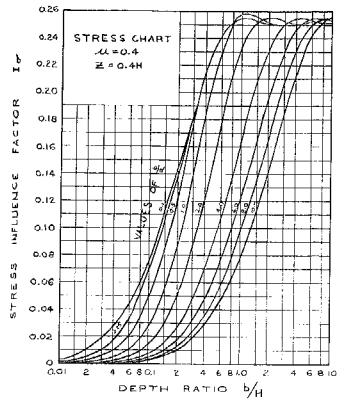
SECTION B-B SHOWING ASSUMED EMBANKMENT LOAD
(NOT TO SCALE)

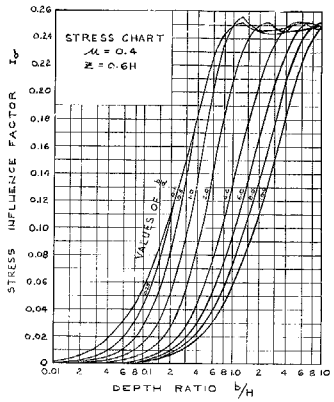
SECTION A-A

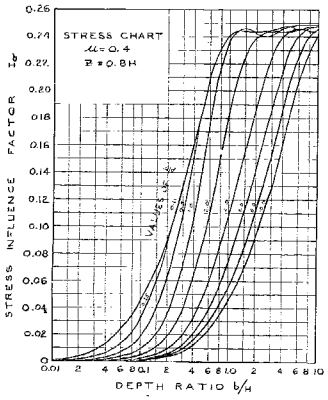
H.G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY. 401 & TILBURY CREEK CROSSING	
GENERAL PLAN AND SECTIONS LOCATION OF ASSUMED LOADINGS AND CHOSEN SETTLEMENT POINTS	
APPROVED	DATE: MARCH, 1960
D. H. Lockhart H.G. ACRES & COMPANY LIMITED	SCALE AS NOTED / JOB No. 8347
	PLATE - I

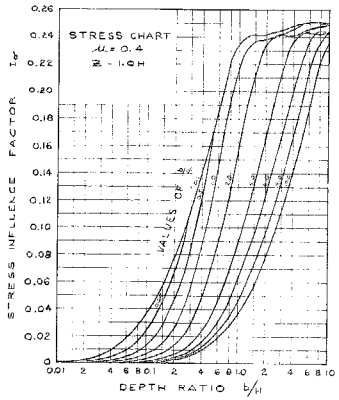
54-257-B-27

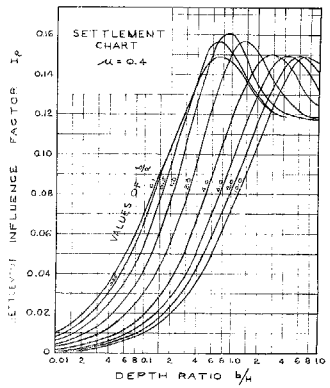


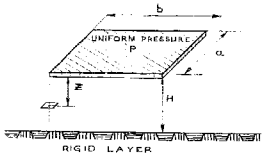












NOTES:

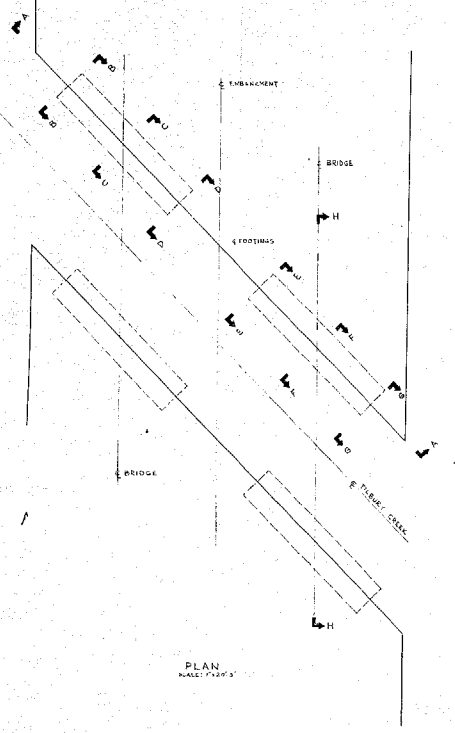
(A) VERTICAL STRESS CHANGE
$\Delta\sigma_z = I_\sigma . P$

(B) IMMEDIATE SETTLEMENT
$\rho_L = I_\rho . \frac{H}{E} . P$

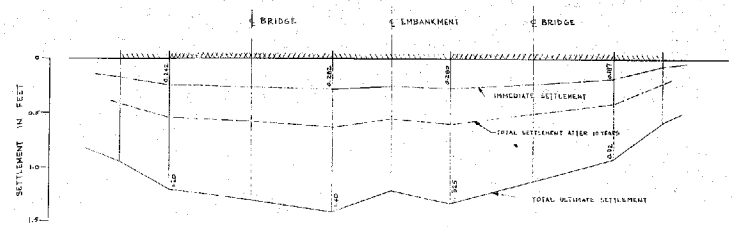
(C) REFERENCE: BURMISTER, D.M. 1956
"STRESS AND DISPLACEMENT CHARACTERISTICS OF A TWO-LAYER RIGID BASE SOIL SYSTEM: INFLUENCE DIAGRAMS AND PRACTICAL APPLICATIONS."
PROCEEDINGS; HIGHWAY RESEARCH BOARD, VOL. 35

H. G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY 401 & TILBURY CREEK CROSSING	
TWO-LAYER RIGID BASE SYSTEM INFLUENCE FACTORS FOR VERTICAL STRESS CHANGES AND VERTICAL SETTLEMENTS	
APPROVED	DATE MARCH, 1960
D. H. MacDonald	SCALE — JOB No. 837
H.G. ACRES & COMPANY LIMITED	PLATE- IV

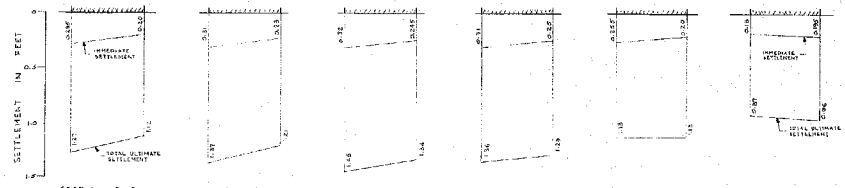
SK-837-B-10



PLAN
SCALE: 1" = 20'-0"

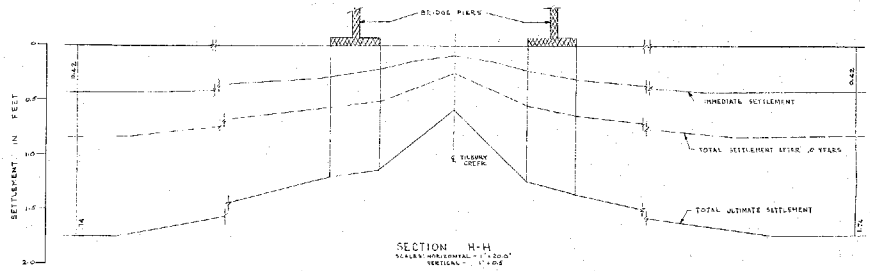


SECTION A-A
SCALES: HORIZONTAL - 1" = 20'-0"
VERTICAL - 1" = 1'-0"

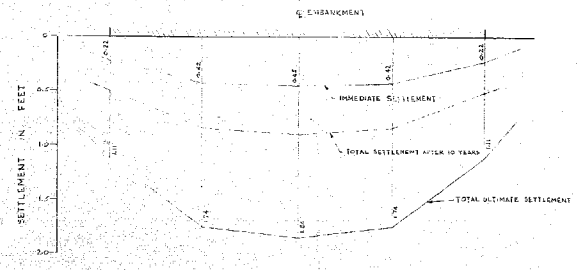


SECTION B-B | SECTION C-C | SECTION D-D | SECTION E-E | SECTION F-F | SECTION G-G

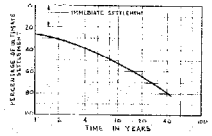
SCALES: HORIZONTAL - 1" = 20'-0"
VERTICAL - 1" = 1'-0"



SECTION H-H
SCALES: HORIZONTAL - 1" = 20'-0"
VERTICAL - 1" = 1'-0"



TYPICAL CROSS SECTION THROUGH EMBANKMENT
SCALES: HORIZONTAL - 1" = 20'-0"
VERTICAL - 1" = 1'-0"



AVERAGE TIME RATE OF SETTLEMENT

LEGEND:

BRIDGE

EMBANKMENT

NOTE:

SETTLEMENTS ARE MEASURED IN FEET.

H. G. ACRES & COMPANY LIMITED CONSULTING ENGINEERS NIAGARA FALLS, CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY. 401 & TILBURY CREEK CROSSING	
SETTLEMENTS OF BRIDGE PIERS AND EMBANKMENT	
APPROVED	DATE: MARCH, 1960
D. H. Macdonald H. G. ACRES & COMPANY LIMITED	SCALE: AS NOTED JOB No. 859
	PLATE - VII

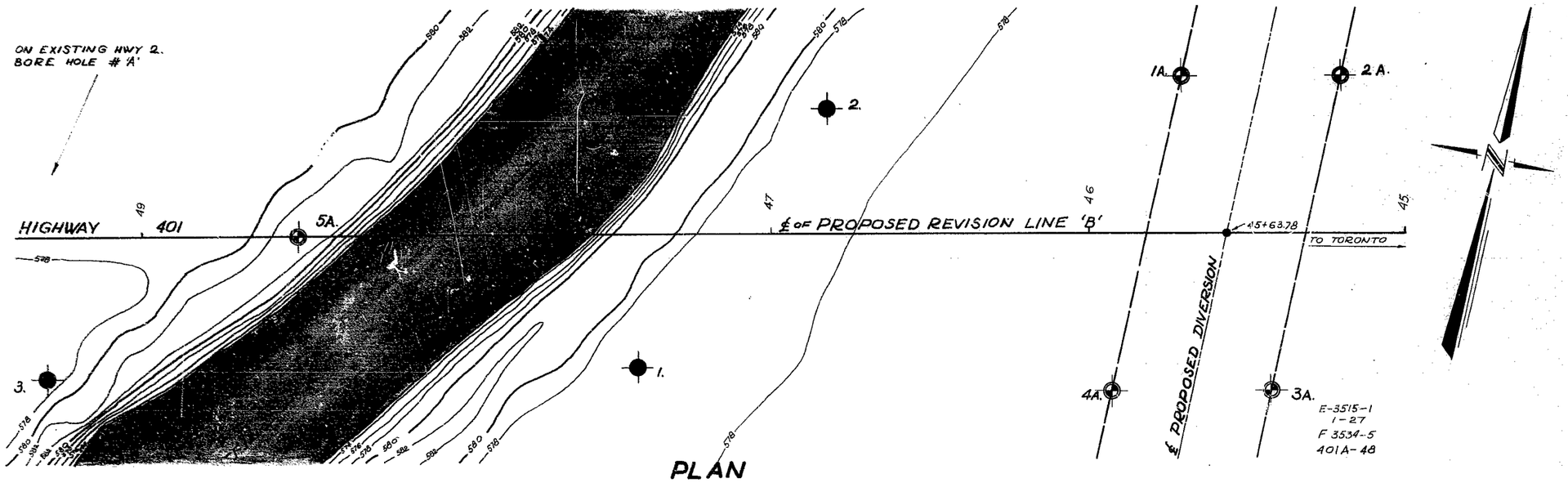
SC-837-B-33

W.P. 160-58

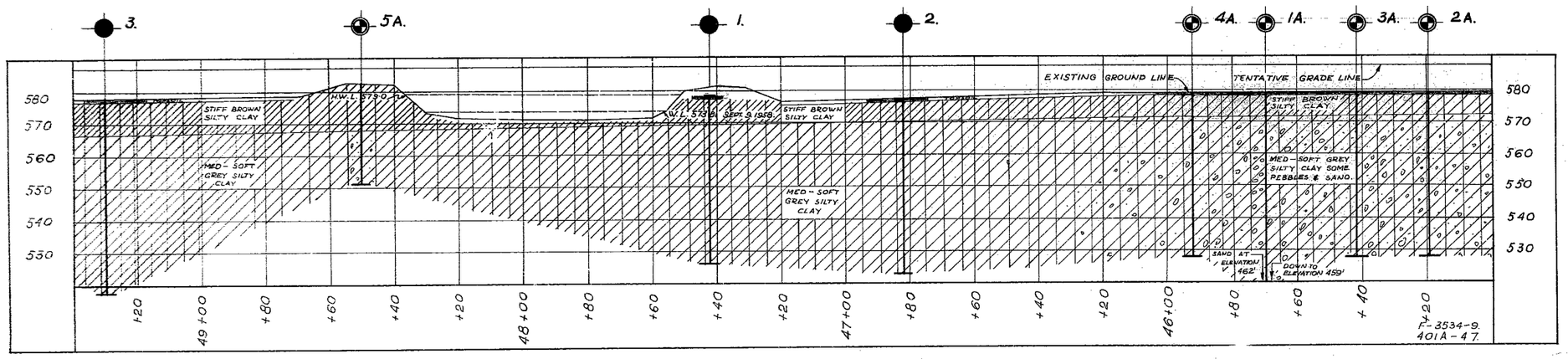
HWY. 401

TILBURY CREEK
BRIDGE

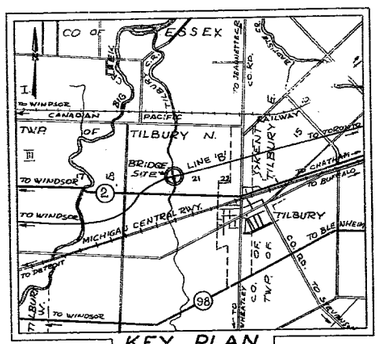
40J8-21



PLAN



PROFILE



LEGEND

BORE HOLE ●

BORE & PENETRATION HOLE DONE BY CONSULTANTS ⊕

HOLE NO	ELEVATION	STATION	DISTANCE FROM ℄
1.	578·0	47+42	42' LT.
2.	578·0	46+82	40' RT.
3.	578·0	49+30	45' LT.
1 A.	578·93	45+70	50' RT.
2 A.	579·08	45+19	50' RT.
3 A.	579·66	45+42	50' LT
4 A.	579·34	45+92	50' LT
5 A.	582·95	48+49	℄

- NOTE -

THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

40J8-21
GEOGRES No.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

TILBURY CREEK PROPOSED CROSSING

SHOWING POSITIONS & ELEVATIONS OF HOLES

HWY. 401 DISTRICT 1 COUNTY ESSEX
TOWNSHIP TILBURY NORTH LOT 20 CON. III.
LOCATION 1 MILE WEST OF TILBURY

DRAWN BY: T. Szegvary	CHECKED BY:	W.P. 160-58.
DATE 25. JUNE 1959.	APPROVED BY:	DRAWING NO. F-59-62 A.
SCALE 1" = 20'		

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS
CANADA

23-60-110

IN YOUR REPLY REFER TO
FILE 837

October 1, 1959

Materials and Research Section,
Department of Highways of Ontario,
Downsview, Ontario.

Attention: Mr. L.G. Soderman,
Principal Soils and Foundation
Engineer

Gentlemen: Department of Highways of Ontario
Tilbury Creek Bridge
Highway No. 401, District No. 1
W.P. No. 160-58

This letter will confirm your telephone conversation with Mr. C. Kenney on September 29, concerning the results of the drilling to date. These results may be summarized as follows:

Hole No.	Location	Elevation of Rock	Rock Core
2	CH 48+22, 55' R	490	5 feet
3	CH 47+35, centre-line	462	5+ feet
4	CH 46+82, 40' R	460	5+ feet

At the present time, the drillers are extending to bedrock an old exploratory hole located at CH 47+40, 45' L. When this hole is finished, the work on the east side of the river will be complete and the drill rig will be moved to the west side of the river to extend to bedrock another old exploratory hole located at CH 49+30, 45' L. The completion of this latter hole will complete the exploratory work at the bridge site, except if additional holes are required to determine more accurately the bedrock profile.

The laboratory work has now proceeded to the stage where settlement calculations can be started. The results of the calculations and further results of the drilling will be 'phoned to you as soon as they are obtained.

Yours very truly,

H.G. ACRES & COMPANY LIMITED

D. H. MacDonald

TCK/rb

D.H. MacDonald
Geotechnical Engineer
Technical Division

LCS

OCT 2 1959

cc: Foundation Section.

September 29, 1959.

De Leuw Cather & Co.,
of Canada, Ltd.,
1491 Yonge Street,
Toronto, Ontario.

Attention: Mr. H. Van Bodegon.

Dear Sir:-

As per your request while at our office this date, enclosed please find two copies of Drawing - No. F-59-62A - for the proposed Tilbury Creek Crossing, W.P. 160-58.

Yours very truly,

for L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER.

/MdeF
Encls.(2)

Karl O Soderman

File

837

September 4, 1959

Ontario Department of Highways,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. A. Rutka
A/Materials and Research Engineer

Gentlemen: Ontario Department of Highways
Tilbury Creek Bridge (WP-160-58)
Foundation Investigation

We wish to thank you for your letter of September 3, in which you authorized us to proceed with a foundation investigation at the Tilbury Creek crossing, and we should like to thank you for entrusting this work to us.

We have assigned our Job 827 to this work, and Mr. T.C. Kenney of our staff, will co-ordinate the work from this office. Mr. G. Wilson will be in charge of the work at the site and will subsequently follow the job in our office under Mr. Kenney's direction, as much as possible.

We have arranged with the F.E. Johnston Drilling Company, to carry out the drilling work and it is planned that they will arrive at the site on Tuesday, September 8. We note in your letter, your interest in obtaining quickly, information relative to the elevation and density of the sand layer encountered below the till at the site. We will forward this information to you as quickly as it is obtained.

We will forward 10 copies of the completed report to you, and in accordance with your request, we shall bill you at the end of the job, unless it should cover an extended period, in which case we would send monthly invoices to you.

We trust that these arrangements are satisfactory

RECEIVED SEP 9 1959 SECTION 75

to you. If there should be any additional requirements, please do not hesitate to contact either Mr. Kenney, or myself.

Yours very truly,

H.G. ACRES & COMPANY LIMITED

D.H. MacDonald

D.H. MacDonald
Geotechnical Engineer
Technical Division

DHM:jf
c.c. Mr. L. Soderman

RECEIVED
SEP 9 1959
SECTION 75

ONTARIO

DEPARTMENT OF HIGHWAYS

Toronto 5,
April 3, 1959.

MEMORANDUM TO:

Mr. L. G. Soderman,
Principal Soils & Foundation Engineer,
Downsview, Ontario.

Re: W.P. 160-58, Tilbury N. Twp. Bridge #2,
(Tilbury Creek), Hwy. #401,
District #1.

Your memo of 13th March, 1959 gave your predictions on the settlement and differential movement of the above proposed structure.

An extension to the bridge over Tilbury Creek on Hwy. #2 was built in 1930. This structure is about $\frac{1}{4}$ mile upstream of the proposed crossing and about $\frac{1}{4}$ mile West of Tilbury.

The bridge is a steel beam, simple span of about 37'. The abutments are gravity type and from the existing drawings the dimensions of the footings are 5'-9" wide, 4 feet deep with the top of footing about 1 foot below bed of stream. Photographs taken in 1957 show no indication of settlement or differential movement.

If soil conditions were the same at both the existing bridge and at the proposed crossing some further light might be shed on the matter. This could possibly be carried still further by investigating the Michigan Central bridge upstream and the C.P. bridge downstream from the proposed site.

J. C. McAllister,
for S. McCombie,
Bridge Planning Engineer.

JCM:bh

Mr. A. M. Toye,

Bridge Engineer.

Materials & Research Section.

March 13, 1959.

Re: Tilbury Creek Crossing - W.P. 160-58. - Review of Settlement Predictions.

Attention: Mr. S. McCombie.

At the request of Mr. H. Van Bodegon of de Leuw Cather, Ltd., we have reviewed the proposed footing design for the above creek crossing.

This review was based upon the following data:-

(1) Hollow type skew abutment with footing dimensions of 22' x 54'. Average net footing pressure of 1660 p.s.f. with base of footing founded at elev. 567.0.

(2) Subsoil conditions as given in foundation report submitted by E. M. Pete and Associates, dated February 3, 1959.

(3) Average fill height of 12 feet at the abutment locations. Fill placement density assumed at 130 p.c.f.

(4) Ground water elevation taken at elev. 576.0.

(5) Stress distribution based upon elastic theory using the classical Bousinesq equations.

The results of the settlement analysis are summarized as follows:-

(1) Total settlement of abutment due to average net footing pressure of 1660 p.s.f. found to be 7 inches (rigidity corrections have been applied to this value.)

(2) Additional total settlement of abutment due to influence of embankment fill found to be 6 inches.

(3) The combined settlement of abutment and embankment at the abutment location will be approximately 12 inches (± 20%).

(2)

cont'd. /2 ...

Results of settlement analysis: (cont'd.) ...

(4) The differential movement obtained from calculating settlements at the toe and heel of the abutment footing was found to be approximately 5 inches. This will result in a tilting of the abutment.

(5) A study of the time rate at which settlement will take place indicates that an estimated 50% of the predicted movement will take place in the five years immediately following completion of construction.

The stress patterns set up in subsurface layers due to skew footing shapes, are not subject to rigorous analysis. The use of spread footings at this site would contribute a good deal to our methods of settlement predictions, provided of course, that time settlement observations were carried out during and after construction.

Installation of settlement pins and permanent adjacent bench marks could be carried out at very little additional expense during construction. We recommend that this be done; the Materials and Research Section could supervise the installation of these control elements and make arrangements to have necessary readings of settlement taken.

If further discussion is required in connection with proposed design or instrumentation procedure, please contact our Office.

A. Rutka,
ACTING MAT'LS. & RESEARCH ENGR.
per:

L. GS

(L. G. Soderman,
PRINCIPAL SOILS & FOUNDATION ENGR.)

LGS/MdeF

cc: Messrs. H. A. Tregaskes
H. D. McMillan
de Leuw Cather & Co of Canada.
L. G. Soderman

File

Mr. J. McAllister,

Bridge Office.

Materials & Research Section.

October 21, 1959.

Re: Boring Results -
Tilbury Creek Crossing,
W.P. 160-58.

Enclosed herewith are the results of borings carried out by H. G. Acres & Co., Ltd., at the above noted Tilbury Creek Crossing. These borings were required to positively determine the elevation of the underlying bedrock formation which is to be used as the bearing stratum for small displacement 'H' piles.

It has been decided that support for the footings for piers and abutments can best be provided for by the use of piles founded on the underlying bedrock. The problem of settlements that can be expected with the use of ordinary spread footings, or a box type footing, is now being analyzed by H. G. Acres under the direction of Dr. D. H. MacDonald, who is an expert in this type of problem. The results of Dr. MacDonald's analyses will not affect the choice of foundation support for this structure but, rather, will be used for other structures in the Tilbury area, where the skews are not as extreme as that of this particular structure site.

A formal report from H. G. Acres will be submitted to your office and discussed with you in detail, immediately it is received by our office.

LGS/MdeF
Encls.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 1-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 11:30 a.M. September 9 1959; FINISHED 5:00 p.M. September 11 1959

METHOD OF DRILLING: SOIL Wash Boring — CASING DIAM. 4 inch; ROCK -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 48+65; DEPARTURE 160 Ft Right; BEARING -; INITIAL DIP 90 degrees; OTHER DIPS -

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM -; GROUND SURFACE 579.0; ROCK SURFACE -; BOTTOM OF HOLE 544.0; WATER TABLE 571.3

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
	Topsoil							
2.0								
	Clay till	Grey						
35.0								
		Note: Carborundum piezometer tip installed 26 feet below ground surface						

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR G. Wilson

LOGGED BY W. Wilson

APPROVED BY

DATE

PRELIMINARY

H. G. A[illegible]S & COMPANY LIMITED — CONS[illegible]ING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Highways of Ontario — JOB No. 837

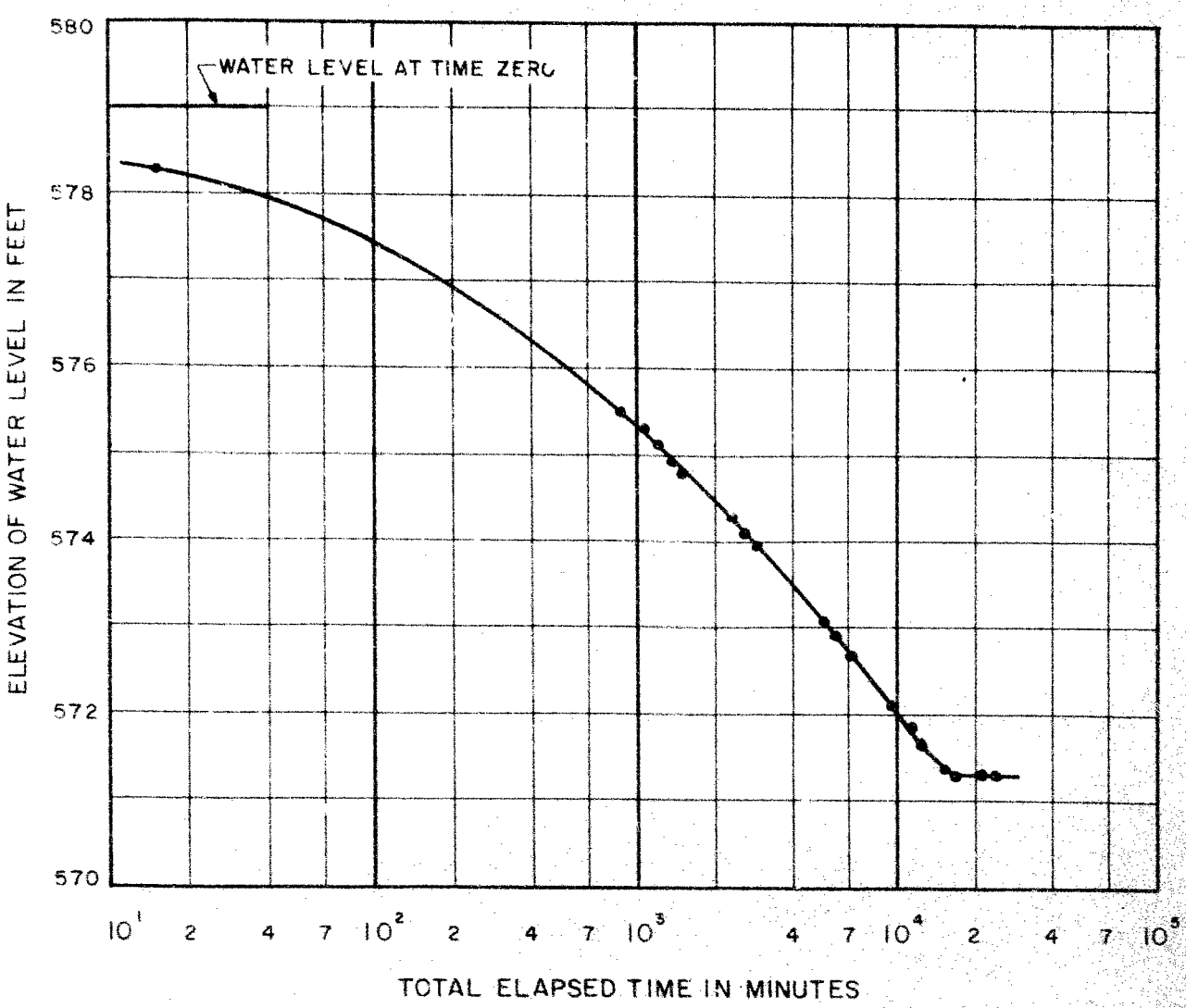
PROJECT Hwy 401 and Tilbury Creek Crossing — HOLE No. 1-B

SITE Tilbury Creek, Ontario — SHEET No. 2 OF 2

Date	Time	Total Elapsed Time in Minutes	Elevation of Water Level in Piezometer Tube
September 14, 1959	1730	0	578.97
September 14, 1959	1745	15	578.26
September 15, 1959	800	870	575.51
September 15, 1959	1100	1,050	575.30
September 15, 1959	1315	1,185	575.09
September 15, 1959	1535	1,325	574.91
September 15, 1959	1728	1,438	574.78
September 16, 1959	745	2,295	574.28
September 16, 1959	1200	2,550	574.12
September 16, 1959	1730	2,880	573.97
September 18, 1959	815	5,205	573.09
September 18, 1959	1730	5,760	572.89
September 19, 1959	730	6,600	572.68
September 21, 1959	800	9,510	572.15
September 22, 1959	1200	11,190	571.84
September 23, 1959	800	12,390	571.66
September 25, 1959	730	15,270	571.39
September 26, 1959	700	16,650	571.30
September 29, 1959	800	21,030	571.34
September 30, 1959	800	22,470	571.32
October 1, 1959	1200	24,150	571.34

NOTE: Ground surface elevation 578.97

PRELIMINARY



NOTE:

MEASUREMENTS WERE STARTED ON SEPT. 14, 1959 AT 1730 HOURS.

WATER LEVEL AT TIME ZERO WAS EL. 578.97 FT.

PRELIMINARY

H. G. ACRES & COMPANY, LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS CANADA

DEPARTMENT OF HIGHWAYS OF ONTARIO

HWY. 401 AND TILBURY CREEK CROSSING

ELEVATIONS OF WATER LEVEL IN PIEZOMETER TUBE, HOLE 18

APPROVED	DATE: OCT. 7, 1959	
	SCALE	JOB No. 837
	PLATE	

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 337

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 2-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 5:00 p.m. September 11 19 59; FINISHED 2:30 p.m. September 19 19 59

METHOD OF DRILLING: SOIL Wash Boring — CASING DIAM. 4 inch; ROCK Diamond drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 48+ 20; DEPARTURE 55.0 Ft Right; BEARING -; INITIAL DIP 90 degrees; OTHER DIPS -

ELEVATIONS: DATUM C.S.C.; DRILL PLATFORM -; GROUND SURFACE 579.4; ROCK SURFACE 490.6; BOTTOM OF HOLE 485.6; WATER TABLE -

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0					Inches	Feet	Feet	
	Topsoil	Dark grey to black, very dry with shrinkage cracks, contains well rounded pebbles and occasional boulders	1	CO	3	5.0	1.0	Tapped
				Vane	Test	7.0		Tapped
				Vane	Test	8.0		Tapped
2.0			2	CO	3	10.0	1.5	Tapped
	Clay till	Brown with grey streaks, weathered, fissured, but not stratified, contains angular pebbles		Vane	Test	13.5		Tapped
				Vane	Test	15.0		Pushed
14.0			3	CO	3	15.0	1.5	Pushed
	Clay till	Grey with brown streaks, stiff, weathered, not stratified, contains angular pebbles, water content increasing with depth		Vane	Test	18.5		Pushed
			4	CO	3	20.0	1.5	Pushed
				Vane	Test	23.5		Pushed
20.0				Vane	Test	24.5		Pushed
	Clay till	Grey with brown streaks, firm, slightly weathered, not stratified, contains angular pebbles	5	CO	3	25.0	1.5	Tapped
			6	CO	3	28.0	1.5	Pushed
28.0								
	Clay till	Grey, unweathered, firm but remoulds to soft clay, contains pebbles as above		Vane	Test	30.0		Pushed
			7	CO	3	31.5	1.5	Pushed

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED:

DATE:

PRELIMINARY

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 2-B

SITE: Tilbury Creek, Ontario — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	TYPE	SIZE Inches	DEPTH Feet	REC'D Feet	PENETRATION
35.0								
	Clay till	Grey, slightly bluish in patches, soft, contains pebbles as above	8	CO	3	36.5	1.5	Pushed
				Vane	Test	40.0		Pushed
50.0				Vane	Test	45.0		Pushed
	Clay till	Grey, firm but remoulds to soft clay with pebbles as above	9	CO	3	46.5	1.5	Pushed
89.0				Vane	Test	50.0		Pushed
				Vane	Test	55.0		Pushed
	Bedrock	Grey hard fossiliferous limestone with thin hard brown shale bands, nodules of pyrite. Dip of bedding varies from horizontal to 20 degrees. Rock is not weathered.	10	CO	3	56.5	1.5	Pushed
				Vane	Test	60.0		Pushed
				Vane	Test	62.5		Pushed
94.0			11	CO	3	64.0	1.5	Pushed
	Note:	No noticeable water loss or gain during drilling for full depth of hole. Hole quickly filled with water after casing was withdrawn. This might possibly be due to water flowing in from fissured zone near surface.		Vane	Test	67.5		Pushed
				Vane	Test	70.0		Pushed
			12	CO	3	71.5	1.5	Pushed
				Vane	Test	75.0		Pushed
				Vane	Test	80.0		Pushed
			13	BO	3	81.6	1.5	Pushed
				Vane	Test	85.0		Pushed
				Vane	Test	89.0		Pushed

PRELIMINARY

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: H wy 401 and Tilbury Creek Crossing — HOLE No. 3-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 7:00 a.M. September 21 1959; FINISHED 5:30 P.M. September 25 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. 4 inch; ROCK: -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 47 + 17; DEPARTURE 8.0 Feet Left; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM --; GROUND SURFACE 576.9; ROCK SURFACE 458.9; BOTTOM OF HOLE 458.9; WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION ~~TEST~~
					Inches	Feet	Feet	
0	Topsoil	Dark grey, very stiff, fissured		Vane	Test	5.8		Pushed
2.0	Clay till	Brown, very stiff, weathered, fissured, not stratified, contains angular pebbles	14	BO	3	6.5	1.5	9" only Pushed
				Vane	Test	10.0		Pushed
				Vane	Test	15.0		Pushed
13.0	Clay till	Grey with some brown streaks, firm, not stratified	15	CO	3	16.5	1.5	Pushed
				Vane	Test	20.0		Pushed
35.0	Clay till	Grey, firm to soft, not stratified, contains angular pebbles		Vane	Test	25.0		Pushed
			16	CO	3	26.5	1.5	Pushed
83.0		Black compact with coarse sand, fine sand, silt and clay. May be stratified with firm and soft bands. No water loss or gain. Soil becoming softer with depth		Vane	Test	30.0		Pushed
				Vane	Test	35.0		Pushed
			17	CO	3	36.6	1.5	Pushed
				Vane	Test	40.0		Pushed
				Vane	Test	45.0		Pushed
118.0	Rock		18	CO	3	46.5	1.5	Pushed
				Vane	Test	50.0		Pushed
				Vane	Test	50.0		Pushed

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
— PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED:

DATE:

PRELIMINARY

H. G. A[illegible]ES & COMPANY LIMITED — CONS[illegible]TING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Highways of Ontario — JOB No. 837

PROJECT Hwy 401 and Tilbury Creek Crossing — HOLE No. 3-B

SITE Tilbury Creek, Ontario — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE	SIZE Inches	DEPTH Feet	RET'D Feet	PENETRATION ~~Feet~~
	Note:	At depth of 118 feet, struck very hard formation which was either bedrock or boulder lying on bedrock	19	CO	3	56.5	1.5	Pushed
				Vane	Test	60.0		Pushed
				Vane	Test	65.0		Pushed
		Very strong natural gas supply blew all water from the casing Pressure dropped gradually with time but further drilling was made impossible	20	CO	3	66.5	1.5	Pushed
				Vane	Test	70.0		Pushed
				Vane	Test	75.0		Pushed
		No water was lost or gained	21	BO	3	76.5	1.5	Pushed
		Hole was sealed with cement and soil		Vane	Test	83.0		Tapped but no penetration
			22	AR	2	83.0 to 84.5	0.5	92 blows 140-lb hammer falling 30"

PRELIMINARY

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 4-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 2:00 p.m. September 30 1959; FINISHED 3:00 p.m. October 1 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. AX; ROCK: Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 49+ 30; DEPARTURE 45.0 Feet Left; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM C.S.C.; DRILL PLATFORM --; GROUND SURFACE 577.4; ROCK SURFACE 464.4; BOTTOM OF HOLE 452.4; WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
	Previously drilled hole by Department of Highways - Hole No. 3							
60.0								
	Clay till	Grey, firm to soft, becoming siltier towards the bottom, contains angular pebbles						
113.0								
	Bedrock	Soft calcareous shale with limestone bands and nodules. No water gain or loss. Practically no core recovery						
125.0								

SAMPLING METHOD
* A – SPLIT TUBE
B – THIN WALL TUBE
C – PISTON SAMPLER
D – CORE BARREL
E – AUGER
F – WASH

SHIPPING CONTAINER
N – INSERT
O – TUBE
P – WATER CONTENT TIN
Q – GLASS JAR
R – CLOTH BAG
S – PLIOFILM BAG
Z – DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED

DATE

PRELIMINARY

H. G. [illegible]RES & COMPANY LIMITED — CO[illegible]LTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 83 7

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 5-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1.

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 3:00 pm. September 29 1959; FINISHED 1:30 pm. September 30 1959

METHOD OF DRILLING: SOIL Wash Boring — CASING DIAM. AX; ROCK — — CORE DIAM. —

LOCATION: ~~LATITUDE~~ CH 47+ 42; DEPARTURE 42.0 Feet Left; BEARING —; INITIAL DIP 90 degrees; OTHER DIPS —

ELEVATIONS: DATUM C.S.C.; DRILL PLATFORM —; GROUND SURFACE 576.8; ROCK SURFACE 459.8; BOTTOM OF HOLE 459.8; WATER TABLE —

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
		Previously drilled hole by Department of Highways - Hole No. 1						
55.0								
	Clay till	Grey, firm to soft, contains angular pebbles						
83.0								
		Compact sand, silt and black earthy material No water loss or gain						
86.0								
	Clay	Brown, soft and plastic						
117.0								
	Rock	Struck bedrock or boulder lying on bedrock						

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED

DATE

PRELIMINARY

NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 6-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 2:00 p.m. September 26 1959; FINISHED 5:30 p.m. September 28 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. AX; ROCK: Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 46+82; DEPARTURE 40.0 Feet Right; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM --; GROUND SURFACE 576.9; ROCK SURFACE 457.9; BOTTOM OF HOLE 446.9; WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
		Previously drilled hole by Department of Highways - Hole No. 2						
55.0								
	Clay till	Grey, contains angular pebbles						
8[illegible].0								
		Fine silty sand and clay No water loss or gain No gas struck						
119.0								
	Bedrock	6-inch gneiss boulder lying on top of soft calcareous shale with fossils and interbedded with thin hard shales and limestone lenticles						
130.0								

SAMPLING METHOD
- * A – SPLIT TUBE
- B – THIN WALL TUBE
- C – PISTON SAMPLER
- D – CORE BARREL
- E – AUGER
- F – WASH

SHIPPING CONTAINER
- N – INSERT
- O – TUBE
- P – WATER CONTENT TIN
- Q – GLASS JAR
- R – CLOTH BAG
- S – PLIOFILM BAG
- Z – DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED

DATE

PRELIMINARY

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS
CANADA

IN YOUR REPLY REFER TO
FILE 837

October 15, 1959

Materials and Research Section,
Department of Highways of Ontario,
Downsview, Ontario.

Attention: Mr. L.G. Soderman
Principal Soils and Foundation
Engineer

Gentlemen: Department of Highways of Ontario
Tilbury Creek Bridge
Highway No. 401, District No. 1
W.P. No. 160-58
Drilling Reports

We are enclosing ten copies each of the drilling reports for the six holes, which were drilled at the site of the Tilbury Creek Bridge. Hole No. 1B was used to install a piezometer, holes Nos. 2B and 3B were used to obtain soil samples and to perform vane tests, and holes Nos. 4B, 5B, and 6B are extensions of the Department of Highways' holes Nos. 3, 1, and 2 respectively, and were used to establish the elevation of bedrock.

In holes Nos. 3B, 4B, 5B, and 6B, bedrock was encountered at approximately elevation 460 feet. In hole No. 2B rock was encountered at elevation 490 feet and a 5-foot length of excellent core was obtained. Since this rock was horizontally bedded limestone, it was initially assumed to be bedrock. However, from the results of the succeeding holes, there remains an element of doubt concerning this result.

With the exception of hole No. 2B, the rock which was encountered was of the Hamilton formation. This formation extends to thickness greater than 200 feet and overlies Norfolk limestone. The Hamilton formation consists of a succession of soft fossiliferous shale and thin limestone beds. Very

little core recovery was realized in this rock. To obtain some idea of its possible properties, a sample was obtained from a surface outcrop approximately 50 miles from the site. This sample has the appearance of a heavily overconsolidated clay; the degree to which it has been weathered is unknown and for this reason its properties may or may not represent the unexposed rock. However, from the fact that there was poor core recovery in this rock, it may be concluded that it is probable that pile penetration into the rock will occur and that this penetration may be variable due to the variability of the rock.

These data are of a preliminary nature and will be expanded upon in our final report.

Yours very truly,

H.G. ACRES & COMPANY LIMITED

D. H. MacDonald.

D.H. MacDonald
Geotechnical Engineer
Technical Division

TCK:pt
Encls.

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 1-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 11:30 a.m. September 9 1959; FINISHED 5:00 p.m. September 11 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. 4 inch; ROCK: -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 48 + 65; DEPARTURE 160 Ft Right; BEARING -; INITIAL DIP 90 degrees; OTHER DIPS -

ELEVATIONS: DATUM C.S.C.; DRILL PLATFORM -; GROUND SURFACE 579.0; ROCK SURFACE -; BOTTOM OF HOLE 544.0; WATER TABLE 571.3

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE*	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
	Topsoil							
2.0								
	Clay till	Grey						
35.0								
		Note: Carborundum piezometer tip installed 26 feet below ground surface						

SAMPLING METHOD
* A – SPLIT TUBE
B – THIN WALL TUBE
C – PISTON SAMPLER
D – CORE BARREL
E – AUGER
F – WASH

SHIPPING CONTAINER
N – INSERT
O – TUBE
P – WATER CONTENT TIN
Q – GLASS JAR
R – CLOTH BAG
S – PLIOFILM BAG
Z – DISCARDED

INSPECTOR G. Wilson

LOGGED BY G. Wilson

APPROVED

DATE

PRELIMINARY

H. G. ACR[illegible] & COMPANY LIMITED — CONSUL[illegible]NG ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Department of Highways of Ontario — JOB No. 837

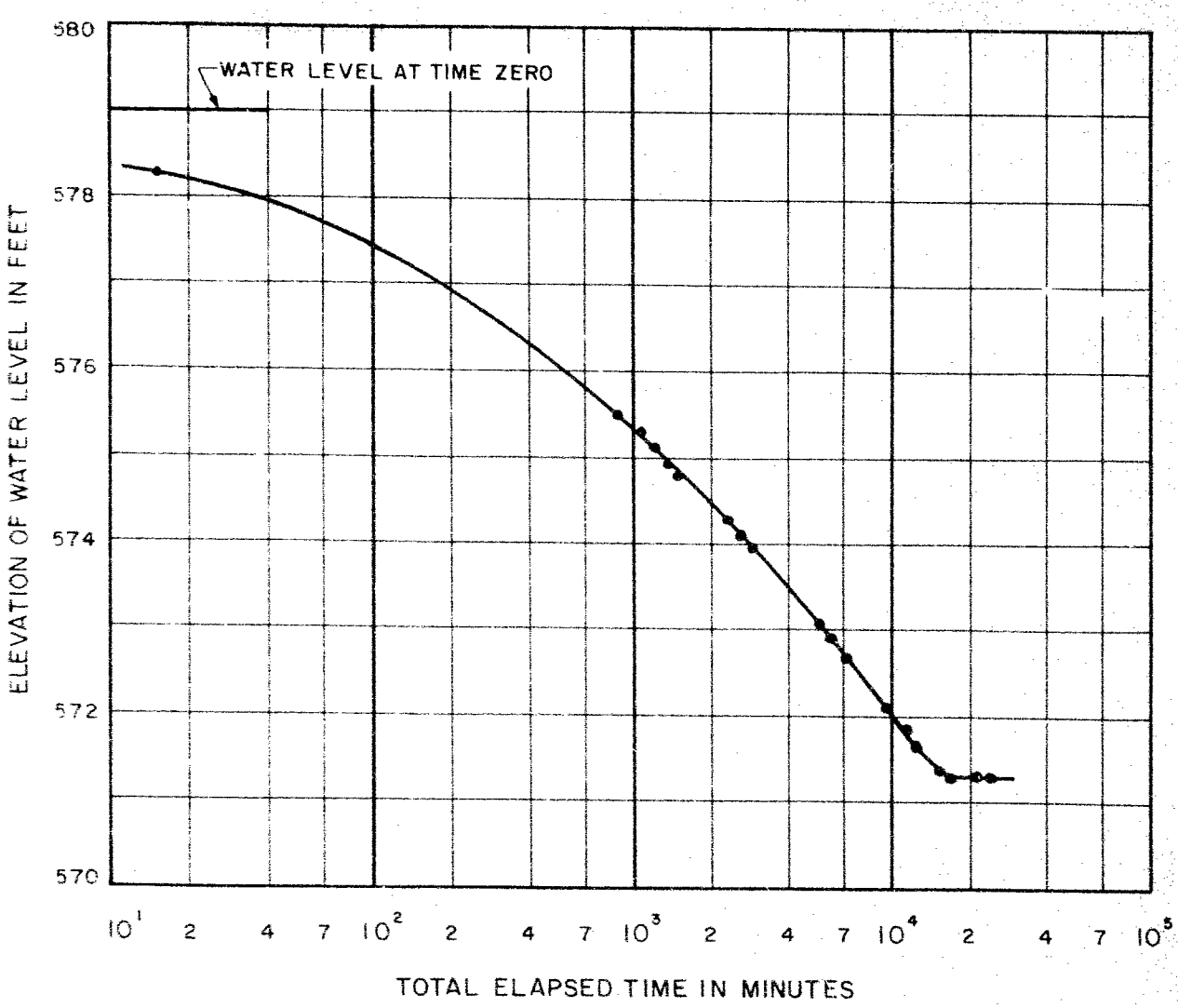
PROJECT Hwy 401 and Tilbury Creek Crossing — HOLE No. 1-B

SITE Tilbury Creek, Ontario — SHEET No. 2 OF 2

Date	Time	Total Elapsed Time in Minutes	Elevation of Water Level in Piezometer Tube
September 14, 1959	1730	0	578.97
September 14, 1959	1745	15	578.26
September 15, 1959	800	870	575.51
September 15, 1959	1100	1,050	575.30
September 15, 1959	1315	1,185	575.09
September 15, 1959	1535	1,325	574.91
September 15, 1959	1728	1,438	574.78
September 16, 1959	745	2,295	574.28
September 16, 1959	1200	2,550	574.12
September 16, 1959	1730	2,880	573.97
September 18, 1959	815	5,205	573.09
September 18, 1959	1730	5,760	572.89
September 19, 1959	730	6,600	572.68
September 21, 1959	800	9,510	572.15
September 22, 1959	1200	11,190	571.84
September 23, 1959	800	12,390	571.66
September 25, 1959	730	15,270	571.39
September 28, 1959	700	16,650	571.30
September 29, 1959	800	21,030	571.34
September 30, 1959	800	22,470	571.32
October 1, 1959	1200	24,150	571.34

NOTE: Ground surface elevation 578.9[illegible]

PRELIMINARY



NOTE

MEASUREMENTS WERE STARTED ON SEPT 14, 1959 AT 1730 HOURS

WATER LEVEL AT TIME ZERO WAS EL. 578.97 FT.

PRELIMINARY

H. G. ACRES & COMPANY, LIMITED CONSULTING ENGINEERS NIAGARA FALLS CANADA	
DEPARTMENT OF HIGHWAYS OF ONTARIO	
HWY. 401 AND TILBURY CREEK CROSSING	
ELEVATIONS OF WATER LEVEL IN PIEZOMETER TUBE, HOLE 1B	
APPROVED	DATE: OCT. 7, 1959
	SCALE / JOB No. 837
H.G. ACRES & COMPANY LTD.	PLATE -

H. G. A[illegible]S & COMPANY LIMITED — CONSU[illegible]TING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 2-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 5:00 P.M. September 11 19 59; FINISHED 2:30 P.M. September 19 19 59

METHOD OF DRILLING: SOIL Wash Boring — CASING DIAM. 4 inch; ROCK Diamond drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 48+ 20; DEPARTURE 55.0 Ft Right; BEARING -; INITIAL DIP 90 degrees; OTHER DIPS -

ELEVATIONS: DATUM C.S.C.; DRILL PLATFORM -; GROUND SURFACE 579.4; ROCK SURFACE 490.6; BOTTOM OF HOLE 485.6; WATER TABLE -

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0					Inches	Feet	Feet	
	Topsoil	Dark grey to black, very dry with shrinkage cracks, contains well rounded pebbles and occasional boulders	1	CO	3	5.0	1.0	Tapped
				Vane	Test	7.0		Tapped
				Vane	Test	8.0		Tapped
2.0			2	CO	3	10.0	1.5	Tapped
	Clay till	Brown with grey streaks, weathered, fissured, but not stratified, contains angular pebbles		Vane	Test	13.5		Tapped
				Vane	Test	15.0		Pushed
14.0			3	CO	3	15.0	1.5	Pushed
	Clay till	Grey with brown streaks, stiff, weathered, not stratified, contains angular pebbles, water content increasing with depth		Vane	Test	18.5		Pushed
			4	CO	3	20.0	1.5	Pushed
				Vane	Test	23.5		Pushed
20.0				Vane	Test	24.5		Pushed
	Clay till	Grey with brown streaks, firm, slightly weathered, not stratified, contains angular pebbles	5	CO	3	25.0	1.5	Tapped
			6	CO	3	28.0	1.5	Pushed
28.0								
	Clay till	Grey, unweathered, firm but remoulds to soft clay, contains pebbles as above		Vane	Test	30.0		Pushed
			7	CO	3	31.5	1.5	Pushed

SAMPLING METHOD
* A – SPLIT TUBE
B – THIN WALL TUBE
C – PISTON SAMPLER
D – CORE BARREL
E – AUGER
F – WASH

SHIPPING CONTAINER
N – INSERT
O – TUBE
P – WATER CONTENT TIN
Q – GLASS JAR
R – CLOTH BAG
S – PLIOFILM BAG
Z – DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED

DATE

PRELIMINARY

H. G. AC[illegible] & COMPANY LIMITED — CONSU[illegible]NG ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 2-B

SITE: Tilbury Creek, Ontario — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR, CONSISTENCY, STRUCTURE WATER CONTENT, PLASTICITY, COMPACTNESS WATER LOSS OR GAIN, ETC.	SAMPLE NO	TYPE	SIZE Inches	DEPTH Feet	RET'D Feet	PENETRATION
35.0								
	Clay till	Grey, slightly bluish in patches, soft, contains pebbles as above	8	CO	3	36.5	1.5	Pushed
				Vane	Test	40.0		Pushed
				Vane	Test	45.0		Pushed
50.0								
	Clay till	Grey, firm but remoulds to soft clay with pebbles as above	9	CO	3	46.5	1.5	Pushed
				Vane	Test	50.0		Pushed
89.0				Vane	Test	55.0		Pushed
	Bedrock	Grey hard fossiliferous limestone with thin hard brown shale bands, nodules of pyrite Dip of bedding varies from horizontal to 20 degrees. Rock is not weathered.	10	CO	3	56.5	1.5	Pushed
				Vane	Test	60.0		Pushed
				Vane	Test	62.5		Pushed
			11	CO	3	64.0	1.5	Pushed
94.0								
		Note: No noticeable water loss or gain during drilling for full depth of hole.		Vane	Test	67.5		Pushed
				Vane	Test	70.0		Pushed
		Hole quickly filled with water after casing was withdrawn. This might possibly be due to water flowing in from fissured zone near surface.	12	CO	3	71.5	1.5	Pushed
				Vane	Test	75.0		Pushed
				Vane	Test	80.0		Pushed
			13	BO	3	81.6	1.5	Pushed
				Vane	Test	85.0		Pushed
				Vane	Test	89.0		Pushed

PRELIMINARY

H. G. A[illegible]S & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: H wy 401 and Tilbury Creek Crossing — HOLE No. 3-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 7:00 a.M. September 21 1959; FINISHED 5:30 p.M. September 25 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. 4 inch

ROCK: -- — CORE DIAM. --

LOCATION: ~~LATITUDE~~ CH 47 + 17; DEPARTURE 8.0 Feet Left; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM --; GROUND SURFACE 576.9; ROCK SURFACE 458.9; BOTTOM OF HOLE 458.9; WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	TYPE*	SIZE	DEPTH	RET'D	PENETRATION ~~FEET~~
					Inches	Feet	Feet	
0	Topsoil	Dark grey, very stiff, fissured		Vane	Test	5.8		Pushed
2.0	Clay till	Brown, very stiff, weathered, fissured, not stratified, contains angular pebbles	14	BO	3	6.5	1.5	9" only Pushed
				Vane	Test	10.0		Pushed
				Vane	Test	15.0		Pushed
13.0	Clay till	Grey with some brown streaks, firm, not stratified	15	CO	3	16.5	1.5	Pushed
				Vane	Test	20.0		Pushed
35.0	Clay till	Grey, firm to soft, not stratified, contains angular pebbles		Vane	Test	25.0		Pushed
			16	CO	3	26.5	1.5	Pushed
83.0		Black compact with coarse sand, fine sand, silt and clay. May be stratified with firm and soft bands. No water loss or gain. Soil becoming softer with depth		Vane	Test	30.0		Pushed
				Vane	Test	35.0		Pushed
			17	CO	3	36.6	1.5	Pushed
				Vane	Test	40.0		Pushed
				Vane	Test	45.0		Pushed
			18	CO	3	46.5	1.5	Pushed
113.0	Rock			Vane	Test	50.0		Pushed
				Vane	Test	50.0		Pushed

SAMPLING METHOD
*A – SPLIT TUBE
B – THIN WALL TUBE
C – PISTON SAMPLER
D – CORE BARREL
E – AUGER
F – WASH

SHIPPING CONTAINER
N – INSERT
O – TUBE
P – WATER CONTENT TIN
Q – GLASS JAR
R – CLOTH BAG
S – PLIOFILM BAG
Z – DISCARDED

INSPECTOR: G. Wilson

LOGGED BY: G. Wilson

APPROVED:

DATE:

PRELIMINARY

H. G. A[illegible]S & COMPANY LIMITED — CONSU[illegible]NG ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 3-B

SITE: Tilbury Creek, Ontario — SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	TYPE	SIZE	DEPTH	REC'D	PENETRATION
					Inches	Feet	Feet	
	Note:	At depth of 118 feet, struck very hard formation which was either bedrock or boulder lying on bedrock	19	CO	3	56.5	1.5	Pushed
				Vane	Test	60.0		Pushed
				Vane	Test	65.0		Pushed
		Very strong natural gas supply blew all water from the casing Pressure dropped gradually with time but further drilling was made impossible	20	CO	3	66.5	1.5	Pushed
				Vane	Test	70.0		Pushed
				Vane	Test	75.0		Pushed
		No water was lost or gained	21	BO	3	76.5	1.5	Pushed
		Hole was sealed with cement and soil		Vane	Test	83.0		Tapped but no penetration
			22	AR	2	83.0 to 84.5	0.5	92 blows 140-lb hammer falling 30"

PRELIMINARY

H. G. A[illegible]ES & COMPANY LIMITED — CONS[illegible]TING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 4-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd. — STARTED 2:00 p.m. September 30 1959; FINISHED 3:00 p.m. October 1 1959

METHOD OF DRILLING: SOIL: Wash Boring — CASING DIAM. AX; ROCK: Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 49+ 30; DEPARTURE 45.0 Feet Left; BEARING --; INITIAL DIP 90 degrees; OTHER DIPS --

ELEVATIONS: DATUM G.S.C.; DRILL PLATFORM --; GROUND SURFACE 577.4; ROCK SURFACE 464.4; BOTTOM OF HOLE 452.4; WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
		Previously drilled hole by Department of Highways - Hole No. 3						
60.0								
	Clay till	Grey, firm to soft, becoming siltier towards the bottom, contains angular pebbles						
113.0								
	Bedrock	Soft calcareous shale with limestone bands and nodules. No water gain or loss Practically no core recovery						
125.0								

SAMPLING METHOD
* A - SPLIT TUBE
B - THIN WALL TUBE
C - PISTON SAMPLER
D - CORE BARREL
E - AUGER
F - WASH

SHIPPING CONTAINER
N - INSERT
O - TUBE
P - WATER CONTENT TIN
Q - GLASS JAR
R - CLOTH BAG
S - PHOTOFILM BAG
Z - DISCARDED

INSPECTOR: C. Wilson

LOGGED BY: C. Wilson

APPROVED:

DATE:

PRELIMINARY

H. G. A[illegible]ES & COMPANY LIMITED — CON[illegible]TING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario

JOB No. 83 7

PROJECT: Hwy 401 and Tilbury Creek Crossing

HOLE No. 5-B

SITE: Tilbury Creek, Ontario

SHEET No. 1 OF 1.

CONTRACTOR: F.E. Johnston Drilling Co. Ltd.

STARTED 3:00 pm. September 29 1959
FINISHED 1[illegible]:30 pm. September 30 1959

METHOD OF DRILLING: SOIL Wash Boring
ROCK —

CASING DIAM. AX
CORE DIAM. —

LOCATION: ~~LATITUDE~~ CH 47+ 42
DEPARTURE 42.0 Feet Left
BEARING —
INITIAL DIP 90 degrees
OTHER DIPS —

ELEVATIONS: DATUM C.S.C.
DRILL PLATFORM —
GROUND SURFACE 576.8
ROCK SURFACE 459.8
BOTTOM OF HOLE 459.8
WATER TABLE —

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO.	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
		Previously drilled hole by Department of Highways - Hole No. 1						
55.0								
	Clay till	Grey, firm to soft, contains angular pebbles						
83.0								
		Compact sand, silt and black earthy material No water loss or gain						
86.0								
	Clay	Brown, soft and plastic						
117.0								
	Rock	Struck bedrock or boulder lying on bedrock						

SAMPLING METHOD
* A — SPLIT TUBE
B — THIN WALL TUBE
C — PISTON SAMPLER
D — CORE BARREL
E — AUGER
F — WASH

SHIPPING CONTAINER
N — INSERT
O — TUBE
P — WATER CONTENT TIN
Q — GLASS JAR
R — CLOTH BAG
S — PLIOFILM BAG
Z — DISCARDED

INSPECTOR G. Wilson

LOGGED BY G. Wilson

APPROVED

DATE

PRELIMINARY

H. G. A[illegible]S & COMPANY LIMITED — CONS[illegible]ING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT: Department of Highways of Ontario — JOB No. 837

PROJECT: Hwy 401 and Tilbury Creek Crossing — HOLE No. 6-B

SITE: Tilbury Creek, Ontario — SHEET No. 1 OF 1

CONTRACTOR: F.E. Johnston Drilling Co. Ltd — STARTED 2:00 p.M. September 26 19 59 — FINISHED 5:30 p.M. September 28 19 59

METHOD OF DRILLING: SOIL Wash Boring — CASING DIAM. AX

ROCK Diamond Drill — CORE DIAM. AXT

LOCATION: ~~LATITUDE~~ CH 46 + 82
DEPARTURE 40.0 Feet Right
BEARING --
INITIAL DIP 90 degrees
OTHER DIPS --

ELEVATIONS:
DATUM G.S.C.
DRILL PLATFORM --
GROUND SURFACE 576.9
ROCK SURFACE 457.9
BOTTOM OF HOLE 446.9
WATER TABLE --

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE NO	TYPE *	SIZE	DEPTH	RET'D	PENETRATION TEST
0								
		Previously drilled hole by Department of Highways - Hole No. 2						
55.0								
	Clay till	Grey, contains angular pebbles						
84.0								
		Fine silty sand and clay No water loss or gain No gas struck						
119.0								
	Bedrock	6-inch gneiss boulder lying on top of soft calcareous shale with fossils and interbedded with thin hard shales and limestone lenticles						
130.0								

SAMPLING METHOD
*A – SPLIT TUBE
B – THIN WALL TUBE
C – PISTON SAMPLER
D – CORE BARREL
E – AUGER
F – WASH

SHIPPING CONTAINER
N – INSERT
O – TUBE
P – WATER CONTENT TIN
Q – GLASS JAR
R – CLOTH BAG
S – PLIOFILM BAG
Z – DISCARDED

INSPECTOR G. Wilson

LOGGED BY G. Wilson

APPROVED

DATE

PRELIMINARY

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS
CANADA

IN YOUR REPLY REFER TO
FILE 837

Files

January 29, 1960

Department of Highways of Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. L.G. Soderman

Gentlemen: Department of Highways of Ontario
Tilbury Creek Bridge - Highway
No. 401 - District No. 1 - Wp-160-58
Settlement Studies

We have now completed our study of the settlement of the proposed Tilbury Creek bridge, and our report on this work is in a state of final preparation. In order, however, that the results of this work may be immediately available to you, we have summarized our general conclusions in the following letter. As you have indicated previously to us, you will want to discuss these results in detail at a meeting to be arranged in the near future when you have had time to peruse our conclusions. We will, of course, be happy to do this at your convenience.

We have computed the settlements, both elastic and consolidation, for the earth-fill embankment and the proposed abutments of the Tilbury Creek bridge in accordance with the general arrangement which you provided. The analysis is based upon the soil profile which we confirmed in the drilling operations at the site last September, and the soil properties which were determined in subsequent laboratory tests, and the results of the analysis, are shown on the attached sketch No. SK-837-LS-1. The movements indicated on this sketch are for the embankment settlements alone and for the combined embankment plus abutment settlements. They include both the elastic and consolidation components of settlement and are the total movements which can be expected when consolidation is 100 per cent complete. These settlements have been calculated on the assumption that both the abutment and the embankment loads are applied instantaneously and

4

simultaneously and their estimation has been further simplified by the fact that the general arrangement is essentially symmetrical, with the exception of the effect of the angle of skew between the river and the highway.

You will note that the settlements vary at different locations in the fill and for different points on the abutment footings. However, because of symmetry, diagonally opposite footings do have similar settlement patterns. This means that the settlements of the two abutments supporting the same bridge deck are different although the settlements of the two bridges, if one is turned end for end, are identical. Because of the location of the abutment footings relative to the embankment load, different settlements will occur at the two ends of the footing, and additionally, the settlements at the toe of the footing will differ from those at the heel of the footing. These settlements at the four corners of an individual footing are not coplanar, and, therefore, a small amount of torsion in the footing is indicated, but, as the analysis assumes complete flexibility of the applied loads, we believe that these torsional movements will be virtually removed by the rigidity of the abutments and will not in fact, occur. As the two abutments supporting the same bridge deck will act independently of each other, and as the settlements of these two abutments differ, it can be expected that separate torsional movements will be introduced into the bridge deck.

In our consideration of this problem, we have assumed that the deck will be of a simply supported reinforced concrete girder and slab construction, and although the torsional movements may be reasonably small, some cracking might be expected. While we realize that any such torsional effects, and particularly those in a skewed bridge, are of concern to bridge designers, we do not believe that we are in a position at this time to comment on the structural significance of the magnitude of strains indicated by this analysis. We feel that any decision of this nature should be left to the structural designers of the bridge, with whom we will, of course, be happy to discuss the problem. We do believe, however, that there are several considerations of a general nature which have some bearing on the consideration of this problem, and we are setting them out below.

The work which we have done is in accord with the most recent methods of settlement analyses, and an attempt has been made to be as rational as possible in our assumptions and methods. While we have, therefore, no reason to question the magnitudes of the estimated

movements, we do feel intuitively that they are somewhat greater than those which will be experienced. But, in the absence of anything more concrete than this suspicion, we believe that we must accept them at face value for further considerations.

On the attached sketch a graph has been included to show the rate at which the settlement can be expected to occur, and this indicates that approximately 15 to 25 per cent of the total settlement will be elastic strain and will occur immediately upon the imposition of the load. The consolidation settlement will occur slowly thereafter, and 45 per cent of the total settlement will be complete in approximately 10 years, and 80 per cent in approximately 50 years. Theoretically, the final settlements will only be reached in an infinite length of time. This time rate of settlement was computed on the basis of double drainage occurring at the top and bottom of a 100-foot thick layer of clay, which condition is typical for conditions at any location on the embankment. If anything, this rate of settlement is too slow, and it can be expected that consolidation will occur more quickly than is indicated here. More accurate estimates of this time rate of settlement are extremely difficult as they depend so much upon the drainage conditions which are sometimes difficult or impossible to know exactly. However, in view of the apparently slow rate of settlement, it is probable that the final computed total settlements and differential settlements will not be realized within the economic life of the structure. All other things being equal, the maximum differential settlements to which the structure may be subjected will be less than those indicated by this analysis. Moreover, if the settlements do take place at a fairly slow rate, it is probable that some stress relief may be realized by the structure through creep effects in the concrete.

The magnitude of the differential settlements to which this structure may be subjected will depend to a considerable degree on the sequence and timing of construction of the various parts of the structures. That is, the construction of the embankment at an appreciable time interval ahead of the bridge abutments. and the construction of the bridge deck as a last operation, could result in a considerable decrease in the differential settlements that the deck will have to accommodate.

The possibility should not be overlooked of designing the bridge seats and bearings so that horizontal movements in all directions due to the tilting of the

abutments can be accommodated and so that differential vertical settlements can be eliminated periodically by shimming. The latter is a particularly attractive method of accommodating small amounts of settlement which occur at a reasonably slow rate. There also exists the possibility of distributing the abutment load to reduce the differential settlement but as most of this settlement results from the embankment load, the beneficial effect of this approach is not very great.

The foregoing comments have, of course, been based on the use of spread footings beneath the abutments. We realize, however, that it has been decided, at least tentatively, to support these abutments on piles bearing directly on the underlying rock at an approximate depth of 100 feet. The use of piles would, we believe, introduce two problems which merit consideration. In using piles it would be assumed that the settlements of the abutments would be negligible as compared to those of the adjoining embankment and, consequently, comparatively abrupt changes in elevation may be created at the ends of the bridge deck. Additionally, the large settlements of the underlying soil which would result almost entirely from the relatively large embankment load would probably impose added load of considerable magnitude upon the bearing piles through negative skin friction effects. Since the piles would be bearing on a shale rock of somewhat dubious strength, we believe that the possibility of settlement of the piles cannot betotally ignored. This latter effect may, however, have a comparatively small influence on the bridge performance.

We hope that the foregoing will provide you with a reasonably good summary of the conclusions of our settlement study, and we shall look forward to discussing them in much greater detail with you in the near future. Our final report on this part of the Tilbury Creek study should be completed within the next week or two and will be forwarded to you at that time.

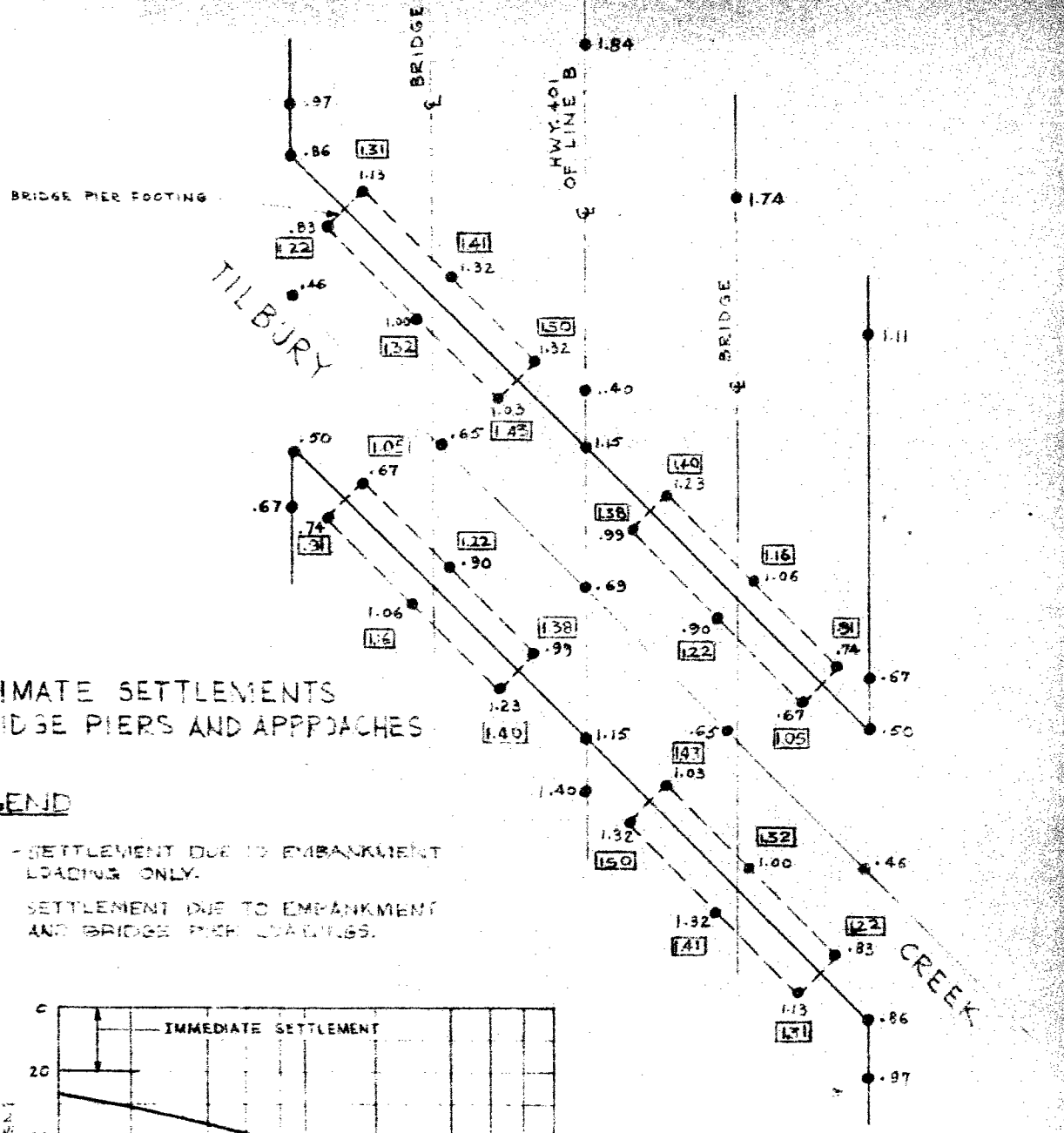
Yours very truly,

H.G. ACRES & COMPANY LIMITED

D. H. MacDonald.

D.H. MacDonald
Geotechnical Engineer
Technical Division

DHM:bcb

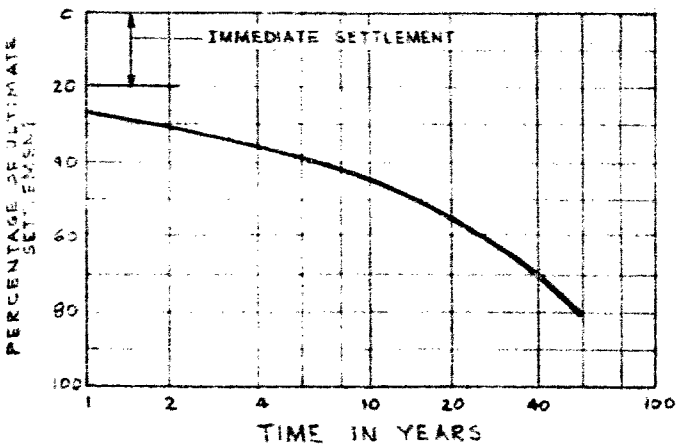


ULTIMATE SETTLEMENTS OF BRIDGE PIERS AND APPROACHES

LEGEND

1.06 - SETTLEMENT DUE TO EMBANKMENT LOADING ONLY.

[1.21] SETTLEMENT DUE TO EMBANKMENT AND BRIDGE PIER LOADINGS.



NOTE:

SETTLEMENTS ARE MEASURED IN FEET.

AVERAGE TIME RATE OF SETTLEMENT

DEPARTMENT OF HIGHWAYS OF ONTARIO	DATE: JAN. 26, 1960		H. G. ACRES & CO. LTD. CONSULTING ENGINEERS NIAGARA FALLS, CANADA
HWY. 401 AND TILBURY CREEK CROSSING	DR. R.A.B.	APP.	
FOUNDATION SETTLEMENTS	SCALE 1" = 40'-0"	JOB No. 837	SHEET No. ——
			DWG. No. SK-837-LS-1

PRIVATE POST CARD

STAMP

H. G. ACRES & COMPANY LIMITED

1259 Dorchester Road

Niagara Falls, Canada

Mr. L.G. Soderman

RETURN TO H. G. ACRES & CO. LTD. **Nº 69932**

Date ..... 19.....

Development .....

We acknowledge receipt of .....

.....Prints of Drawing No. .....

.....Prints of Drawing No. .....

.....Prints of Drawing No. .....

.....Prints of Drawing No. .....

FIELD OFFICE COPY **Nº 69932**

Date January 29 19 60

Development 837

Enclosed herewith please find--

3 Prints of Drawing No. SK-837-LS-1

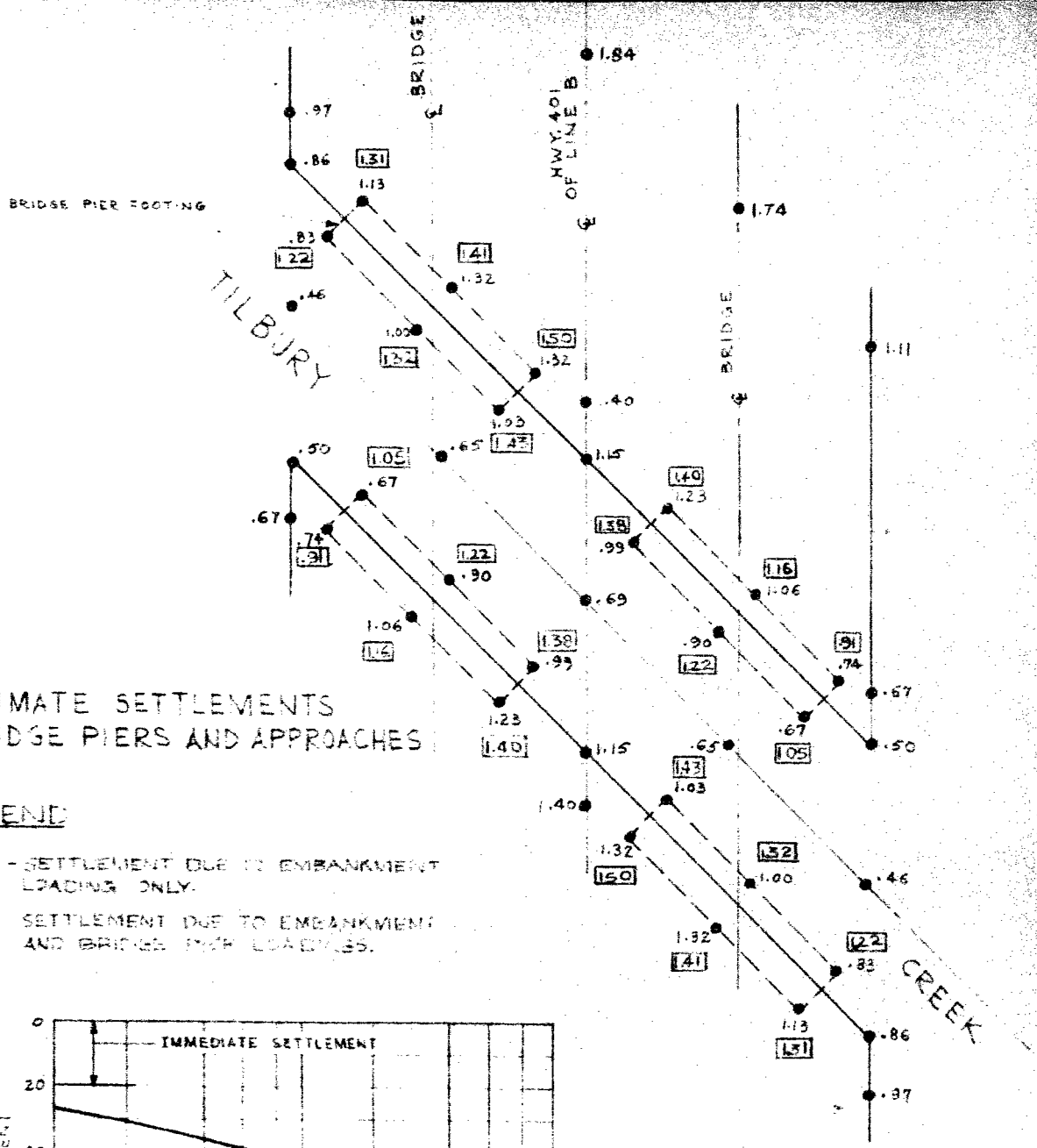
.....Prints of Drawing No. .....

.....Prints of Drawing No. .....

.....Prints of Drawing No. .....

H. G. ACRES & CO. LTD.

bcb

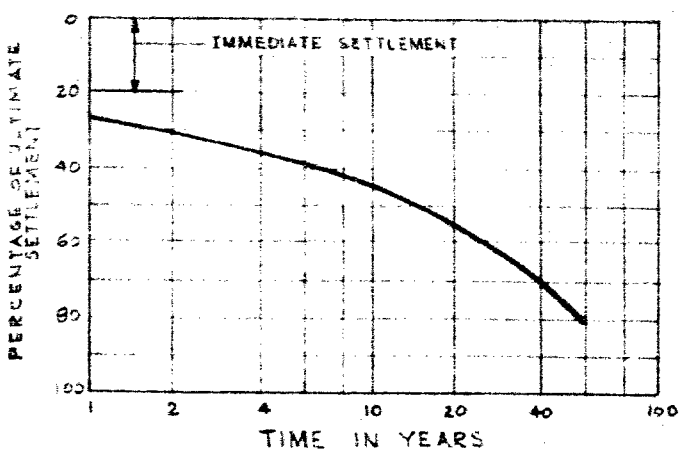


ULTIMATE SETTLEMENTS OF BRIDGE PIERS AND APPROACHES

LEGEND

1.08 - SETTLEMENT DUE TO EMBANKMENT LOADING ONLY.

[1.21] SETTLEMENT DUE TO EMBANKMENT AND BRIDGE PIER LOADINGS.



AVERAGE TIME RATE OF SETTLEMENT

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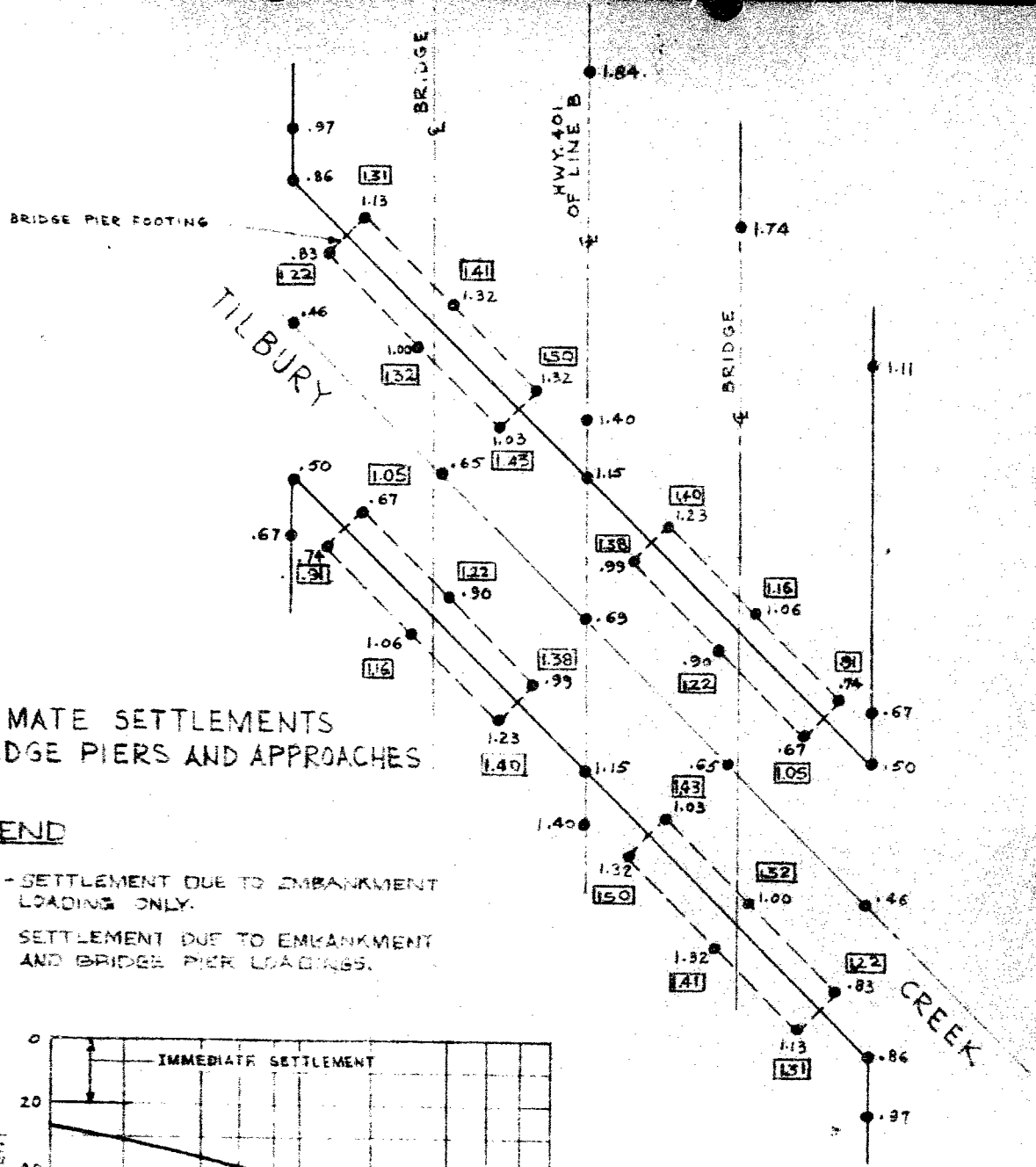
FOUNDATION SETTLEMENTS	SCALE 1" = 40'-0"	JOB No 837	SHEET No. ———
			DWG. No. SK-837-LS-1



ULTIMATE SETTLEMENTS OF BRIDGE PIERS AND APPROACHES

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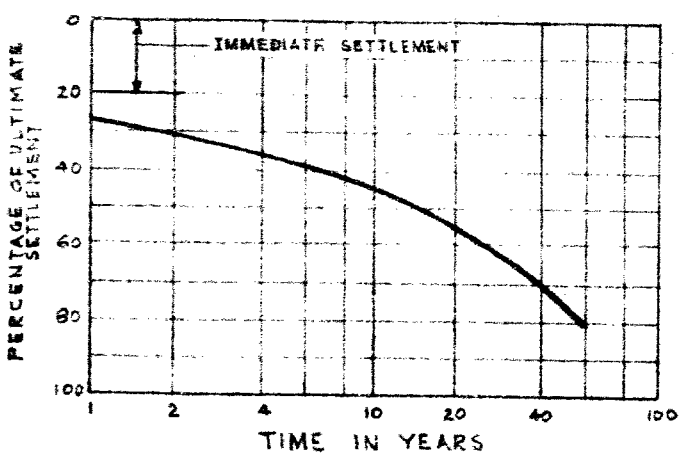

AVERAGE TIME RATE OF SETTLEMENT

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FOUNDATION SETTLEMENTS	SCALE 1" = 40'-0"	JOB No. 837	SHEET No. ———
			DWG. No. SK-837-LS-1

c.c. L.G.Soderman

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS
CANADA

IN YOUR REPLY REFER TO
FILE 837

PERSONAL AND
CONFIDENTIAL

January 29, 1960

W.P. 160-58

Department of Highways of Ontario,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. A. Rutka,
A/Materials and Research Engineer

Dear Alex:

We were very glad to receive your letter of January 25. and to see that the situation concerning our invoice for job No. 837 is considerably clarified, and that we both agree that the actual costs are appreciably better than those originally anticipated. We are also glad to learn that our revised billing will be satisfactory to the Department and a new invoice conforming with our suggested changes has already been sent to you.

We appreciate the specific comments which you have made, and as we now understand each others views on these points which have arisen, we believe that a great deal has been accomplished towards making future work more efficient and more economical. We also appreciate your interest in having our firm competitive with other consulting organizations, and we want to assure you that we will do everything we can to maintain a good position in this regard. In particular, we are happy to know that we both share the intention to define the scope of our work more carefully on future jobs.

With best wishes,

Yours sincerely,
H.G. ACRES & COMPANY LIMITED

D.F. MacDonald
Geotechnical Engineer
Technical Division

Dm:bcb

Department of Highways of Ontario
Mr. L. G. Soderman

FIELD OFFICE COPY

Nº 71536

Date March 22 1960

Development 837

Enclosed herewith please find—

3 Prints of Drawing No. Sketch No. SK-837-LS-1

Prints of Drawing No.

Prints of Drawing No.

Prints of Drawing No.

H. G. ACRES & CO. LTD.

H. G. ACRES & COMPANY LIMITED
CONSULTING ENGINEERS
NIAGARA FALLS
CANADA

IN YOUR REPLY REFER TO
FILE 837

March 22, 1960

Materials and Research Section,
Department of Highways of Ontario,
Downsview, Ontario

Attention: Mr. L.G. Soderman
Principal Soils and Foundation
Engineer

Gentlemen: Department of Highways of Ontario
Tilbury Creek Bridge
Highway 401, District No. 1
W.P. 160-58
Settlement Studies

We are enclosing with this letter three extra copies of sketch No. SK-837-LS-1. This drawing originally accompanied our letter of January 29, 1960, dealing with the results of our settlement studies of the proposed Tilbury Creek Bridge. Extra copies were requested by Mr. L.G. Soderman on March 18, 1960.

Yours very truly,

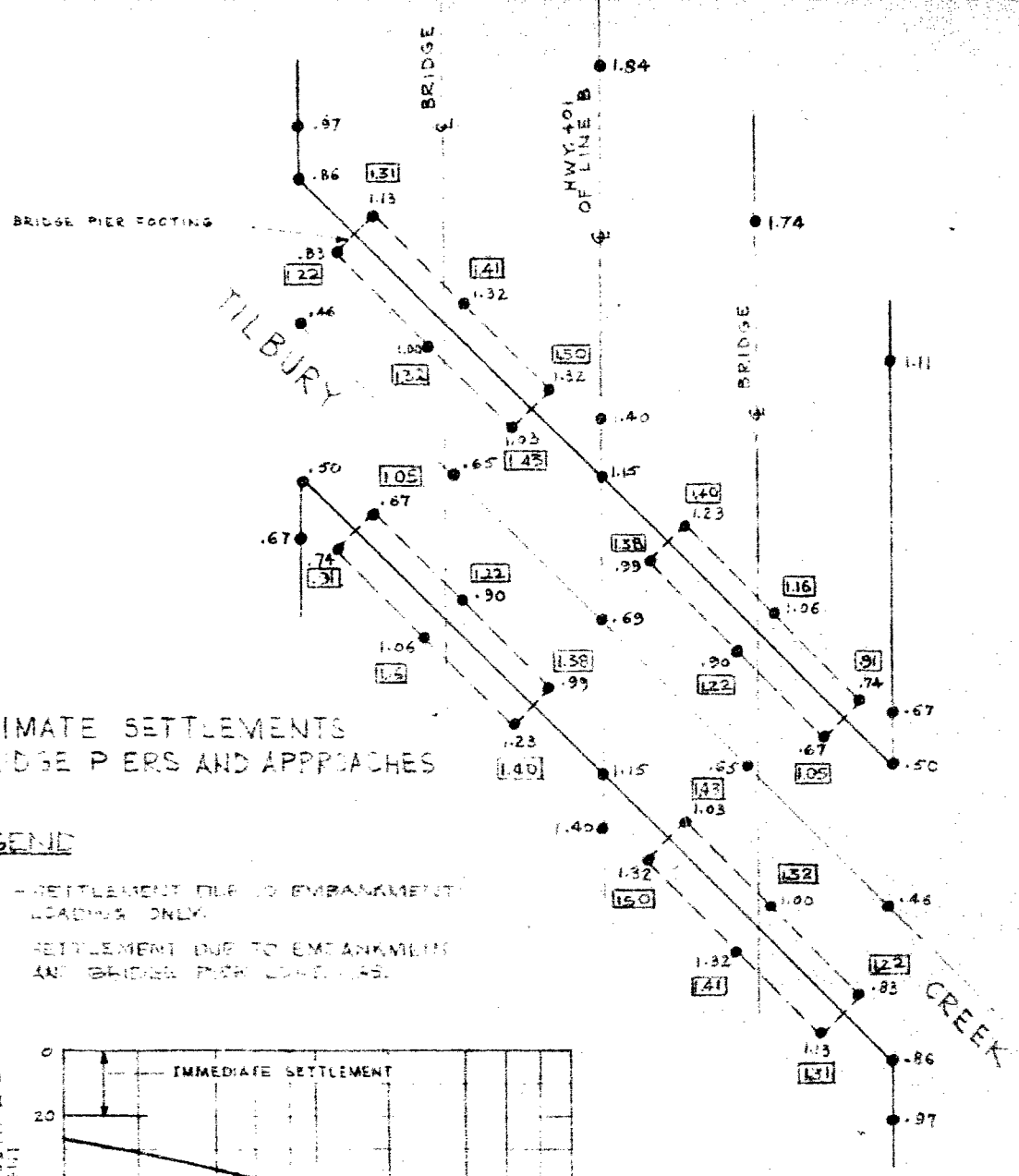
H.G. ACRES & COMPANY LIMITED

D.H. MacDonald

D.H. MacDonald
Geotechnical Engineer

TCK:kc
Encl.

RECEIVED
MAR 23 1960
DEPT. OF HIGHWAYS
POSTAL
ROOM

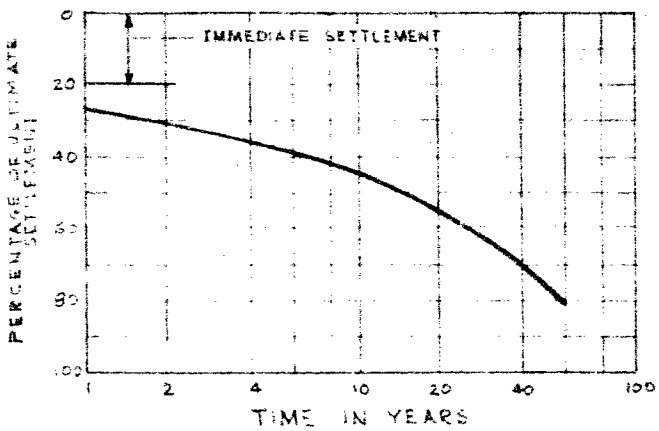


ULTIMATE SETTLEMENTS
OF BRIDGE PIERS AND APPROACHES

LEGEND

1.08 – SETTLEMENT DUE TO EMBANKMENT LOADING ONLY.

[1.31] SETTLEMENT DUE TO EMBANKMENTS AND BRIDGE PIER LOADINGS.



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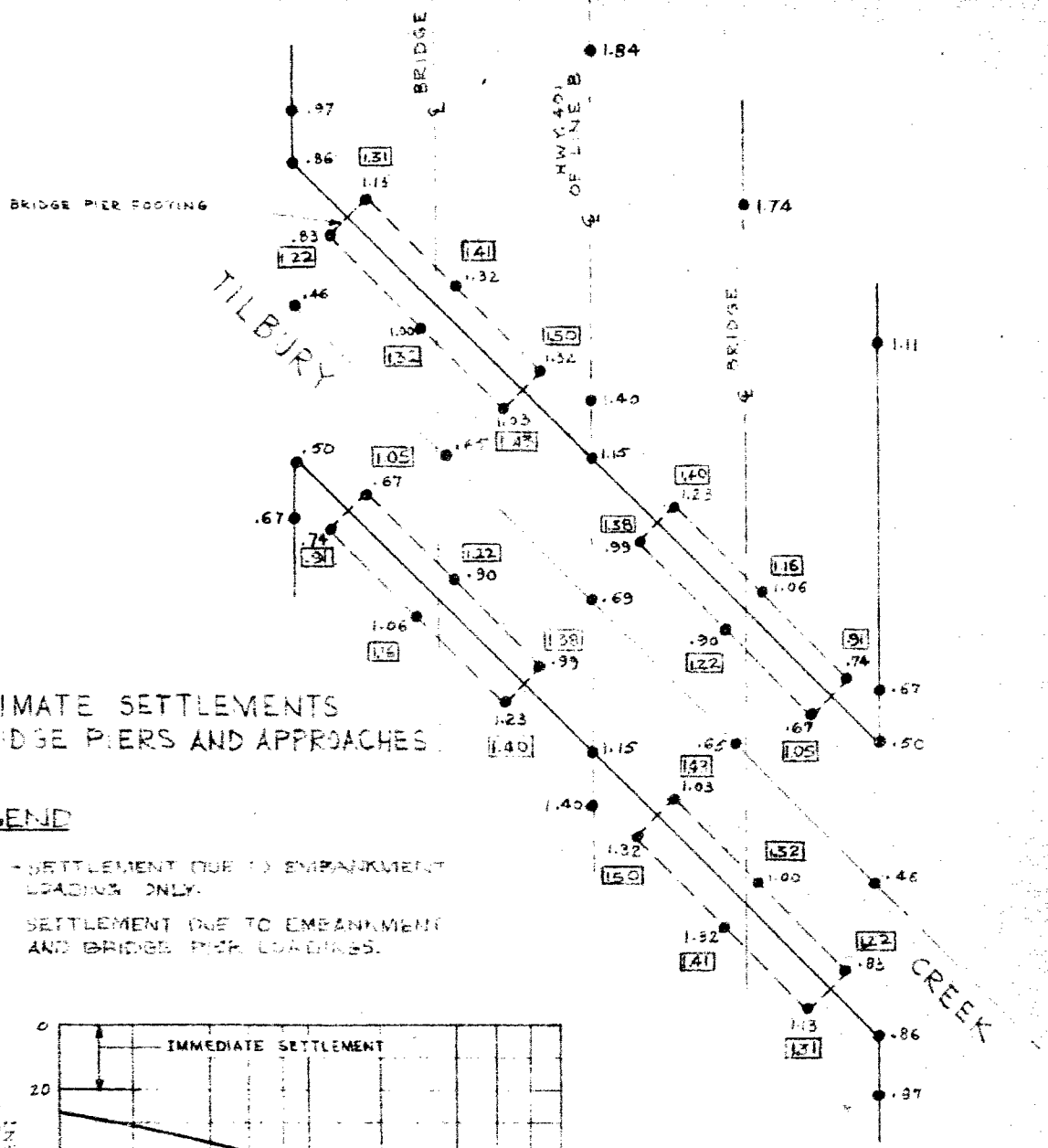
DEPARTMENT OF HIGHWAYS OF ONTARIO	DATE JAN. 26, 1960		H. G. ACRES & CO. LTD. CONSULTING ENGINEERS NIAGARA FALLS, CANADA
HWY. 401 AND TILBURY CREEK CROSSING	DR. R.A.B.	APP.	
FOUNDATION SETTLEMENTS	SCALE 1" = 40'-0"	JOB No. 837	SHEET No. —— DWG. No. SK-837-LS-1



ULTIMATE SETTLEMENTS OF BRIDGE PIERS AND APPROACHES

LEGEND

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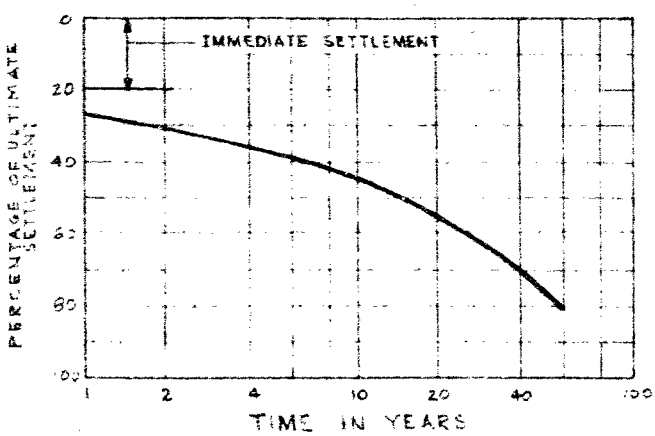

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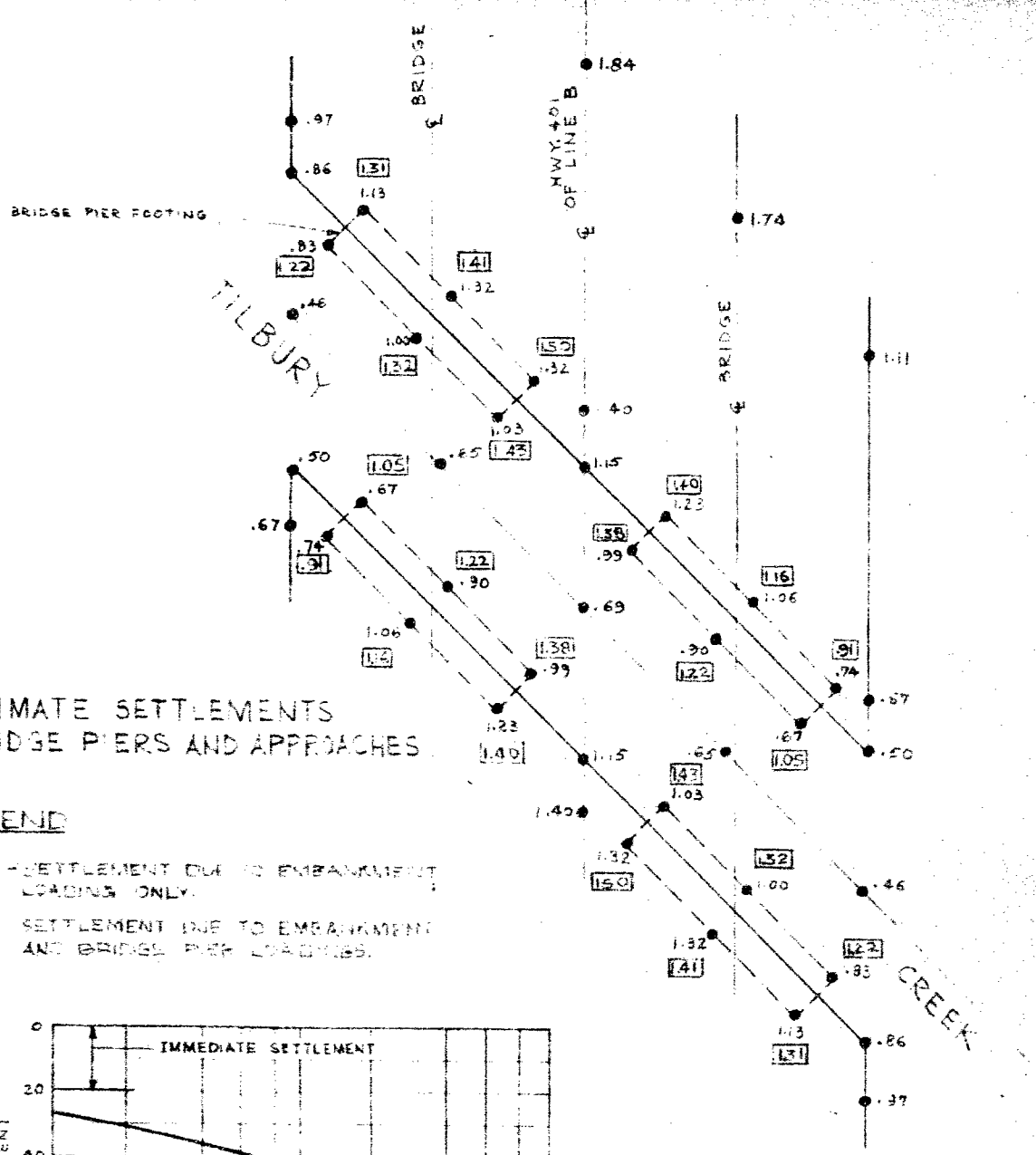
FOUNDATION SETTLEMENTS	SCALE 1" = 40'-0"	JOB No. 837	SHEET No. ——
			DWG. No. SK-837-LS-1



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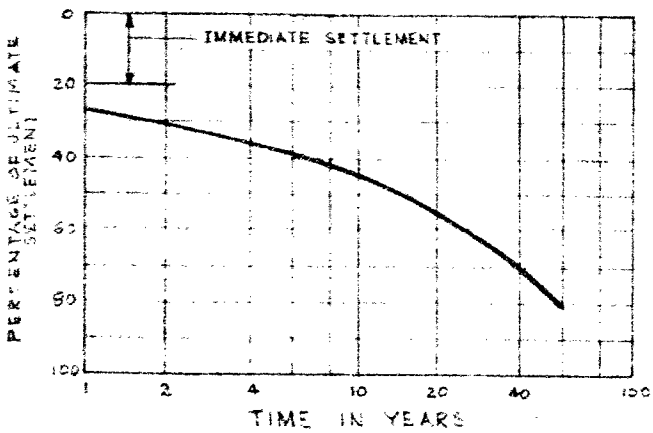

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FOUNDATION SETTLEMENTS	SCALE 1" = 40'-0"	JOB No. 837	SHEET No. ———
			DWG. No. SK-837-LS-1

April 22, 1960.

Mr. Leon J. Marshall,
Chief Bridge Engineer,
De Leuw, Cather & Co. of Canada, Ltd.,
Consulting Engineers,
226 Sparks Street,
Ottawa 4, Ontario.

Dear Leon:-

Re: Meeting on Tilbury Soil Investigation
-- Your Letter - April 20, 1960 --
Ref. No. 348-Q-3b.

Your letter received -- expected a cigar! I will be contacting Dr. MacDonald of H.G. Acres, Ltd., and arranging a meeting for one day next week. I will advise you by 'phone, two days in advance of the meeting date.

Respectfully submitted -

Larry

to one who has 4 sons
by one who has 3 sons.

LGS/MdeF

Meeting Thursday April 28 1:30 pm

DE LEUW, CATHER & COMPANY
OF CANADA LIMITED
CONSULTING ENGINEERS
TORONTO OTTAWA

226 SPARKS STREET
OTTAWA 4, ONTARIO
CEntral 3-9663

Our Ref. 348-Q-3b

April 20th, 1960

Mr. Larry Soderman,
Soils Engineer,
Department of Highways,
Parliament Buildings,
Toronto, Ontario.

Dear Larry,

Re: Meeting on Tilbury Soil Investigation

As soon as my fourth son was born at the beginning of the month, I telephoned you about our postponed meeting and learned you were away ill. As my wife was in hospital, I made the trip anyway as I had to see Art Toye, Ted Hewson and Dick Hobson on other matters. When I was down there it was confirmed that you were still on sick leave and I asked to be informed when it would be convenient to meet with you and Acres engineer. In case you are still waiting to hear from me will you let me know when you can arrange the proposed meeting if you still feel it is necessary.

I gather from our Toronto office that although it may not be feasible to use an alternative foundation design for the Tilbury structure due to the severe skew, you are still interested in discussing the subject for application to future structures in this area.

Yours very truly,
DE LEUW, CATHER & CO. OF CANADA LIMITED

Leon J. Marshall

Leon J. Marshall, P. Eng.,
Chief Bridge Engineer

LJM/rm

APR 21 10 25 AM 1960

FORM SB-OS-20
JUNE 1959

DEPARTMENT OF HIGHWAYS
TELETYPE MESSAGE

1960 NOV 29 AM 10 1 32

MARK WITH X

URGENT [X]
ROUTINE [ ]

FROM Mr. L. G. Soderman,
Foundations Office, Lab. Bldg., D.H.O.
Downsview, Ontario.

TO Mr. P. A. Peacock,
Construction Engineer,
Chatham, Ontario.(Dist. #1)

DATE November 29/60

TIME

SUBJECT-

RE: CONTRACT 60-110 - HWY. 401 - TILBURY CREEK
(YOUR TELETYPE DATED NOVEMBER 25/60)

ARRANGEMENTS HAVE BEEN MADE FOR OUR PROJECT ENGINEER, MR. ROLY. SALVAS, TO MAKE THIS INSTALLATION, (BENCH MARK) THE LATTER PART OF THIS WEEK. HE WILL REPORT TO YOUR OFFICE.

LGS/MdeF

for - Mr. de Leuw

L. G. Soderman,
PRINCIPAL FOUNDATIONS ENGR.

SIGNATURE

NOTE- MESSAGES OUTGOING AND MARKED URGENT ARE HANDLED AS SOON AS POSSIBLE. ROUTINE MESSAGES HANDLED WHEN CIRCUITS PERMIT. MESSAGES INCOMING TO TORONTO H.O. ARE MAILED TO ADDRESSEE. IF MARKED URGENT MESSAGES ARE TELEPHONED

CH CHATHAM NOV 25/60 4:31 PM 125[illegible]

MR L SODERMAN
SOILS

RE CONTRACT 60-110 HWY 401 TILBURY CREEK

RE BENCH MARK. WHEN WILL YOU BE INSTALLING THIS?
WORK WILL SHORTLY COMMENCE. WE ARE TYING BACK TO THE BRIDGE ON NO.2
HWY FOR OUR CONTROL

~~P A PEACOCK~~
~~CONST ENGR~~
~~DDL CLR~~
~~AT~~

Replied telephone Nov 29

TELEPHONED
TIME — NOV 28 AM 9:1[illegible]
BY
TO A.P.

TELETYPE

80
CHATHAM SEPT 27/60 5;11:05 PM 1960 SEP 27 PM 2:47

T. Sterman, Foundations
~~B DAVIS~~
BRIDGE DESIGN ENG
BRIDGE OFF
TORONTO

RE CONTRACT 60-110 HWY 401

TILBURY CREEK STRUCTURE

IT IS OUR UNDERSTANDING THAT YOUR OFFICE WISHES TO ESTBLISH A PERMANENT BENCH MARK NEAR THE STRUCTURE BEFORE THE CONTRACTOR STARTS WORK. THE CONTRACT HAS BEEN ADVERTISED AND WILL BE AWARDED ON OCTOBER 26/60.

J H BLEVINS
ACT CONSTR ENG
MJ CLR

BM

Re: As above and this teletype

Foundations Section will install bench mark. For emerging your office will be contacted. Thank you for information.

L.B.C

EARTH BORROW TEST DATA

PIT NAME	LOCATION	SAMPLED BY	LAB. NO.	FIELD NO.	DECISION	REMARKS
ROAD SAMPLE	TILBURY CK. TILBURY E. TWP.	J.H. HENDERSON	61-0-2238 61-S-85739	61-HP-29	A	J. ROY. per. D. SUZUKI. 10-7-61
HARRY SMITH.	CONC. 3. LOT 21.	"	61-0-2240 61-S-85741	61-HP-31	A	" " " "
" "	" "	"	61-0-2239 61-S-85740	61-HP-30	A	" " " "

FORM OB-ML-140
FEB. 58

DIST 1. CHATHAM.

AGGREGATE TEST DATA SUMMARY

OVER

CONTRACT NO. 60-110 HWY. NO. 401

WORK AND LOCATION STRUCTURE & APPROACHES HWY #01 TILBURY NORTH TWP BRIDGE #2 (TILBURY CREEK)

PIT NAME	PIT LOCATION	FIELD NO.	LAB NO.	DATE SAMPLED	COARSE AGGREGATE %	LA	D	S	ABS	C	PN	FINE AGGREGATE R8	R14	R28	R48	R100	R200	S	PN	ABR	O	T
CHATHAM #4		MX 2028, 3033	60-S-70451	9/11/60	0.5							8.1	31.3	59.5	85.7	96.8	98.9					
ADAMS SA & GRAVEL	(STOCKPILE)	EX STRUCT CONC SAND																				
DUNNVILLE #20		MISC 2031	60-S-12519	9/11/60	97.2	23.5		BSG 11.6	0.23	0.7	107.9											
CAYUGA QUARRIES.	CAYUGA	DN STRUCT CONC STONE DUE TO COARSENESS OF GRADING											BSG. 2.67 ASG. 2.78									
"	"	MISC 2034-35	60-S-70452	9/11/60	98.0																	
		DN STRUCT CONC STONE DUE TO COARSENESS OF GRADING																				
ROMNEY #2.		MD-6048	MD-6048 61-S-89286	6-7-61	10.9							10.4	25.6	39.4	65.0	96.0	98.8					
D.A. SHANKS.	—	MX																				
" #15		MD-1043	" 89419	20/7/61	23.3							18.8	42.0	66.3	91.4	96.6	97.6					
D ELDERBECK	S. #3 HWY LAKESHORE	ASC								DIST & DN												