VANCOUVER SOILS GROUP

PEAT SYMPOSIUM

27th February 1967

Chairman: J.W. Gadsby — Moderator: C.O. Brawner

Session I — 4:15 p.m.

Dr. C.A. Rowles & Dr. L.E. Lowe	Occurrence and Nature of Organic Deposits in B.C.
R.G. Doyle	Laboratory Testing
G. MacLeod	Laboratory Testing
Dr. K.V. Helenelund	Field Sampling
Questions and Discussion	
C.O. Brawner	Summary

Supper Break 6:15 to 7:45 p.m.

Session II — 7:45 p.m.

P. Cook H. Nasmith E.E. Readshaw A.G. Reid K.I. Morrison Dr. K.V. Helenelund	Field Problems
Questions and Discussion	
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Chairman:
J.W. Gadsby

S.F. Hillis

Secretary:
Dr. W.D. Liam Finn

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OCCURRENCE OF ORGANIC SOILS IN BRITISH COLUMBIA

by

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In many respects this report on the organic soils of British Columbia will be incomplete and tentative. This is because of the very extensive nature of the province and the large areas that have not yet been soil surveyed. Also, Canadian soil scientists have not studied organic soils to the same extent that they have the mineral soils, and understanding of them is far from complete.

Confusion often arises because of differences in definition and interpretation of terms, and therefore, at the outset, consideration will be given to the term "organic soils" as used in this paper. For several years the National Soil Survey Committee of Canada has been studying organic soils and their classification, and although the work is not complete, a number of tentative suggestions for trial under Canadian conditions have been made (Ehrlich 1965). Some of these suggestions will be presented here.

Tentatively, the National Soil Survey Committee have suggested that at the highest level of classification, i.e., the Order, organic soils be defined as soils with 30 percent or more organic matter and with more than 12 inches of drained, cultivated or pastured, or 18 inches of natural or undisturbed material.

Following this concept, organic soils are natural bodies occurring in bogs as a result of the accumulation of plant remains in a water-saturated environment.

They vary in morphological features, primarily because of varying degrees of alterations on different kinds of plants by microbial activities as affected by certain local geological, chemical, topographical and microclimatological influences (Farnham and Finney, 1965).

EXTENT AND DISTRIBUTION OF ORGANIC SOILS IN B.C.

Organic soils occur throughout the world but are most extensive in the cool and moist portions of the northern hemisphere, particularly in the glaciated areas. However, for the formation of organic soils an environment is required that is water-saturated for extended periods of time, and that contains low quantities of oxygen or is anaerobic (Farnham and Finney, 1965).

It has been estimated that 60 percent of the world's organic soils occur in Russia, and that Finland and Canada rank next. The Atlas of Canada (1957) indicates that there are about 125,000 sq. miles, or 3 % of the area of Canada, that is mainly peat, and that the largest areas occur in Ontario, Quebec and Manitoba.

Rowles, Farstad and Laird (1957), in writing on the soil resources of British Columbia, commented as follows - "Organic soils are found in areas where organic matter forms more rapidly than it decomposes. This is most frequently the case in very moist, cool areas where drainage is restricted. Since these conditions are found in many parts of the province, organic soils are very widespread. Large areas of organic soils are found in the alpine regions and in the flat-lying, incompletely drained northeast portions of the province. An estimated 4.6 million acres are believed to occur in the province."

Recently, a small-scale map of the province has been in preparation through the co-operation of the Provincial, Federal and University pedologists, and this indicates somewhat more precisely where the main areas of organic soils occur. The largest areas are shown on this map to occur in the Lower Fraser Valley and valleys of the coastal mountains, on Vancouver Island, Queen Charlotte Islands, and in the northeast corner of the province, in the Great Plains region.

Sprout (1967) in commenting on the organic soils of British Columbia notes that - "looking at the province as a whole, organic soils occupy only a small portion of the landscape. A notable exception is the northeastern part of the province comprised of the Fort Nelson lowlands and surrounding area. Here organic deposits are the dominant feature of the terrain and consist mainly of accumulations of sphagnum moss. Permafrost is common.

In the plateau and laking basin country of the central interior, organic deposits occupy generally less than 10 percent of the area. Both sphagnum and sedge deposits occur. These acidic deposits occupy topographic lows (e.g., between drumlins) and vary in depth from about a foot to twenty feet or more. Some of the bogs are utilized for native pasture and hay.

In the southern interior, organic deposits are very limited, generally occurring in areas of seepage or along margins of streams. Reaction of these deposits varies depending on the type of bedrock area.

Substantial organic deposits occur on the northern extremity of Vancouver Island and along the east coast. Mixtures of sedge and sphagnum types again are dominant although Sedimentary peats also occur.

With regard to the Lower Fraser Valley, organic deposits occupy approximately 35,000 acres. About 23,000 acres are classed as deep and shallow muck. The remainder consists of essentially raw peat mainly composed of sphagnum in the upper part at least."

In summary, it has been estimated that there are over four million acres of organic soils in British Columbia, and that they occur principally in the Lower Fraser Valley and other valleys of the Coast Range, on Vancouver Island and the Queen Charlotte Islands, in the central interior and in the Great Plains region in the northeast corner of the province.

NATURE AND CLASSIFICATION OF ORGANIC SOILS IN BRITISH COLUMBIA

Although the organic soils of British Columbia have not been completely studied, it is known that many different kinds occur. It is important, therefore, that they be characterized and classified appropriately. Unfortunately, none of the approaches tried, utilizing such factors as botanical origin, degree of decomposition, cation exchange capacity, C-N ratio, ash content, etc., have proved entirely satisfactory.

The problem of characterising and classifying organic soils was reviewed recently by Farnham and Finney (1965) and after considering the available systems classed under the headings:-

(a) Topographical
(b) Surface vegetation
(c) Chemical properties
(d) Botanical origin
(e) Morphology and
(f) Genetic processes

they concluded that all systems have their shortcomings. They also concluded that the criteria which presently appear to be most useful for organic soil classification are:-

Fibre characteristics (size and count)
Bulk density
Water-holding capacity
pH
Ash content
Degree of decomposition

Based upon these criteria they proposed a system of classification which, in modified form, is being developed and tested in the United States and by the National Soil Survey Committee in Canada. The tentative classification is based essentially on the type of vegetation and its degree of decomposition. Since the organic material in the bog often varies with depth, the classification is based on a control section which includes surface, sub-surface and bottom tiers, totalling 40 inches in drained or cultivated bogs and 60 inches in natural or undisturbed bogs. The emphasis is placed on makeup of sub-surface material. An outline of the classification being tested by the National Soil Survey Committee is given below:-

ORDER	GREAT GROUP	SUBGROUP
7.0 Organic	7.1 Fibrisol	7.11 Fennic Fibrisol
		7.12 Mucinic Fibrisol
		7.1/3 Stratic
		7.1/4 Clasto-
		7.1/5 Limno-
		7.1/6 Cumulo-
		7.1/7 Cryo-
		7.1/8 Terric
		7.1/9 Lithic
	7.2 Mesisol	7.21 Unic Mesisol
		7.22 Luvic Mesisol
		7.2/3 Stratic
		7.2/4 Clastro
		7.2/5 Limno-
		7.2/6 Cumulo-
		7.2/7 Cryo-
		7.2/8 Terric
		7.2/9 Lithic
	7.3 Humisol	7.31 Unic Humisol
		7.32 Luvic Humisol
		7.3/3 Stratic
		7.3/4 Clasto-
		7.3/5 Limno-
		7.3/6 Cumulo-
		7.3/7 Cryo-
		7.3/8 Terric
		7.3/9 Lithic

Diagnostic Layers

Fibric - least decomposed stage
It must have more than 2/3 fibres (140 mesh > .1 mm) in the total mass.

Mesic - Intermediately decomposed stage
It has a fibre content between 1/3 and 2/3 in the total mass.

Humic - Most decomposed stage.
It must have less than 1/3 fibre in the total mass.

Farnham and Finney (1965) summarized data on the physical and chemical characteristics typical of three diagnostic layers and some of these are given below:-

	Range in fibre content (> 0.1 mm dry wt.basis %)	Range in ash content (% dry wt. basis)	Range in water-holding capacity (% dry wt. basis)	Range in bulk density g/cc	Range in pH (H_2O)	Range in nitrogen content (% dry wt. basis)
Fibric	76-91	2.5- 9.2	850-1920	0.06-0.17	2.9-7.5	0.4-3.0
Mesic	48-51	11.4-21.2	625-660	0.21-0.23	4.5-7.5	1.5-3.0
Humic	12-20	26.2-59.4	187-418	0.31-0.45	4.5-8.0	1.0-3.0

Other Layers

Unic - consisting of one diagnostic mesic or humic layer throughout the organic section in the subsurface and bottom tiers or in shallow peats throughout the organic section of the surface and subsurface tiers.

Fennic - dominantly fibric fen peat in the subsurface tier or dominantly fibric fen peat in the surface and subsurface tiers if a mineral substratum occurs in the subsurface tier.

Mucinic - dominantly fibric moss peat in the subsurface tier or dominantly fibric moss peat in the surface and subsurface tiers if a mineral substratum occurs in the subsurface tier.

Stratic - two or more kinds of diagnostic peat layers in significant proportions in the subsurface and bottom tiers or in the surface and subsurface tiers if the mineral substratum occurs in the subsurface tier.

<u>Luvic</u> - an illuvial layer in the subsurface tier or in the upper bottom tier with more colloidal material than the underlying peat. This layer has fine materials with a greasy, glossy appearance in fractions and in root channels.

<u>Clasto</u> - a layer(s) of significant proportions with 30 to 70 percent mineral material in the organic part of the control section.

<u>Limno</u> - a significant layer(s) ($>$2 inches) of marl, diatomaceous earth, sedimentary peat, bog iron, possibly others.

<u>Cumulo</u> - alternate layers of organic and mineral materials.

<u>Cryo</u> - permanently frozen within the control section.

<u>Terric</u> - unconsolidated mineral substratum with less than 30 percent organic matter occurring in the subsurface or bottom tiers.

<u>Lithic</u> - consolidated mineral substratum (bedrock) occurring in the subsurface or bottom tiers.

At the present time, organic soils of British Columbia are being studied in greater detail in an attempt to test the workability of the tentative classification, and to establish a range of characteristics for each category, i.e., Order, Great Group, Sub-group, Family and Series.

Organic soils in the Lower Fraser Valley have been studied more than those of other areas and it appears that members of all three groups, viz., Fibrisols, Mesisols and Humisols occur. The Fibrisols correspond to the undecomposed acid sphagnum peats which are concentrated in the large bogs at the western end of the valley. Their more complete classification would be Stratic Mucinic Fibrisols. They are often quite deep and are used for peat harvesting, blueberry and cranberry production. The classification of Sedimentary peats in the proposed system appears to present some difficulty.

The humisols correspond to mucks that are well decomposed. These are more common in the central and upper part of the Lower Fraser Valley. They, too, are acid and range in depth from about 18 inches to 25 feet or more. The nature of the original vegetation is often hard to determine, but generally appears to be sedge and woody plant remains (Sprout, 1967). The Mesisols are of an intermediate stage of decomposition. Some representative values for some of the soils of the three groups are given in Table 1:-

TABLE 1: Representative chemical data for surface layers of three organic soils of the Lower Fraser Valley (Paynton, 1965)

Soil Series	Type	Tentative classification	pH	C.E.C. me/100 gm	C %	N %	C/N ratio
Trigg	Sphagnum peat	Fibrisol	3.1	198.9	57.3	.64	89
Lumbum	Muck	Mesisol	4.0	135.0	55.2	1.34	41
Gibson	Muck	Humisol	3.2	164.2	49.0	2.49	21

Characteristically of the organic soils of the Lower Fraser Valley, all three samples included in Table 1 are strongly acid and have a high cation exchange capacity. Also, as the degree of decomposition increases, the percent carbon decreases, the percent nitrogen increases and the ratio of carbon to nitrogen narrows.

Many of the organic soils of the area are stratified, with layers of different degrees of decomposition, and therefore would be classed as Stratic.

Organic soils of the valleys of the Coast Range, on Vancouver Island and the Queen Charlotte Islands in general will be similar to those of the Lower Fraser Valley.

The most significant characteristics of organic soils appear to be the origin and the degree of decomposition, as recognized in the proposed classification. Other soil properties commonly measured often reflect the degree of decomposition and this is illustrated in some results from the Peace River area presented in Table 2. The results represent mean values for a number of organic layers, classified as fibric, mesic or humic on the basis of degree of decomposition.

TABLE 2: Some chemical and physical characteristics of sphagnum-derived peat layers from the Peace River Area.

Classification	Ash %	C.E.C. me/100 gm (D.A.F.)[1]	Base Saturation %	N % (D.A.F.)[1]	pH %	Water Holding Capacity %
Fibric	5	160	43	0.7	4.4	1500
Mesic	18	232	84	1.7	5.0	680
Humic	33	354	98	3.2	6.1	330

[1] Dry Ash Free Basis

The degree of decomposition often varies considerably with depth within a given soil, and this is one of the reasons why the surface vegetation is a relatively unreliable guide to the general characteristics of underlying organic material.

The recently completed soil survey of the Peace River Block of British Columbia (Farstad et al., 1965) covers 4,363,105 acres, of which 258,695 acres (or about 6 percent) is classed as organic soils. Within the organic area, two series were mapped, Eaglesham, which accounted for 40,095 acres, mainly of sedge origin, and Kenzie, 195,810 acres, principally of sphagnum origin. To the northeast of this block, the proportion of organic soils is much higher and permanently frozen layers occur. The organic soils of this region, therefore, are probably mainly cryic Fibrisols and cryic Mesisols.

Van Ryswyk (1966) recently provided some interesting data on organic soils of the central interior, showing that some of these contain solubles and are saline. Some of his results are given below:-

TABLE 3: Chemical characteristics of some organic soils from the Central Interior of British Columbia (Van Ryskwyk, 1966)

pH Range	Conductivity of the Saturation Extract mmhos/cm	Number of Samples
7.0 - 7.8	.6 - 2.8	12
6.0 - 6.9	.4 - 2.3	16
4.9 - 5.9	.3 - .8	16

It is evident that within the wide distribution of organic soils in British Columbia many kinds occur, differing in chemical and physical properties. The tentative system of classification proposed by the National Soil Survey Committee shows promise, but a great deal more research is needed on the organic soils of British Columbia in order to characterize and classify them adequately. With the aid of such work it appears possible that the classification may permit interpretation in terms of Engineering applications.

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SECONDARY CONSOLIDATION OF PEAT

by Ronald G. Doyle. P. Eng.

Compressibilty is one of the most important characteristics of peat that concerns Soil Mechanics and Foundation Engineers. Laboratory consolidation tests which determine the compressibilty indices and constants are a means of estimating the settlements that will occur at projects that are built on a peat deposit. This talk will describe and discuss some of the complications that arise in the interpretation of secondary consolidation.

The standard consolidation test procedures that have been developed for clays are also usually used for peats. Although peat varies from clay in that it is much more compressible and has a secondary consolidation which continues for a long period of time. Following the discussion of the test results and their interpretation, some minor modifications to the standard test procedures will be suggested.

The consolidation process is arbitrarily divided into two portions, primary and secondary. Primary consolidation is defined as the portion of the consolidation process where the rate of consolidation is governed by the hydrodynamic effect, as described by Terzaghi's theoretical equation. Excess pore pressures are associated with primary consolidation. Secondary consolidation, by definition is the consolidation which occurs after the excess pore pressures are at or near zero. In most cases secondary continues as a straight line when plotted with deflection versus the logarithm of time. Some field and laboratory measurements of secondary indicate a break in the straight line, with a resulting steeper slope as secondary progressess. Figure 1 shows the complete series of deflection versus time curves for a consolidation test. The deflection reading have been adjusted in order to compress all the curves on the one plot. The fifth curve shows an example of the break in the secondary curve. The initial secondary curve is well defined between 40 and 400 minutes. Then the slope steepens for the remainder of the load increments. Although the slope steepens the actual rate of secondary does not increase. If this data were plotted to a natural scale the deflection would be proceeding at an ever decreasing rate.

There are many theories as to the mechanism that causes this break in the slope of the secondary curve. These theories could possibly be prejudiced by the arbitary definition of secondary consolidation. Many researchers feel that if secondary deviates from its straight line there must be some mechanism or linkage breakdown in the soil that causes this deviation. Let us now consider another possible cause for this break in slope that only depends on the test procedure and the method of test result presentation.

Consolidation test results are most commonly presented and compared by means of a semi-log plot. Therefore a brief description of the geometry of lines and curves on a semi-log plot will give valuable insight for the interpretation of the test results.

First consider a straight line on the semi-log plot as shown in Figure 2. The general equation on this figure gives a straight line when $n = 0$. The general equation shown actually only adds or subtracts a constant value of time from the data required to plot the straight line. For example curve $n = 1$ adds 900 minutes to all the deflection values of curve $n = 0$. In other words we have a secondary consolidation curve but we do not have the correct time at which secondary begins. Thus by changing the reference of zero time two types of plots occur, those with less than and those with slopes greater than the original straight line plot. It will be noted however that all the plots eventually approach a slope of -b (the deflection for one log cycle). From the general equation it can be shown that all plots with n less than zero approach within 5% of slope -b at 1¼ log cycles past t_o and to within 1% at 2 log cycles past t_o; and with n greater than zero they approach -b within 5% at n plus 1¼ log cycles and within 1% at n plus 2 log cycles past t_o. The curves with slopes less than -b are the ones of interest for the interpretation of secondary consolidation.

Now an example of how these curves can occur on plots of secondary consolidation. Figure 3, with compression versus the log of time in minutes, shows only the secondary portion of a consolidation curve. This is the seventh load increment (3.81 kg/cm^2) on a peat sample 8 inches diameter by 5 inches initial height. In this test the secondary consolidation continued as a straight line between approximately 7,000 and 70,000 minutes. The arrows indicate the time when small loads have been added. For example at t = 28000 minutes a load of 0.03 kg/cm^2 was added. This load did not interrupt the straight line trend of the secondary consolidation. Also a further increase in load of 0.06 kg/cm^2 was added and again the secondary continues as a straight line. However, if the consolidation curve is plotted assuming zero time when this load was applied, we obtain the lower curve. This curve indicates an initial slower rate of secondary which increases in slope with respect to time. As pointed out on Figure 2. which showed the geometrical relationship of curves on the semi-log plot, this increase in slope will continue until it approaches the slope of the original line. This example shows that there can be an apparent increase in the slope of the secondary consolidation curve if the zero time for the secondary is assumed to be the time at which the load increment is applied; or alternately it suggest that the secondary curve will plot as a continuous straight line if the correct zero time can be established.

Figure 1 shows the compression versus log time curves for a complete consolidation test. The test was completed on a 2½ inch diameter by 1 inch sample using the standard consolidation test procedures . The point to note is the fifth curve which shows a break in the secondary consolidation at approximately 600 minutes. You will also note that this break occurs at approximately one log cycle past the end time of the proceeding load increments. This is fairly close to the time that would be predicated by the geometry to come to within 5% of what is possibly the true slope of the curve. This fifth curve was at a load of 2 kg/cm^2.

Two ways that breaks in the secondary consolidation curve can occur on plots of test results have been demonstrated,

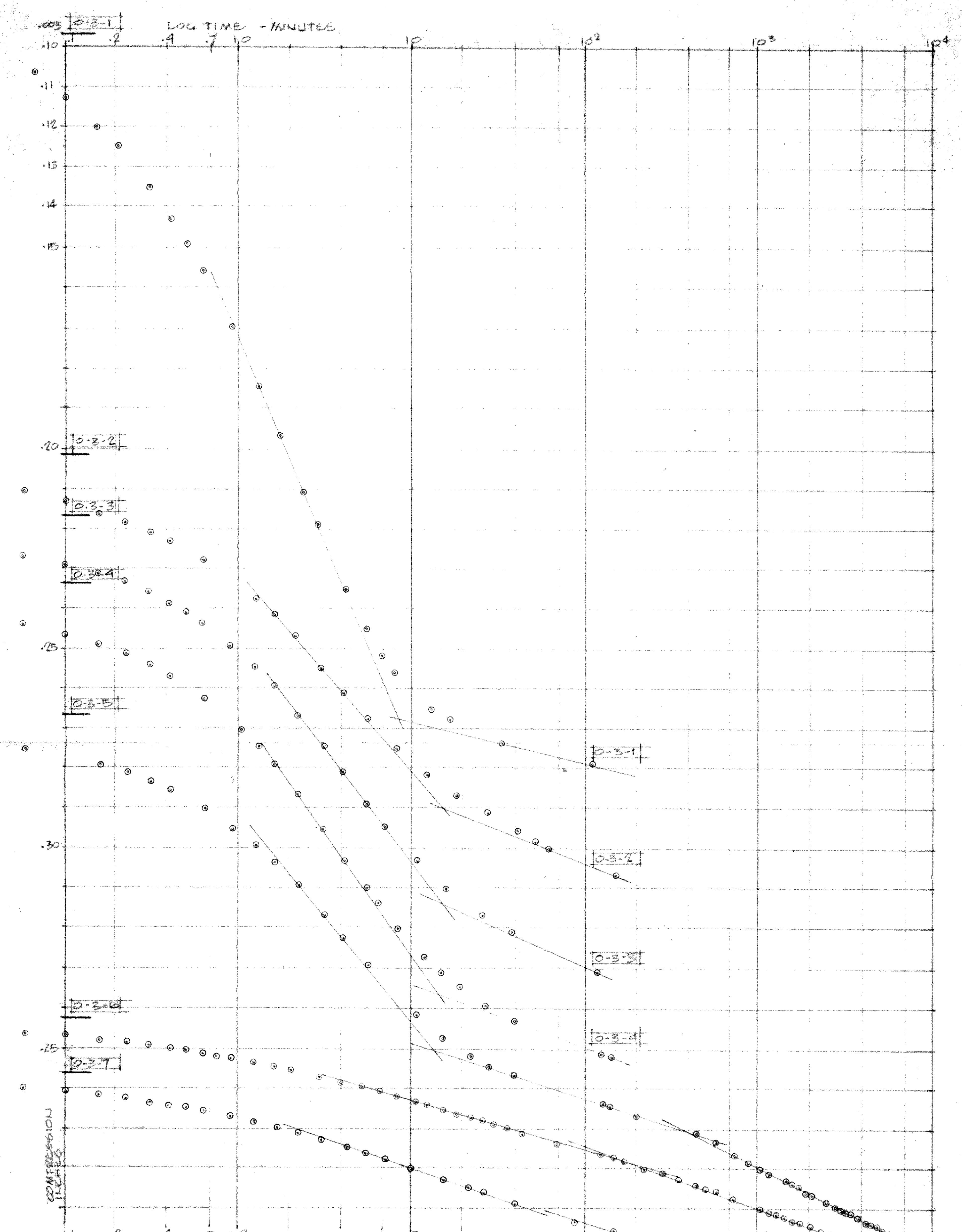
1. from plotting, where a small load is applied, the compression from this load does not deviate from a previously established straight line secondary curve; but if the time of application of the small load is considered as zero time a curve with increasing slope will result.

2. from the test procedure, as shown on Figure 1 the break occurs at approximately one log cycle of time past the final time of the previous load increments.

Now the question is what laboratory test procedure is required to determine the correct secondary curve for any particular applied load?

One possible test procedure is described by Figure 1. that is, apply the load increments only for the time required to obtain 100% primary, prior to the load at which the secondary rate is wanted, then run the test to at least 2 or 3 log cycles of time past the end of primary consolidation. In other words one consolidation test is required for each particular load at which the secondary consolidation rate is desired.

In conclusion, I realize, that a great deal of laboratory work is still required to completely verify the theory presented. However, in the interpretation of consolidation tests and in the prediction of secondary rates from field measurements, the question to keep in mind is "What is the real zero time for the secondary curve?" Then by choosing the proper zero time, the secondary consolidation will continue as a straight line on a semi-log plot; as nature intended.



LOAD NO.	σ_T Kg/CM²	2Hf IN.	2H(e100) IN.	e100	b	Cs
O-3-0		.9820	.9820	14.10		
O-3-1	.129	.7760	.7875	11.10	.0103	.0131
O-3-2	.258	.6716	.6879	9.58	.0166	.0241
O-3-3	.516	.5578	.5748	7.84	.0193	.0336
O-3-4	1.03	.4417	.4554	6.00	.0154	.0338
O-3-5	2.06	.3012	.3580 .3360	4.51	.0114 .0226	.0318 .0672
O-3-6	4.12	.2476	.2645	3.07	.0115	.0580
O-3-7	8.35	.1986	.2329 .2123	2.47	.0126	.0593

TEST O-3 COMPRESSION VS LOG TIME

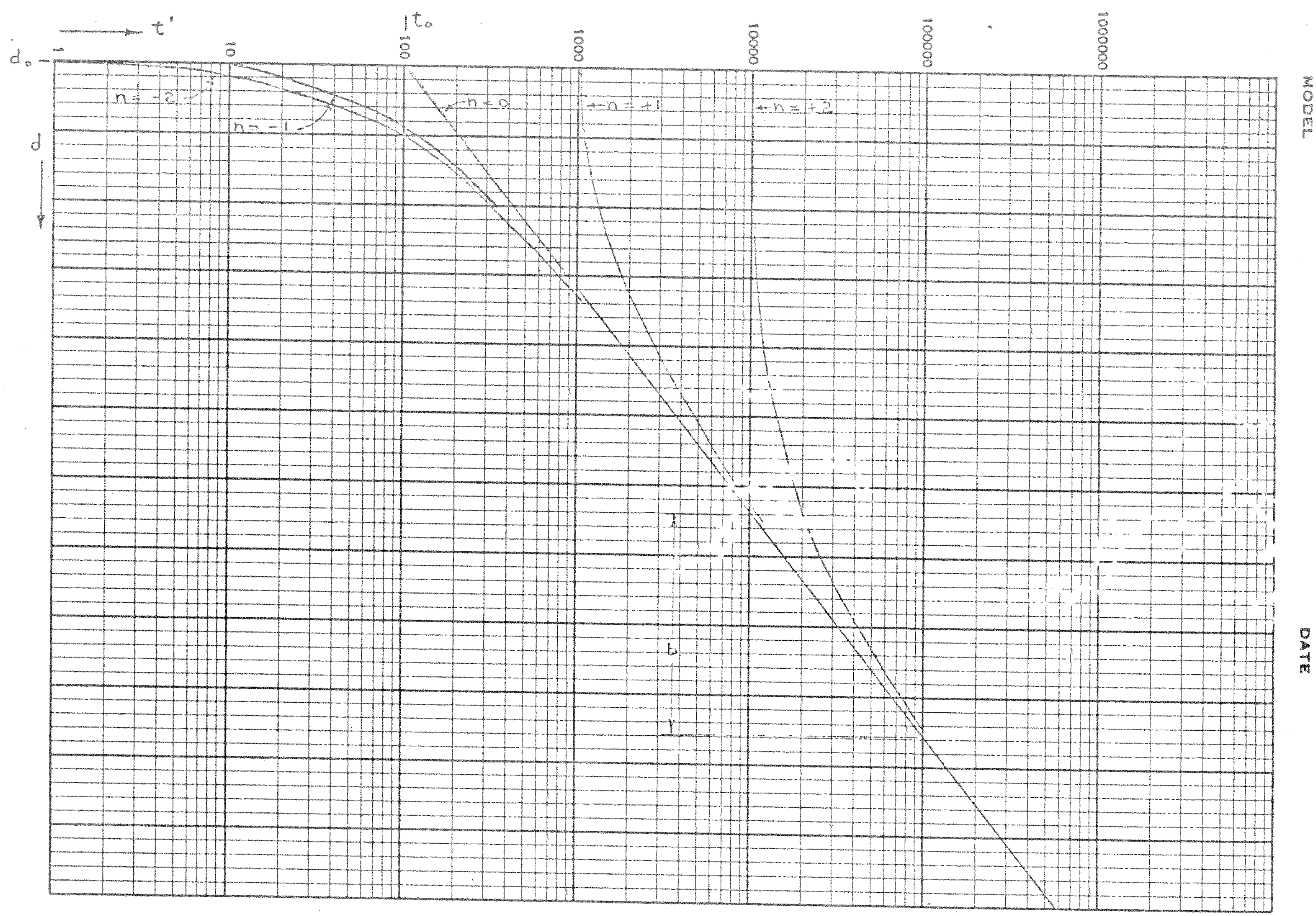


FIGURE 2 PLOT OF FUNCTION $d = d_0 - b \log\left[\frac{t'}{t_0} - (10^n - 1)\right]$

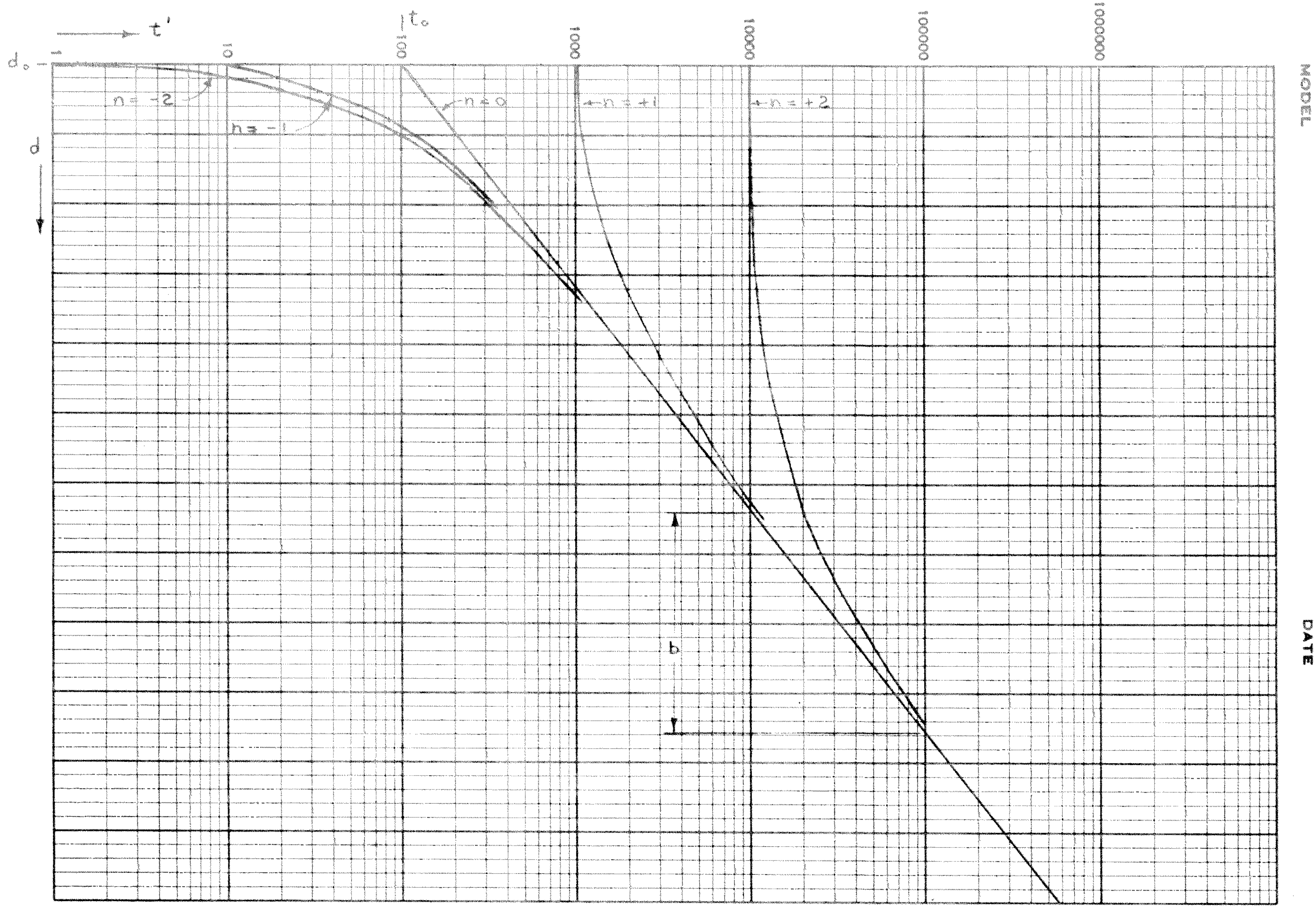


FIGURE 2 PLOT OF FUNCTION $d = d_o - b \log\left[\frac{t'}{t_o} - (10^n - 1)\right]$

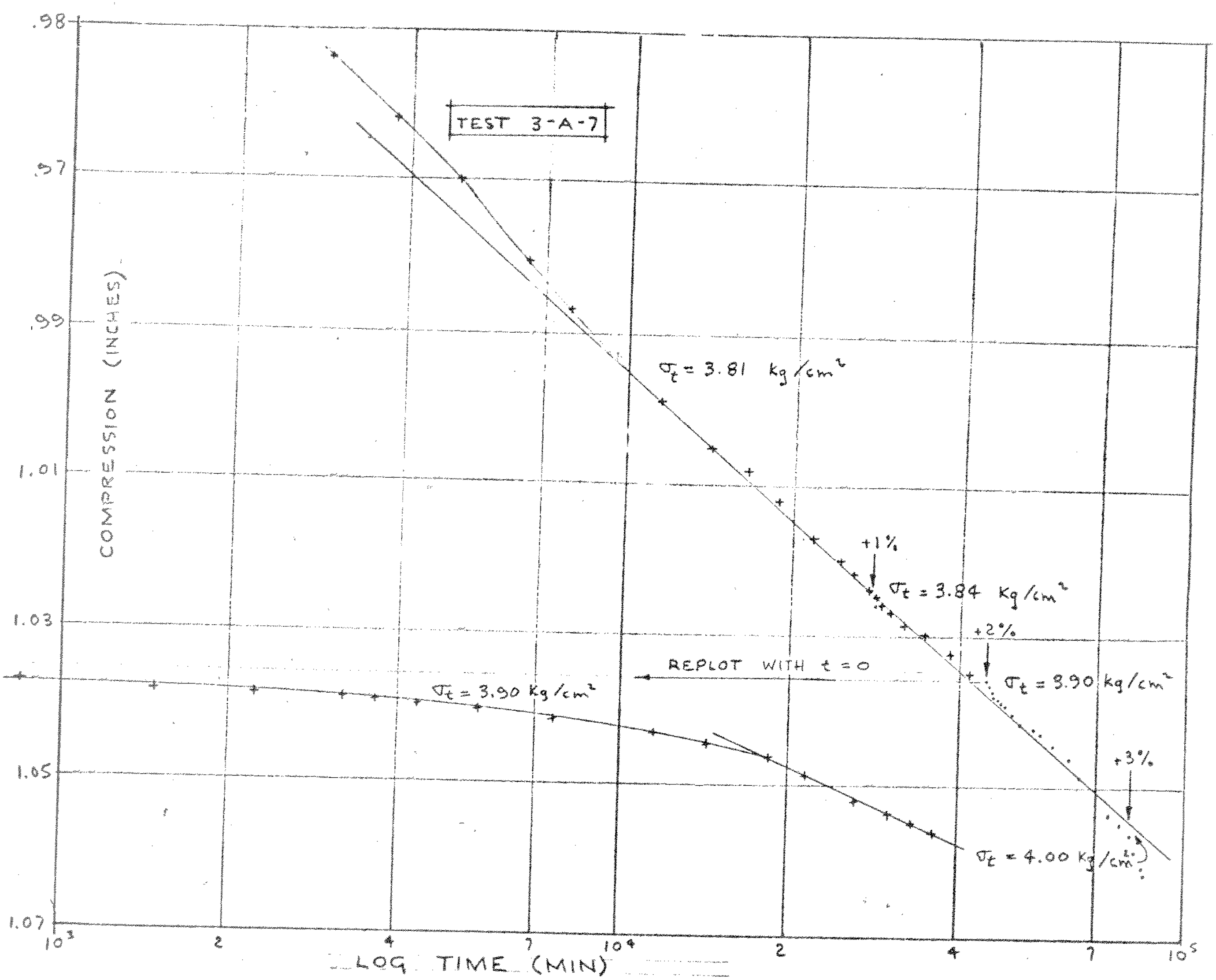


Figure 3 Compression vs log time, test 3-A-7

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CONSOLIDATION OF FIBROUS PEAT

Introduction

This paper summarizes some of the results of a preliminary investigation on laboratory consolidation test procedures for fibrous peat. Graphs of the results are attached.

Purpose

The purpose of this test program was twofold:-

1. To determine the effect of using different Load Increment Ratios and different Load Durations on a standard consolidation test of fibrous peat.
2. To probe a little further into the mechanism of sécondary settlement in fibrous peat.

Test Program & Material- See Test Data Summary FIG.3-1

It should be noted that precautions were taken to ensure rapid drainage, to avoid contamination, to reduce side friction and generally to keep all test conditions similar except for the desired variable factors.

A natural "undisturbed" lump sample of fibrous peat was used rather than some type of remolded material as it was felt that in this way more useful information could be gained.

Results

1. a. Typical Settlement vs. Log Time Curves For LD = 24 Hr. Nominal/LIR 0.33, 1.0, & 3.0 At Approx. The Same Applied Pressure. FIG 1

   b. Definitions

Primary Settlement - Hydrodynamic settlement occurring under conditions of significant excess pore water pressure.

Secondary Settlement - Remaining settlement occurring without significant excess pore water pressure. Typically this settlement is the straight line portion of the settlement - log time graph coming after completion of the primary settlement.

Coefficient of Secondary Settlement

$$= \frac{\text{increment of settlement per log cycle of time}}{\text{initial height of sample}}$$

(or sometimes height after settlement under previous applied load)

Load Increment Ratio

$$= \frac{\text{increment of load}}{\text{previous total load}}$$

Curve Types classified as Type 1 & Type 2 following previous work on clay - see graph inset.

100% Primary Consolidation Point - Determined by standard Tangent Intercept Method for Type 1 curves and by Tangent Point Method for Type 2 curves - see graph inset

1. c. NOTE i. Similarity of amount of settlement at 100% primary.

ii. Similarity of secondary rate.

iii. Decreasing initial settlement with decreasing LIR.

iv. Test #1 & #6 reached 100% primary in 10 to 20 mins.

v. Test #2 reached 100% primary in 100 to 200 mins. (i.e. approx. one log cycle in time longer)

vi. Only test #2 had curve Type 2 (beyond preconsolidation pressure = 0.12 Kg/cm^2)

2. a. Rapid tests - Settlement vs. Log p @ approx. 100% primary FIG. 3

LD = 15 mins

LIR = 0.33, 1.0 & 3.0

b. Note i. Close agreement #4 & #5

ii. Poor agreement # 3A & #3B - probably due to 100% primary point not being reached.

3. a. 24Hr. tests - settlement vs. Log p at 100% primary FIG. 4

LD = 24Hr. (Nominal)

LIR = 0.33, 1.0 & 3.0

b. Note i. Close agreement #2 & #1

ii. Close agreement #2, #1 & #6 up to p = 1.0 Kg/cm^2 (60% settlement)

c. Comparison Rapid & 24 Hrs. Tests - Settlement vs. Log p @ 100% primary

d. Note i. Similar shape and therefore similar C_c

ii. Rapid tests show increased amount of settlement (±10%)

4. a. Comparison combined Primary + Secondary settlement vs. Log p FIG. 2
for nominal 24 Hr. tests LIR = 0.33, 1.0 & 3.0

b. Note i. Difference between Primary curve & Primary + 2 Log Cycles Secondary curve is amount of secondary settlement actually measured. This implies that rate of secondary is proportional to applied pressure. Only test #2 shows appreciable scatter.

ii. Small differences in measured rate of secondary are magnified in successive extrapolations. However scatter beyond $p = 1.0$ Kg/cm^2 reflects mostly scatter in primary settlement since the values of secondary rate become more consistent at higher applied pressures.

c. The hypothesis of unique Settlement - Log Applied Pressure curve at 100% primary consolidation appears to offer a plausible interpretation of the results of this test series on fibrous peats. It is interesting to note that the same result was obtained for remolded clay by D.W. Taylor in 1942 and H.E. Wahls in 1962.

5. a. Rate of Secondary Settlement vs. Log p.

b. Note i. Good agreement #1 and #6

ii. Rough agreement #1, #6 & #2

iii. Also spot results from test #7 & #3B plot in good agreement

iv. Probable reason for low values of #2 was a larger than normal load duration under the previous load increment. Certainly the apparent scatter of results for test #2 is connected with the fact that is was the only test with a Type 2 curve.

v. Peak values of secondary appear at higher applied pressure than preconsolidation pressure determined in normal manner.

FIG 2 AGAIN

Conclusion

The main practical conclusion to be drawn from this test series was that variation of the Load Increment Ratio between 0.33 and 3.0 and Load Duration between the 100% primary consolidation point and about 5 days can be done without significantly affecting the primary S-log p curve for fibrous peat providing that the settlement used is that occurring at 100% primary consolidation. Thus accelerated tests using LIR greater than 1.0 could be expected to provide all the information usually obtained with standard laboratory consolidation tests using LIR = 1.0 at 24 hr. load periods.

A further important result is that close similarity between the laboratory settlement behaviour of a naturally occurring completely organic material - fibrous peat - and completely inorganic clay soils. The implication being that whatever mechanism is postulated for the secondary settlement of clay must also apply to fibrous peat.

Finally there are included a number of laboratory test results which have some bearing on secondary settlement in fibrous peat.

A. Settlement at 100% primary vs Log p — FIG. 7

Agreement of test #8 without ring with tests #4 & #5.

B. Settlement vs. Log time — FIG. 6

Comparison of typical curves of test #4 & test #2 showing different curve types.

C. Settlement vs. Log time <u>FIG. 8</u>

i. Zero pore pressure at 100% primary point for type 1 curves.

ii. <u>Upside down</u> type 1 curve for unload 3.50 to 0.4 Kg/cm^2

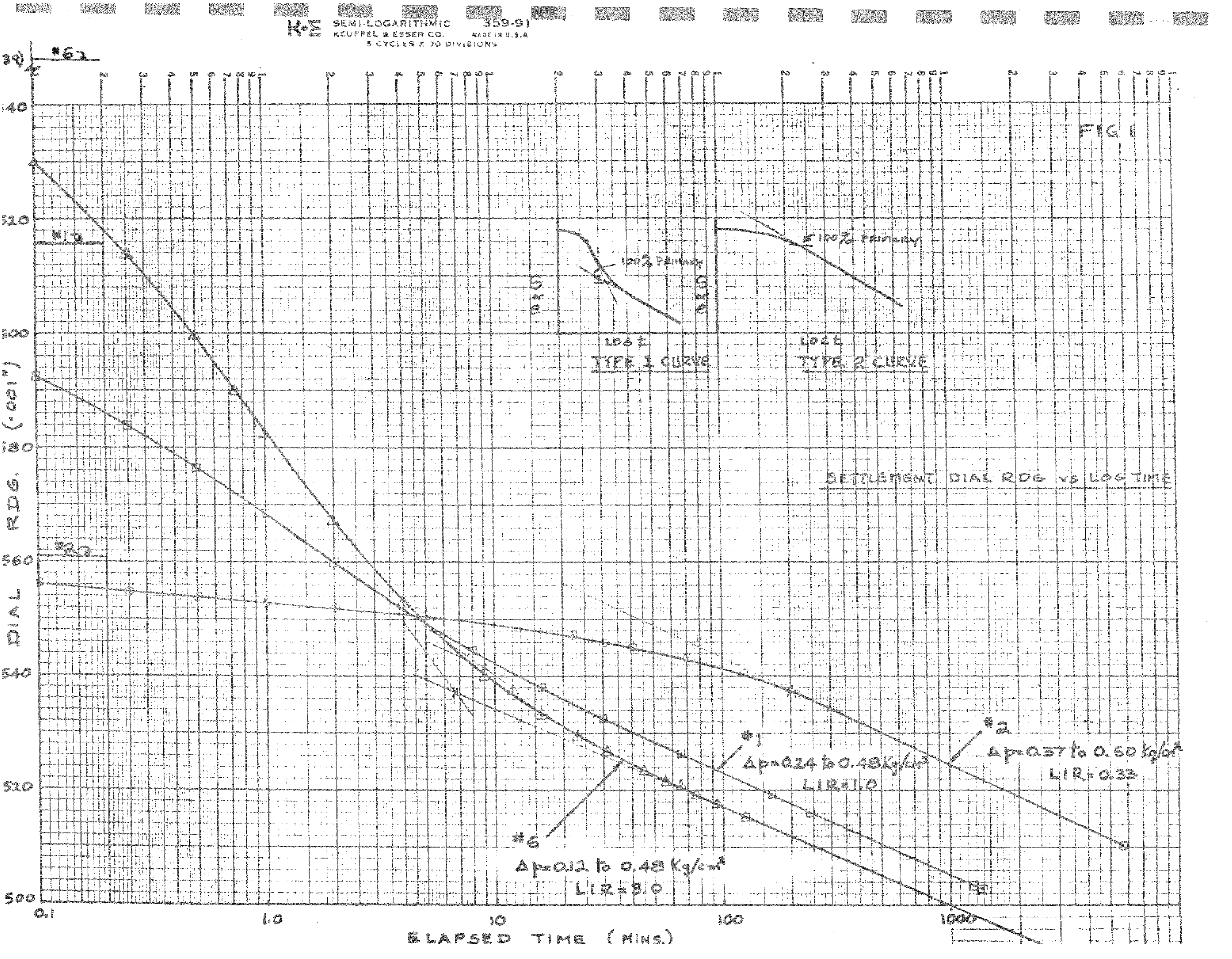
iii. <u>Reversed</u> (i.e. upwards) secondary swelling curve - straight line on log time plot.

D. Settlement vs. Log time <u>FIG. 9</u>

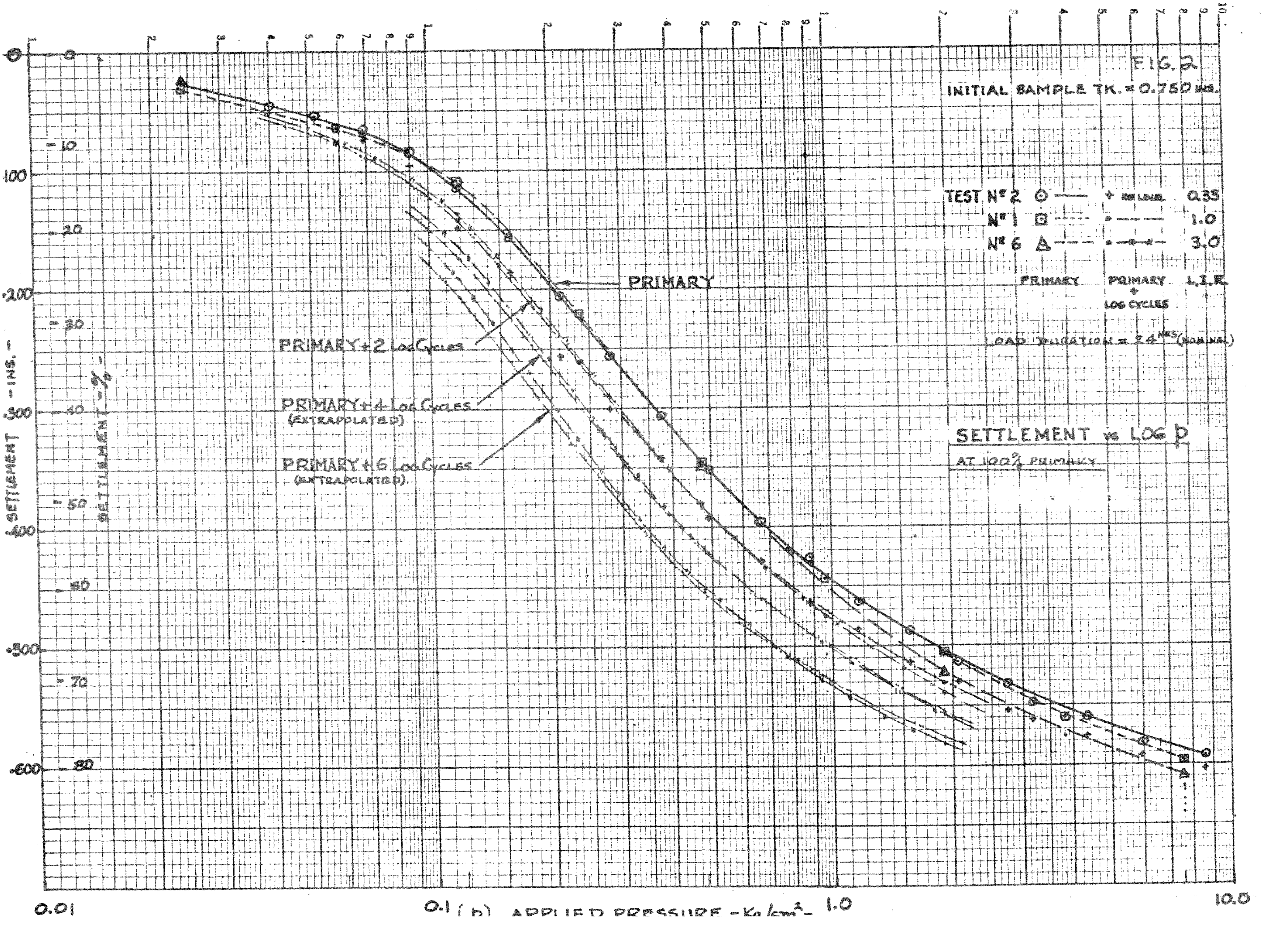
Similarity of behaviour of synthetic sponge material & fibrous peat.

<u>G. Macleod</u>

<u>February, 1967.</u>



K&E SEMI-LOGARITHMIC
KEUFFEL & ESSER CO.
359-91
MADE IN U.S.A.
5 CYCLES X 70 DIVISIONS
39)
#6
FIG 1
#1
#2
100% PRIMARY
LOG t
TYPE 1 CURVE
TYPE 2 CURVE
SETTLEMENT DIAL RDG VS LOG TIME
Δp=0.24 to 0.48 Kg/cm²
LIR=1.0
Δp=0.37 to 0.50 Kg/cm²
LIR=0.33
Δp=0.12 to 0.48 Kg/cm²
LIR=3.0
DIAL RDG. (.001")
ELAPSED TIME (MINS.)
0.1
1.0
10
100
1000
500
520
540
560
580
600
620
640



K+E SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS
FIG. 2
INITIAL SAMPLE TK. = 0.750 INS.
TEST Nº 2
Nº 1
Nº 6
0.33
1.0
3.0
PRIMARY
PRIMARY + LOG CYCLES
L.I.R.
LOAD DURATION = 24 HRS (NOMINAL)
SETTLEMENT vs LOG P
AT 100% PRIMARY
PRIMARY
PRIMARY + 2 Log Cycles
PRIMARY + 4 Log Cycles (EXTRAPOLATED)
PRIMARY + 6 Log Cycles (EXTRAPOLATED)
SETTLEMENT - INS. -
SETTLEMENT - % -
.100
.200
.300
.400
.500
.600
10
20
30
40
50
60
70
80
0.01
0.1
1.0
10.0
(b) APPLIED PRESSURE - Kg/cm² -

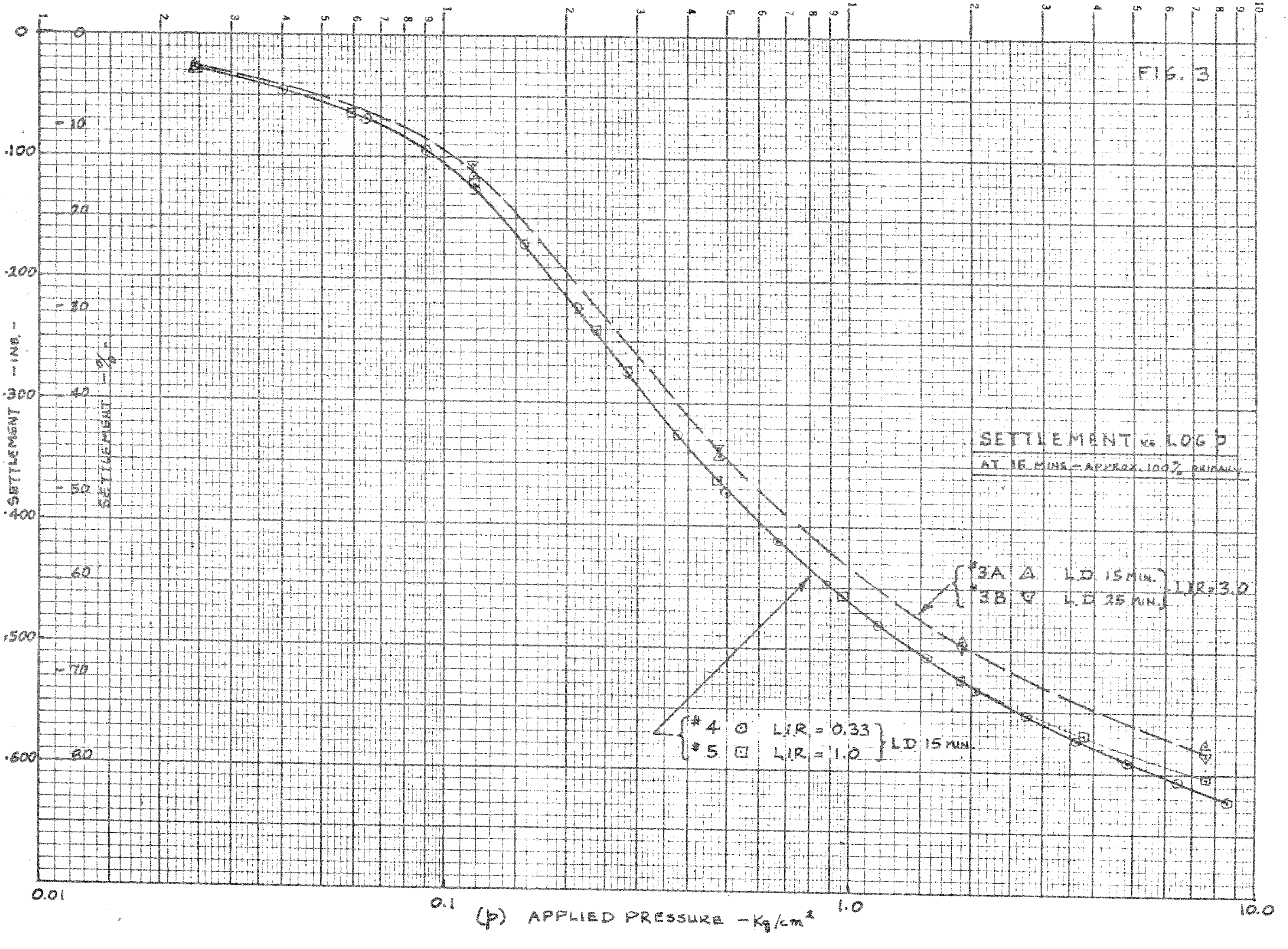


FIG. 3
SETTLEMENT vs LOG p
AT 15 MINS - APPROX. 100% PRIMARY
#3A △ L.D. 15 MIN.
#3B ▽ L.D. 25 MIN.
L.I.R. = 3.0
#4 ⊙ L.I.R. = 0.33
#5 ⊡ L.I.R. = 1.0
L.D. 15 MIN.
SETTLEMENT - INS. -
SETTLEMENT - % -
(p) APPLIED PRESSURE - Kg/cm²
0.01
0.1
1.0
10.0
0
.100
.200
.300
.400
.500
.600
10
20
30
40
50
60
70
80

TEST DATA SUMMARY

- FIG. 3-1 -

[TAKING G_s = 1.5]

TEST No	L.I.R. $\frac{\Delta P}{P}$	LOAD DURATION (NOMINAL)	INITIAL WATER CONTENT (%)	INITIAL VOID RATIO e	INITIAL SATURATION (%)	COMPRESSION INDEX - C_c (0.2 TO 0.5 kg/cm²)	OVEN DRY WT. OF SOLIDS (GMS)
2	0.33	24 HRS	1,025	16.0	96.0	9.6	5.28
4	0.33	15 MINS	1,030	16.6	93.2	9.4	5.14
1	1.0	24 HRS	1,140	18.1	94.2	10.0	4.72
5	1.0	15 MINS	1,120	17.9	94.2	9.2	4.79
8 (NO RING)	1.0	12 MINS	1,000	16.3	92.7	10.4	5.24
6	3.0	24 HRS	1,035	16.8	92.7	10.0	5.07
3A	3.0	15 MINS	1,045	17.3	91.3	9.4	4.97
3B	3.0	25 MINS	1,120	17.5	95.3	9.4	4.91
7	1.0 & 3.0	15 MINS & 24 HRS + 2 STOPS	1,130	18.6	91.4	10.4	4.61
AVE. VALUES			1,110	17.3	94	9.8	4.97

ALL SAMPLES WERE CUT CLOSE TOGETHER FROM SAME HORIZONTAL LAYER OF PEAT SAMPLE [LUMP SAMPLE (I) 2 FT BELOW GRADE]

SAMPLE DESCRIPTION
BROWN FINE FIBROUS PEAT, OCC. TWIGS OR ROOTS ±1/4", NOTICEABLE HORIZ. LAMINATIONS, FAINT ODOUR H_2S, VERY COMPRESSIBLE, SOME HORIZ. TENSILE STRENGTH
pH : 4 (approx.)

NOTE: ALL TESTS EXCEPT #8 WERE CARRIED OUT USING STAINLESS STEEL CONSOLIDATION RINGS LIGHTLY COATED WITH MOLYBDENUM DISULPHIDE GREASE : RINGS 2.50" I.D. & 0.750" HT. : TOP & BOTTOM DRAINAGE.

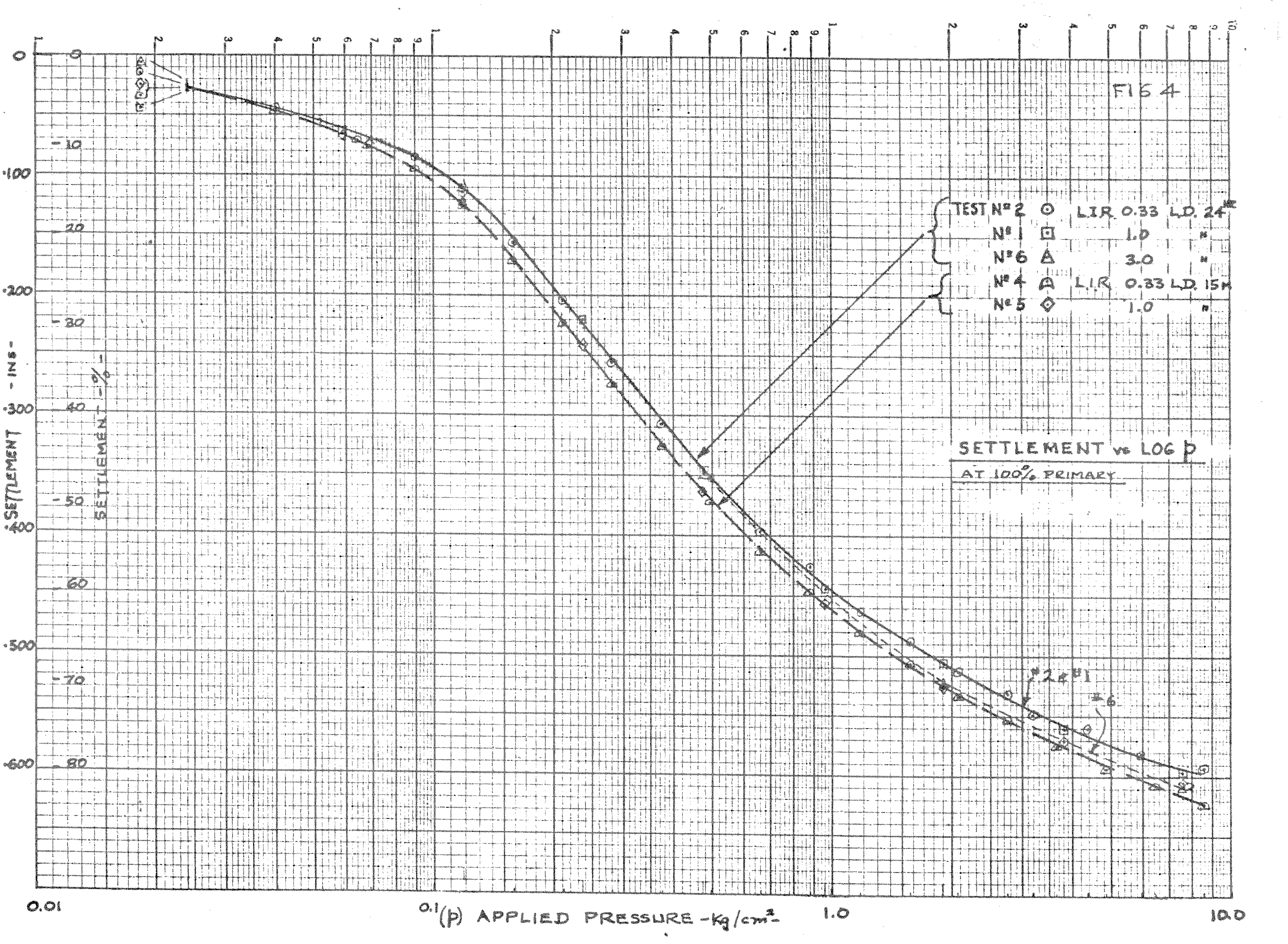
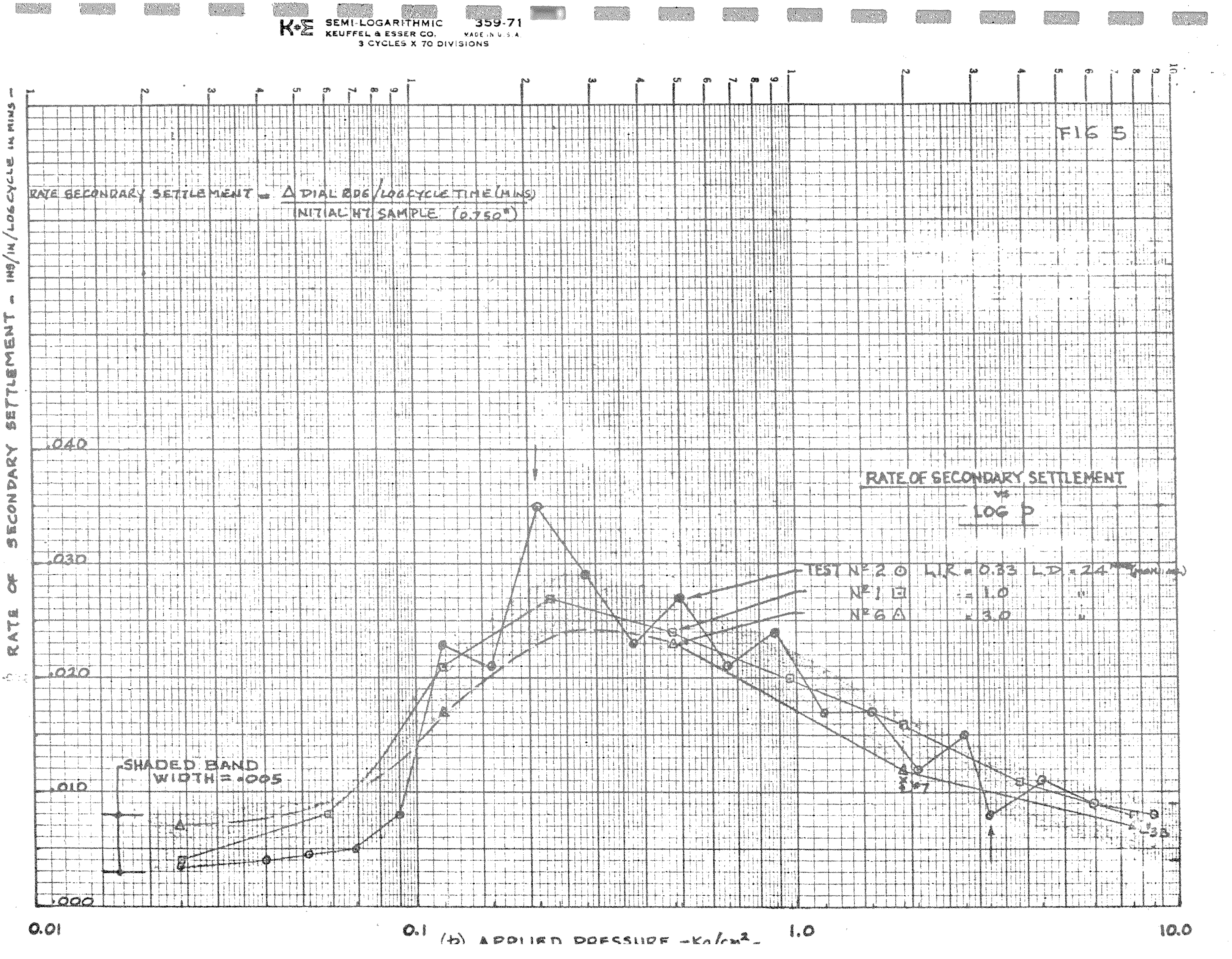
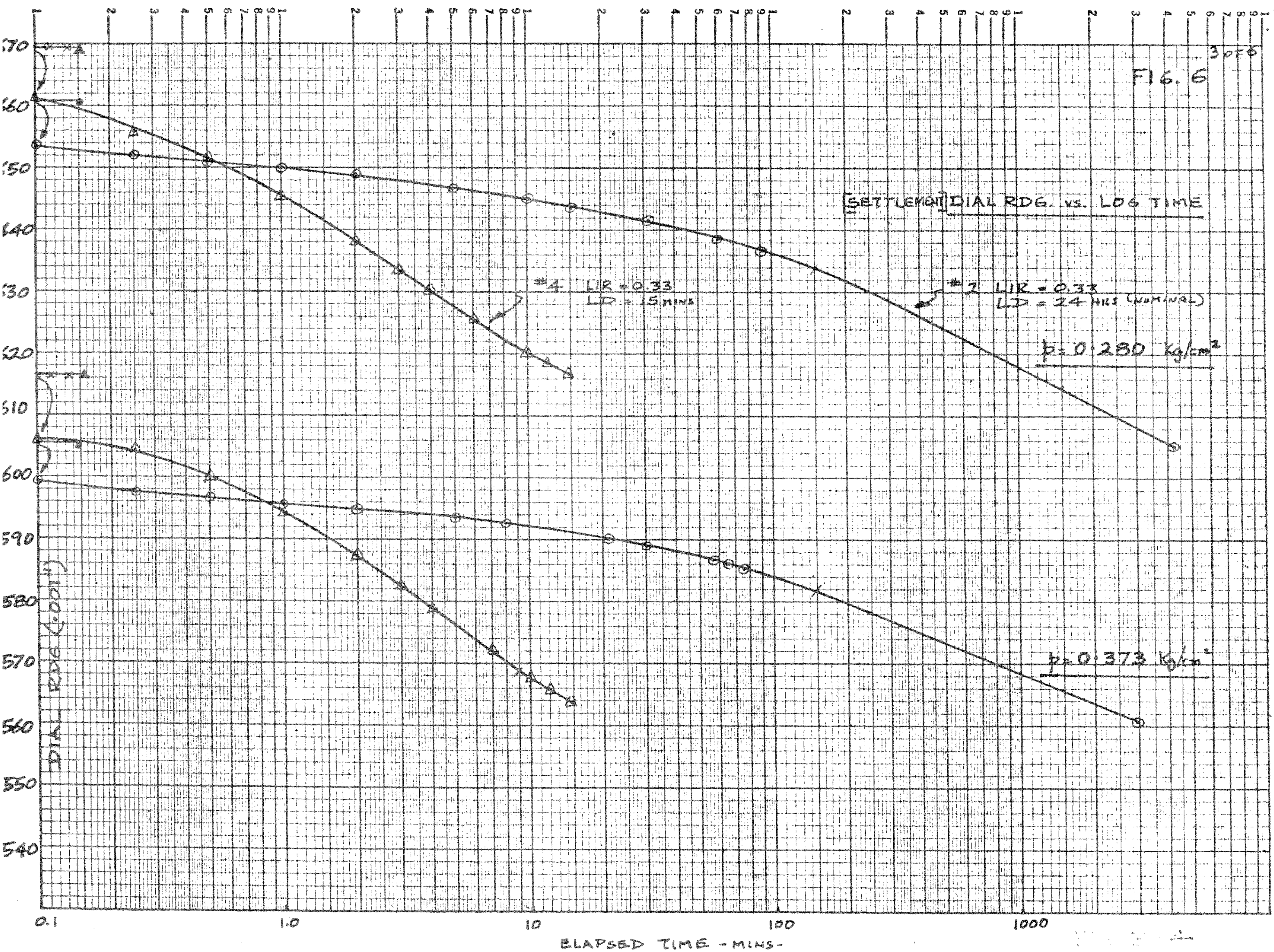


FIG 4
TEST Nº 2 ⊙ L.I.R. 0.33 L.D. 24 HR
Nº 1 ⊡ 1.0 "
Nº 6 △ 3.0 "
Nº 4 ◬ L.I.R. 0.33 L.D. 15 HR
Nº 5 ◇ 1.0 "
SETTLEMENT vs LOG p
AT 100% PRIMARY
#2 & #1
#6
SETTLEMENT - INS -
SETTLEMENT - %
(p) APPLIED PRESSURE - Kg/cm²
0.01
0.1
1.0
10.0
KEUFFEL & ESSER CO.
3 CYCLES X 70 DIVISIONS
MADE IN U.S.A.


K+E SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 70 DIVISIONS
FIG 5
RATE SECONDARY SETTLEMENT = Δ DIAL RDG / LOG CYCLE TIME (MINS) / INITIAL HT SAMPLE (0.750")
RATE OF SECONDARY SETTLEMENT vs LOG p
TEST No 2 ⊙ LIR = 0.33 LD = 24
No 1 ⊡ = 1.0
No 6 △ = 3.0
SHADED BAND WIDTH = .005
.040
.030
.020
.010
.000
0.01
0.1
1.0
10.0
(p) APPLIED PRESSURE - Kg/cm²
RATE OF SECONDARY SETTLEMENT - INS/IN/LOG CYCLE IN MINS

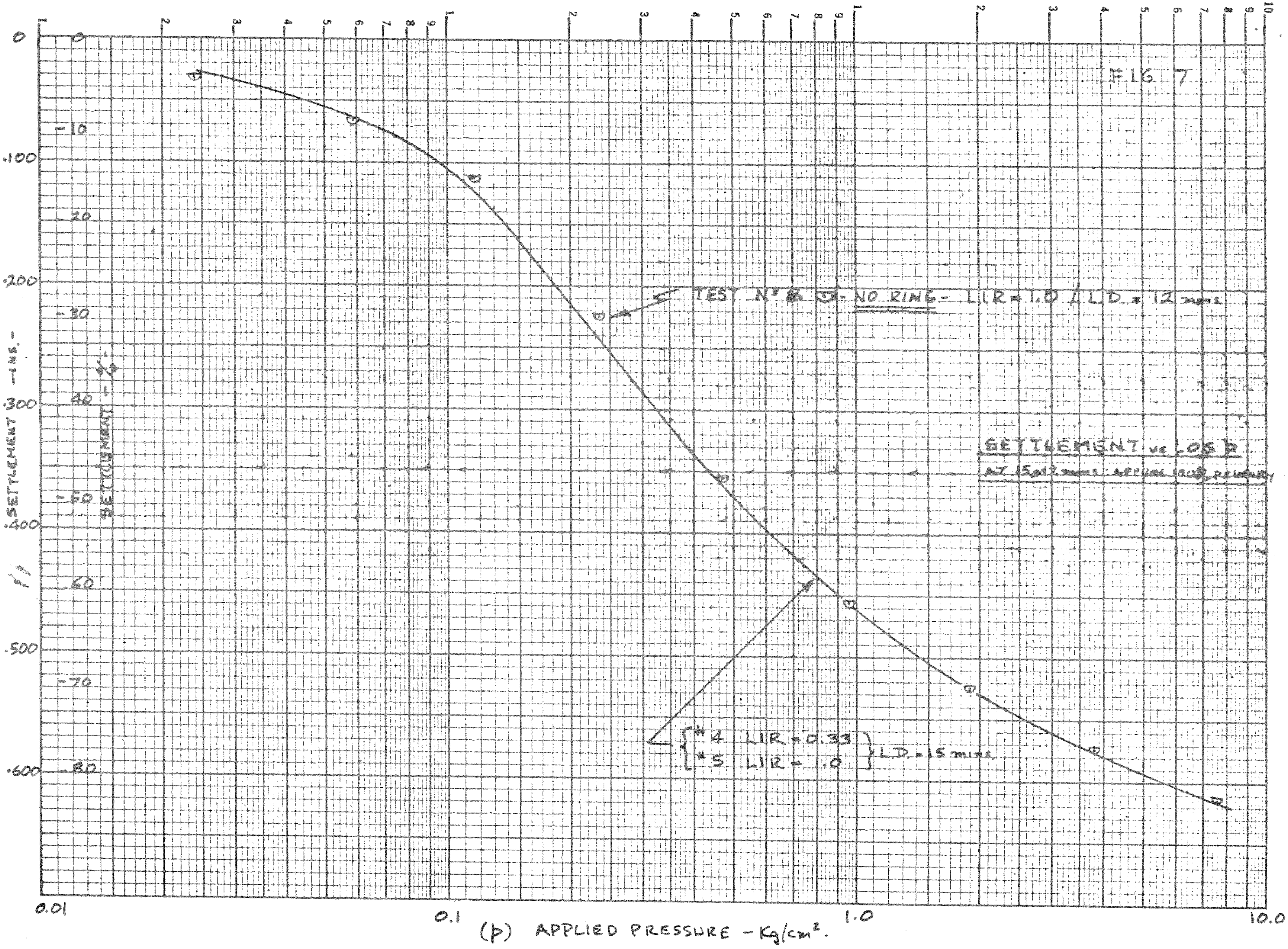
3 OF 6

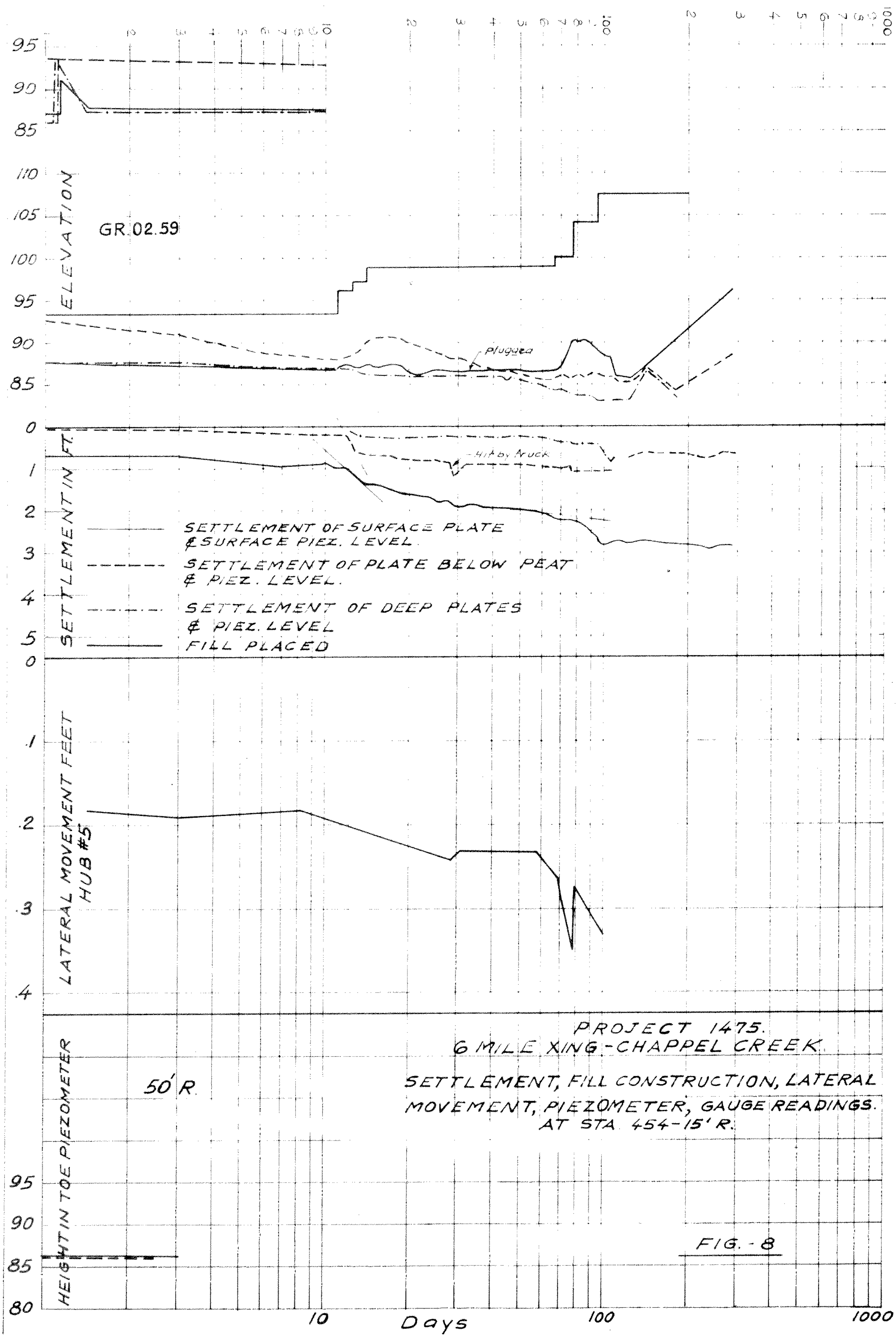
FIG. 6



KEUFFEL & ESSER CO. MADE IN U.S.A.
5 CYCLES X 70 DIVISIONS

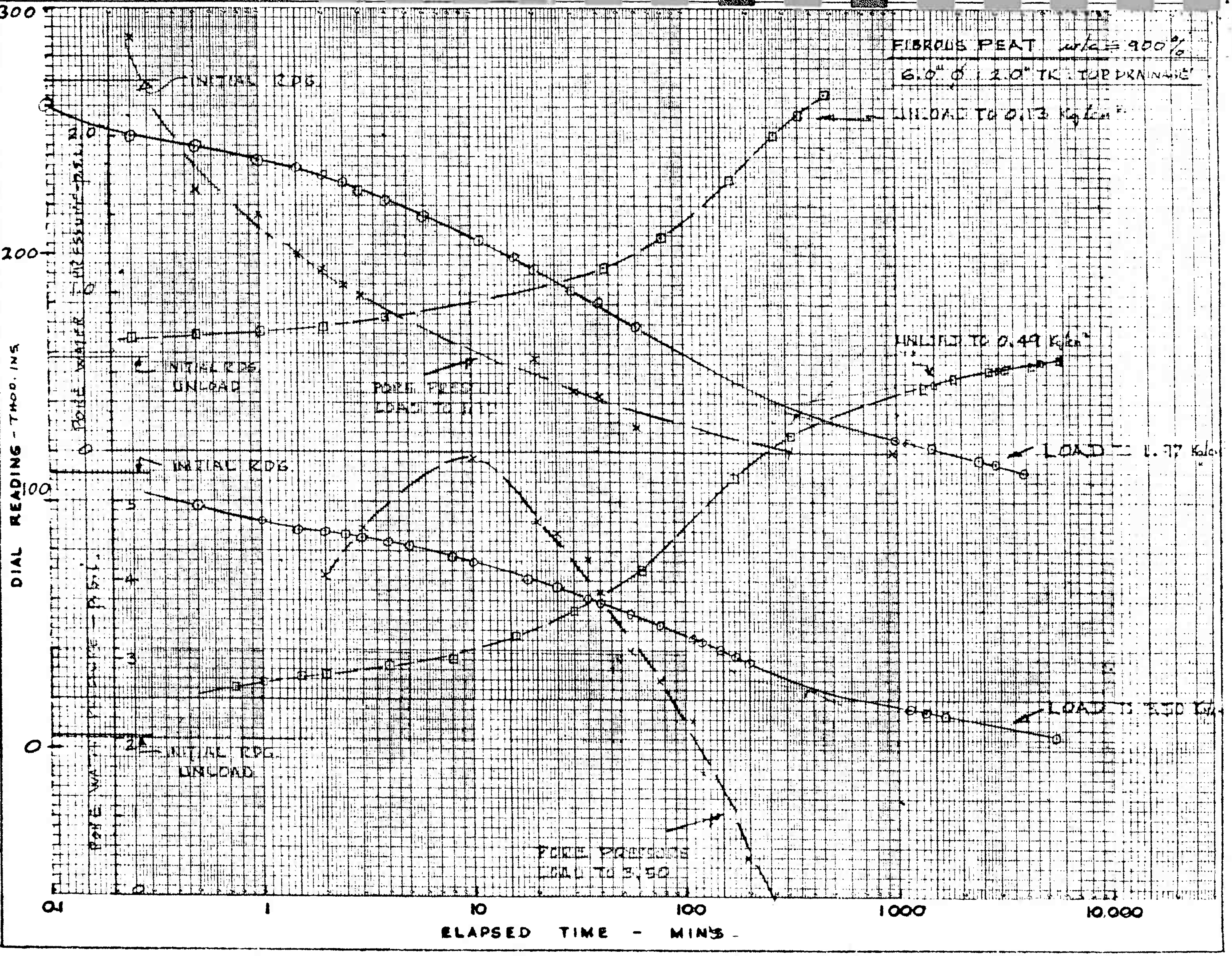
K+E SEMI-LOGARITHMIC 3 CYCLES X 70 DIVISIONS 46 5493 MADE IN U.S.A. KEUFFEL & ESSER CO.





ELEVATION
GR.02.59
Plugged
SETTLEMENT IN FT.
Hit by truck
SETTLEMENT OF SURFACE PLATE
¢ SURFACE PIEZ. LEVEL
SETTLEMENT OF PLATE BELOW PEAT
¢ PIEZ. LEVEL
SETTLEMENT OF DEEP PLATES
¢ PIEZ. LEVEL
FILL PLACED
LATERAL MOVEMENT FEET
HUB #5
HEIGHT IN TOE PIEZOMETER
50' R.
PROJECT 1475.
6 MILE XING - CHAPPEL CREEK
SETTLEMENT, FILL CONSTRUCTION, LATERAL
MOVEMENT, PIEZOMETER, GAUGE READINGS.
AT STA 454-15' R.
Days

FIG. - 8



CONSOLIDATION TEST | HOLE | S'PLE | SHEET 5 | JOB X-P

I was going to relate experience on the B.C. Hydro Freight Yard in Westminster, however, up-to-date level information is not available on this project and, on this account, it is not possible to compare predicted settlements with actual values. For this reason, I am substituting a description of experience on a lumber manufacturing plant in the lower mainland region.

The situation which existed in 1962 at the time of this study was that a plant was already in existence on the site. It was some 150 feet wide in two bays of 75 feet each by about 600 feet long. The building was on piles with the floor on fill. The finished product storage part of the operation involves floor loads of up to 800 pounds per square foot.

The soil profile was approximately 5 or 6 feet of fill on 23 feet of peat and clayey peat on another 20 feet of slightly organic clay silt and, finally, sand.

Naturally, the floor settled. At the time of inspection, the settlement had reached a figure up to 42 inches in places and it was stated that settlement was appreciable even before the plant went into operation.

The project under consideration was to design a foundation scheme for an identical building to be built right tight up against the existing building on its west side, that is along a 600 foot length. On this side, the sand fill had been placed some time ago and enough settlement had taken place in the peat that there was 18 feet of sand on approximately 5 to 8 feet of peat on another 25 or 30 feet of organic clay silt before the underlying sand was reached.

Economy eliminated a complete pile supported job. The solution was to use piles for the walls and columns and special equipment, with special treatment for the floor.

The special treatment consisted of:

1. Approximately 135 12" diameter sand drains were placed by the pile driving method in an area approximately 140 feet wide by 340 feet long. This length being the extent of that portion of the building built on the peat. The pattern was 20 feet centre to centre staggered which gave approximately 22 feet centres on the drains. These piles were driven until resistance indicated that the underlying sand had been reached.

2. Approximately 18 feet of sand was applied to the peat portion of the site. Grade settlement gauges were read and when deemed appropriate, this fill was removed, piles driven for the walls, columns and equipment and the building built in the usual way with floor slab last.

So much for the history. The following points will be of interest relating to the engineering aspect.

1. Preload without sand drains was estimated to have required between 3 and 4 years for primary consolidation. With sand drains, assuming the drains to be reduced to 50% of their

diameter due to mud smearing, it was estimated that 10 or 11 months would be required for 66% consolidation. It was found that movement had reduced to a very slow rate at 8 months and sand removal was started at that time. It was reported that during this removal when the sand drain sand was cut into, that water was oozing up.

2. Settlements for the floor with the above treatment are estimated to be 3½ inches in the first 10 years after construction with another 3½ inches in the next 10 years. The plant has been in operation about 2½ years now and settlement has reached 5 inches in places but with an average probably more like 3 inches.

3. It is worth noting that no downward movement of the pile caps has been observed despite the fact that the piles were designed for 20-Ton load and that no account was taken of negative friction.

The next project to be described sheds some light on the common decision facing foundation engineers with respect to soft sites, that is, whether to play safe and put the building walls and columns on piles and accept the differential settlements which result between the pile caps and the floor slab or whether to take a chance and place walls, columns and floor all on fill, accept some risk or cracks in the block walls but eliminate or greatly reduce any differential movements between footings and floor. In describing this job, it will be interesting to first observe the problem, then consider the usual solutions, thirdly, consider the actual solution, and finally, the performance of the building.

Stated in engineering terms, the problem was to provide adequate support for a storage building on a peat site with details as follows:

1. The soil profile is 8 or 9 feet of peat, moisture content 400 to 800% on organic clay silts to at least 15 foot total depth.

2. The building is 72 feet wide by approximately 200 feet long. The long walls in frame, one end wall consisting of 23 courses of 8-inch pearlite block (550 pounds per running foot) another end wall with pearlite block to 6 feet height and frame above that. Column spacing is 24 feet by 24 feet which with the roofing system used, applies a dead load of about 6 Kips and a design snow load of 24 Kips. Floor loads required are between 75 and 100 pounds per square foot for the concrete floor performing well enough for small diameter wheel fork-lift trucks to operate.

USUAL SOLUTIONS

(a) The usually conservative solution would be to pile the floor, columns and walls with a probable extra cost of \$1.60 to \$1.80 per square foot of building.

(b) Alternate designs would be to preload the site and either carry the building walls and columns on piles and place the floor on the preloaded fill or, as a sub-alternative, to place enough fill that walls could be carried on spread footings with enough fill under them to distribute the load to the peat and, in the case of the column footings, possibly to allow steel to project from these in both top face and by bending up steel from the bottom face to tie these with the concrete floor.

ACTUAL DESIGN

The steps in the actual design are approximately as follows:

1. Two to three feet of random fill was placed without benefit of sawdust and probably without very much compaction.

2. Walls were supported on wall footings with integral cast-in-place pony wall for a total height of 18 inches. The width of the walls varied from 18 inches wide for the frame wall to 36 inches wide for the concrete block wall (this would bring the dead load on the concrete block wall footing to approximately 300 pounds per square foot with an unknown amount of snow load over and above that). These wall footings were placed directly on the peat.

   The columns were supported on footings 3 feet square. This gives a pressure under dead load of 650 pounds per square foot, with an unknown extra amount for snow. The underside of these footings is thought to be within 12 or 16 inches of the peat.

3. The concrete floor slab was placed on 5 or 6 inches of drain gravel. In this connection, it is worth noting that shear transfer was provided by number 3 bars on 12-inch centres.

PERFORMANCE

Walls

This building was built in 1954 and, at this date, there are no cracks visible in the full height concrete block wall. There are some minor cracks in the block pony wall, but these were caused by a plumbing contractor digging a trench under this wall and then turning at right angles and going along parallel to the wall for a distance of 4 or 5 feet. When it is considered that this trench was 5 feet deep and was dug right beside the footing after the block had been placed, it is small wonder that there was some cracking. The cast-in-place footing walls show only minor shrinkage cracks.

Columns

There is no relative movement visible between columns and the floor slab excepting for two of the exterior columns where there has been some tipping of the footing in a direction which suggests an incipient failure towards the outside of the building where there is no confining effect of a floor slab.

In commenting on this design and performance, it is thought that the success has been attributable to one or more or all of the following features:

1. It is possible that since 1954, the building has not sustained the design snow load. This would be particularly applicable to the columns where loads seem to be the highest.

2. The surface mat of the peat was not disturbed and, as suggested by experience elsewhere, this provides tensile strength with an attendant increase in bearing capacity over what one would anticipate from laboratory tests.

3. The confining effect of the 2 or 3 feet of fill and, in the cases of the interior columns, the extra weight of the floor slab.

   __Note:__ The confining effect of fill is not working along the long frame wall, but here the load per square foot is quite low indeed. It is probably significant that the only sign of failure in the building was on two isolated exterior footings and that their failure is in the direction towards the side of least confinement.

NORTH THOMPSON HIGHWAY

CONSTRUCTION OVER PEAT

(Abridged)

A.G. REID, P.Eng.

Materials Testing Branch
Department of Highways

February 1967.

NORTH THOMPSON HIGHWAY

CONSTRUCTION OVER PEAT

(Abridged)

INTRODUCTION

Over 150 miles of new highway is under contract on the North Thompson Highway between Kamloops and the Alberta Border. See Fig.1. Construction was begun on all of five projects between 1964 and 1965. Work has progressed 50 to 70% towards completion. Numerous swamps could not be avoided by the new highway high standard alignment (equivalent to Trans Canada Highway Standards). The muskeg deposits shown in Fig.1 total 6 miles in length.

FIELD INVESTIGATION

Time did not permit extensive soil survey work or sophisticated laboratory testing of samples. Field work consisted of probes and hand power auger holes with field vane shear tests in the swamps and diamond drill, auger and Becker holes in some of the proposed cuts.

SOILS DESIGN

The original highway design comprised stripping and wasting the peat down to hard bottom. In many cases, the limited soil survey showed soft clays below the peat. Excavation and backfilling would have cost $450,000 for the 540,000 cubic yards of peat involved. Surcharging plus an allowance for settlement totalled $200,000. As a general guide therefore it was decided to excavate the peat only where it was less than 4 feet thick. This was not always done. Instrumentation and surcharging was proposed on swamps thicker than 4 feet.

Figure 2 shows the distribution of peat thicknesses (5 to 8 feet) and fill heights (4 to 10 feet) above original ground. The numbers ranging 2 to 8 are the applied surcharges.

Figure 3 is similar to Figure 2, but shows the swamp lengths, commonly 600 to 1000 feet with a maximum of 1.7 miles.

Fill stability was checked using the field vane shear data. Grade reductions were accommodated where necessary.

An estimate of fill settlement over the peat areas was necessary for the Location Designers to produce a balanced grade. Since no Shelby sampling and consolidation testing had been done, the settlement allowance had to be based on experience at other locations. A chart, Fig.4, was prepared based on limited field experience on peats 20 feet thick.

The Designer knowing the safe fill height (H_f) above original ground, the original peat thickness (H_p) enters the ratio H_f/H_p and obtains the settlement(S) as a fraction of the original peat thickness.

RESULTS

Only on some 6000 feet total of highway has settlement sufficiently progressed for the surcharge to be removed. Using the same parameters H_f, H_p and S, the results from settlement plates in these various sections plot as shown in Fig.5. The wide scattering reduces when the data is separated in terms of range of peat thickness Fig.6. "Average" curves drawn through the points are shown in Figure 7. Evidently therefore the original chart overestimates the settlement on the North Thompson peats for depths less than 20 feet, e.g. a 6 ft. fill on 12 ft. of peat gives H_f/H_p of 0.5 and 4.8 ft. of settlement as against $12 \times 0.13 = 1.6$ ft. which the new curve would predict. The fill has therefore been constructed wider than necessary to achieve 44 ft. top after settlement has terminated. It is proposed to continue with the study to see if refinement to the curves can be made.

PERIOD OF FILL CONSTRUCTION AND SURCHARGING

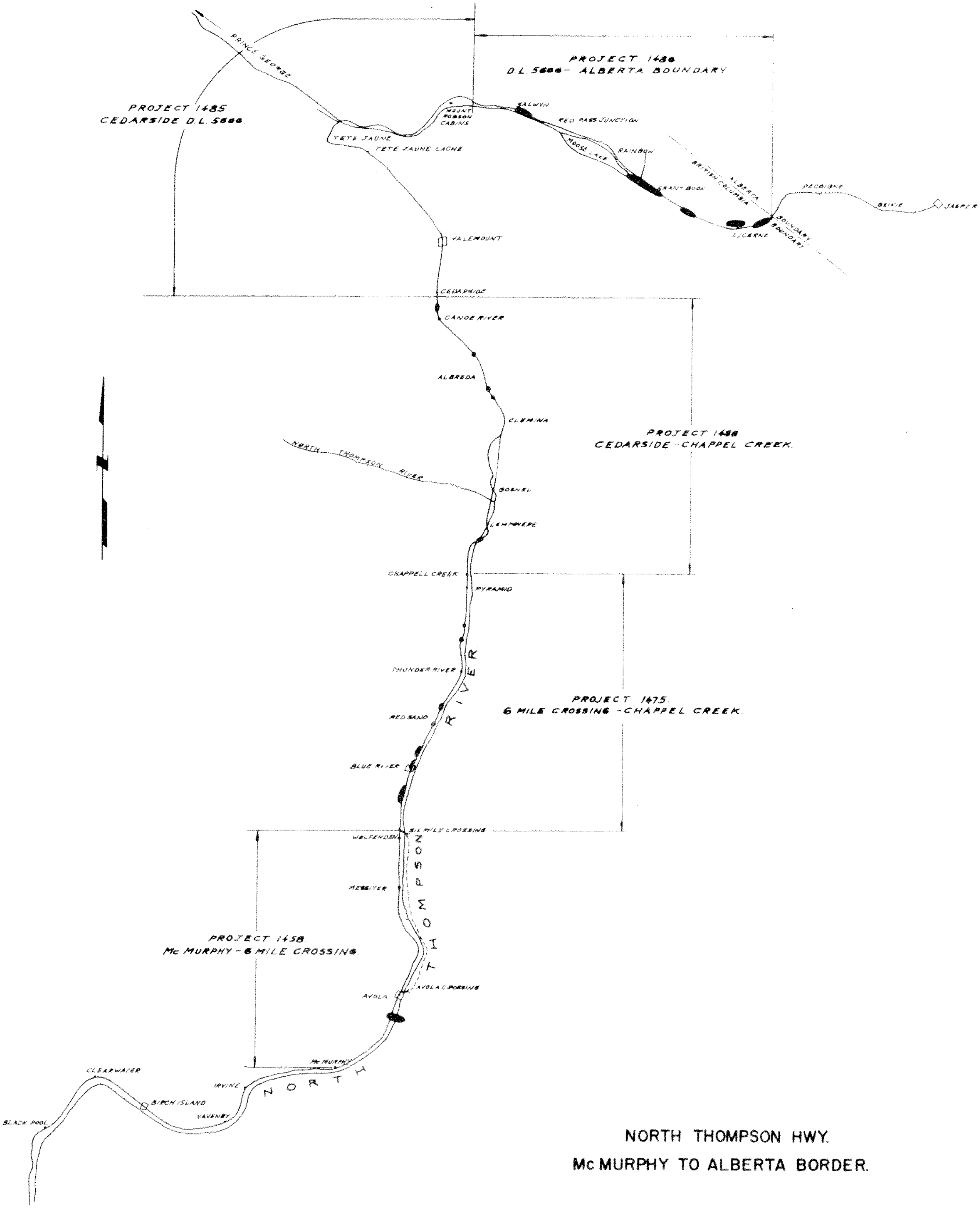
Native granular fill was placed as a 4 to 5 ft. thick working platform, instrumentation installed and fill construction in 3 to 5 ft. lifts begun with 10 to 15 days generally between lifts. Construction to surcharge height was finished within 90 days. Figure 8 shows typical relationship obtained for settlement-log time.

The height of surcharge to be placed was estimated from "rule of thumb" half (settlement plus fill height) and one of two criteria applied as to the length of time it should stay on; first the "leave as long as possible" criteria; second, the "25 year settlement - load line - time" approach shown in Fig.9. The big question with the latter criteria is the validity of the straight line extension of the settlement-log time relationship to 25 years.

CONCLUSIONS

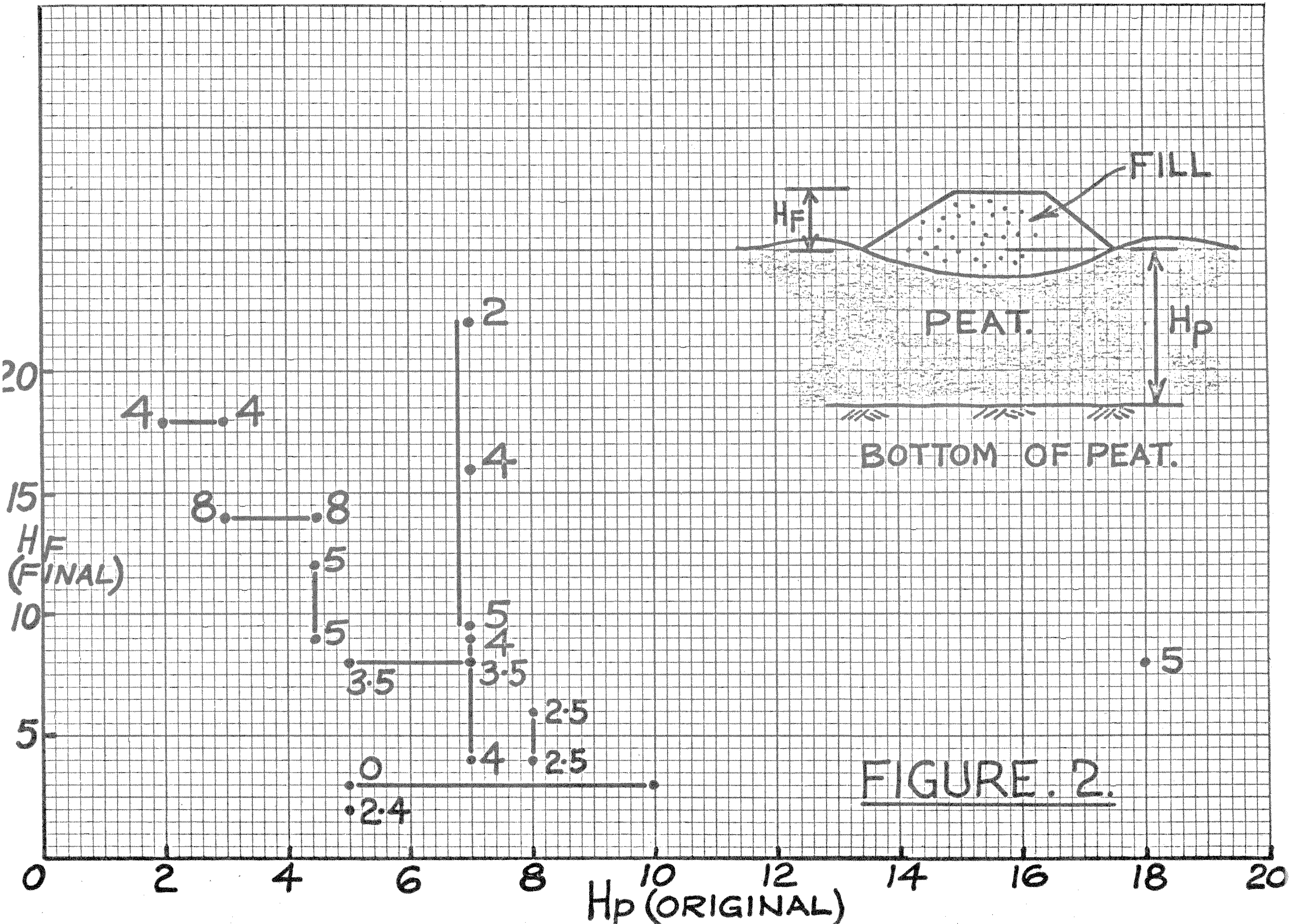
The interim chart used in the settlement design of the North Thompson Highway peat areas, proved to overestimate the settlement. Actual results have been used to improve the relationships. This is still only a rough guide, but when considered in relation to the heterogeneity of peat measured by the notorious variation in properties such as moisture content, density, specific gravity, permeability, etc., it is probably no worse than the results of conventional analysis based on consolidation tests. Even here, factors such as the mat strength and the degree of heterogeneity cannot be allowed for quantitatively.

The study will be continued to see if further relations or refinements evolve.



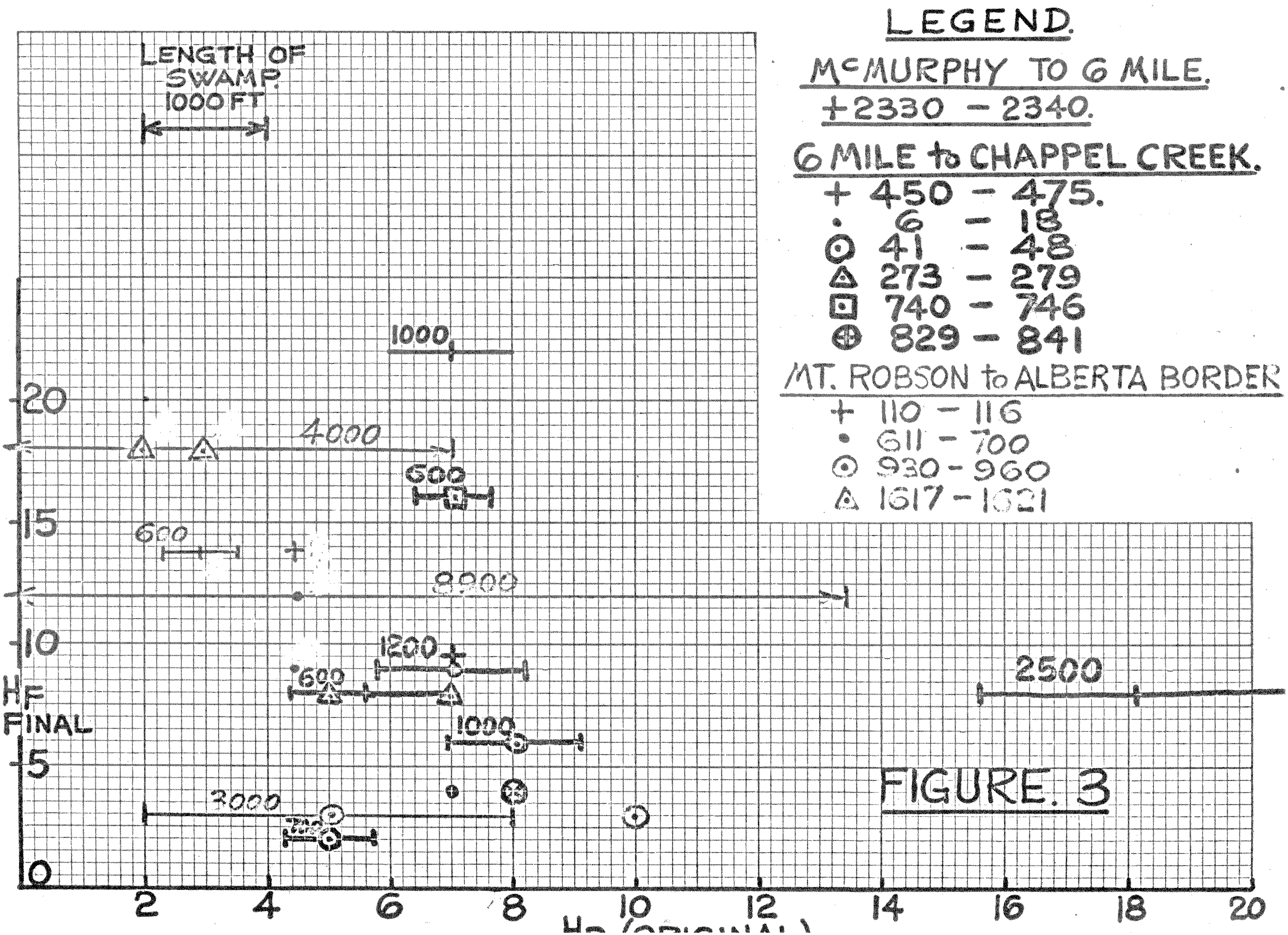
NORTH THOMPSON HWY.
Mc MURPHY TO ALBERTA BORDER.

FIG. I



FILL
HF
HP
PEAT.
BOTTOM OF PEAT.
HF (FINAL)
HP (ORIGINAL)
0
2
4
6
8
10
12
14
16
18
20
5
10
15
20
4
4
8
8
5
5
2
4
5
4
3·5
3·5
2·5
2·5
4
0
2·4
5

FIGURE. 2.



LEGEND.
McMURPHY TO 6 MILE.
+ 2330 - 2340.
6 MILE to CHAPPEL CREEK.
+ 450 - 475.
• 6 - 18
⊙ 41 - 48
Δ 273 - 279
▣ 740 - 746
⊕ 829 - 841
MT. ROBSON to ALBERTA BORDER
+ 110 - 116
• 611 - 700
⊙ 930 - 960
Δ 1617 - 1621
LENGTH OF SWAMP. 1000 FT
1000
4000
600
600
8900
1200
600
1000
2500
2000
HF FINAL
0
5
10
15
20
2
4
6
8
10
12
14
16
18
20

FIGURE. 3

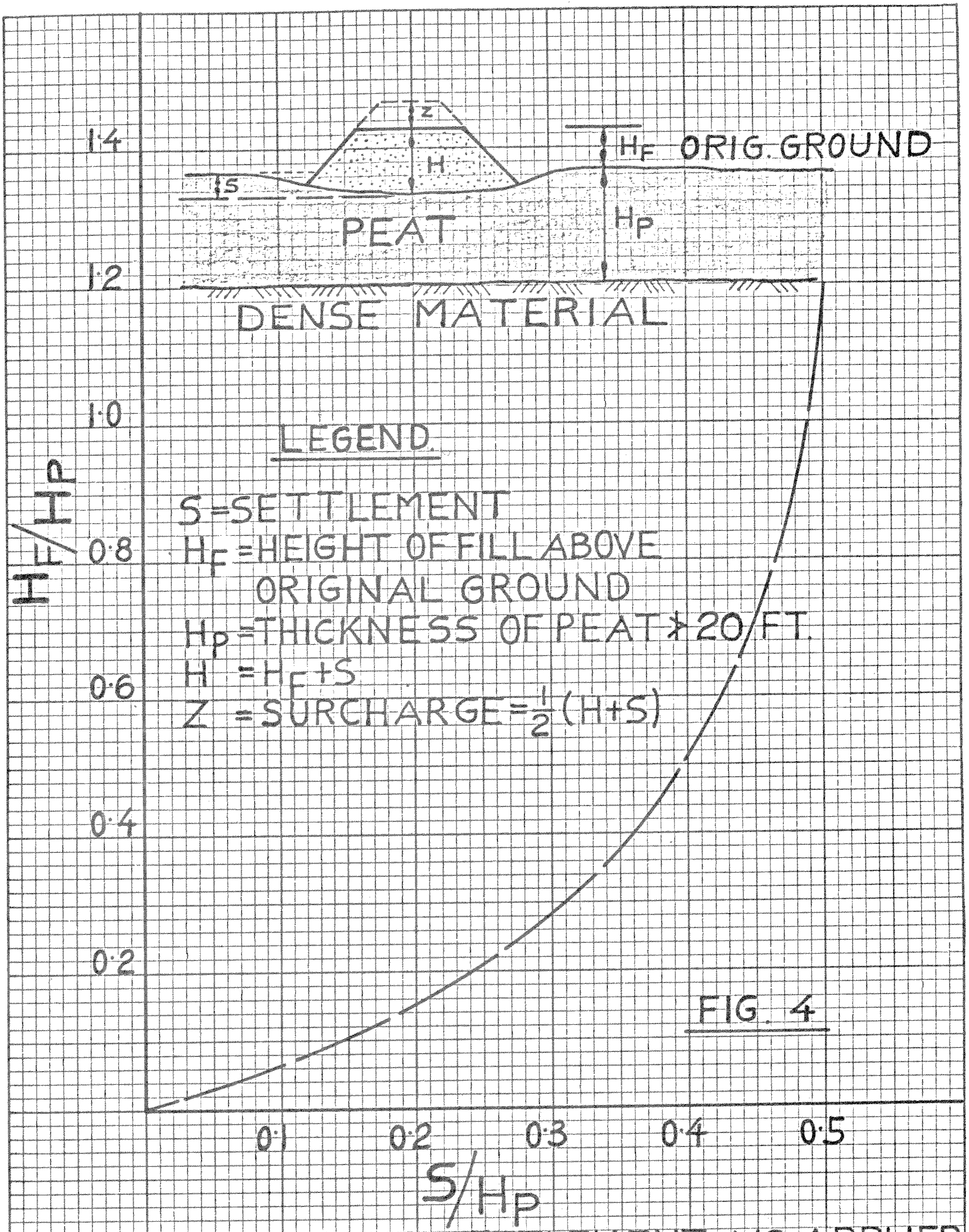


FIG. 4

RELATIONSHIP OF SETTLEMENT VS. APPLIED FILL FOR SOME B.C. CASE HISTORIES OF FILL CONSTRUCTION OVER PEAT.

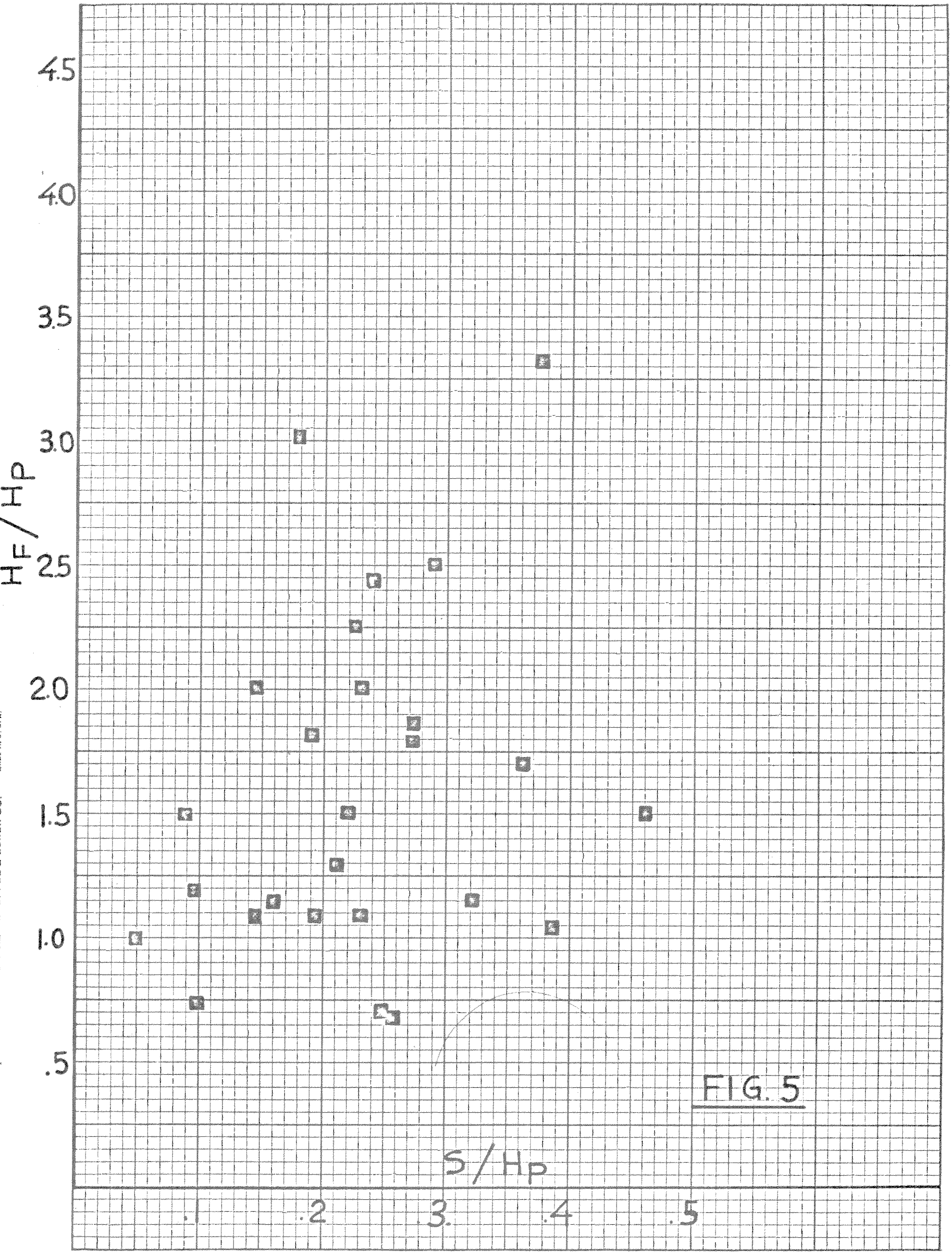
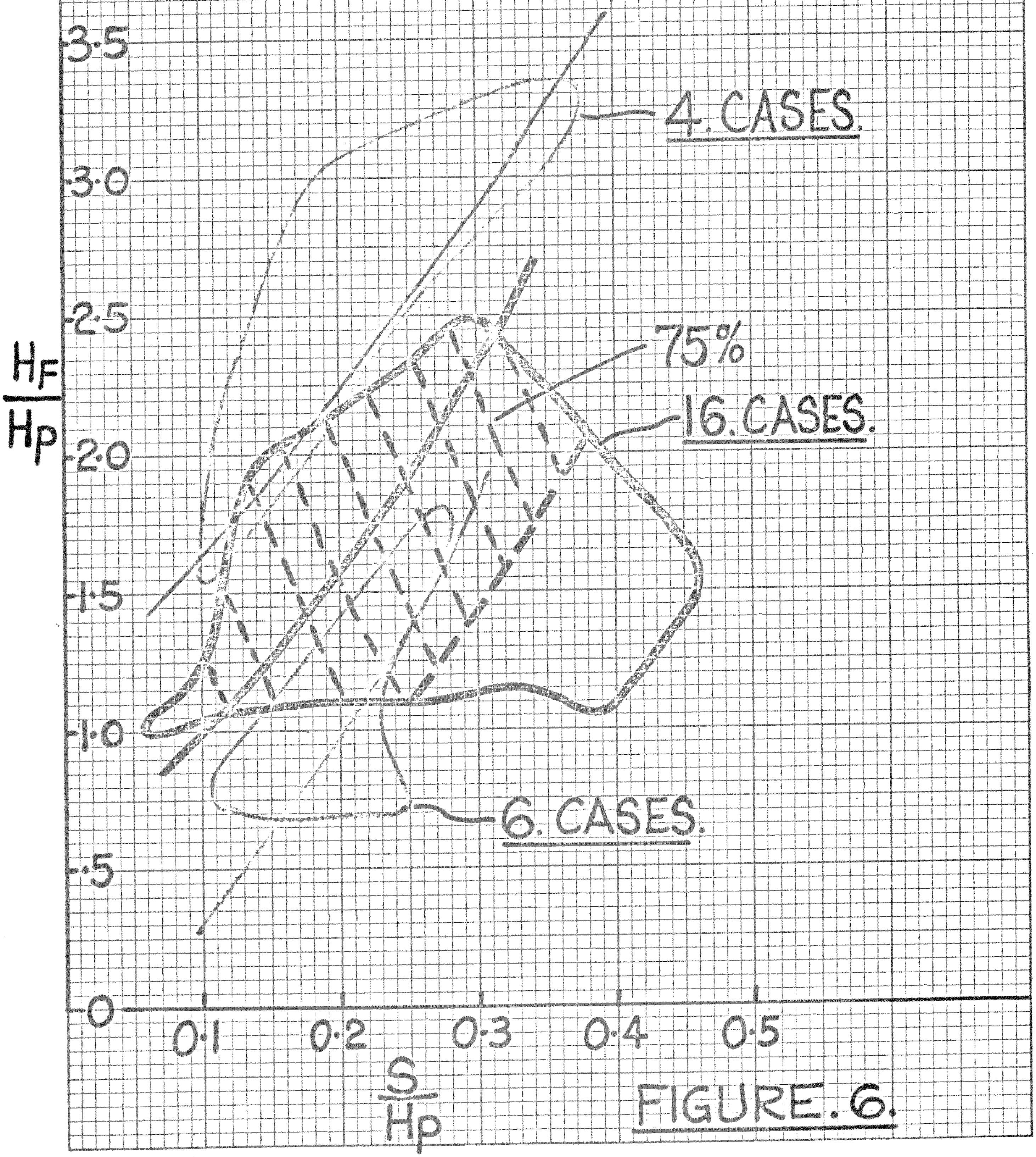
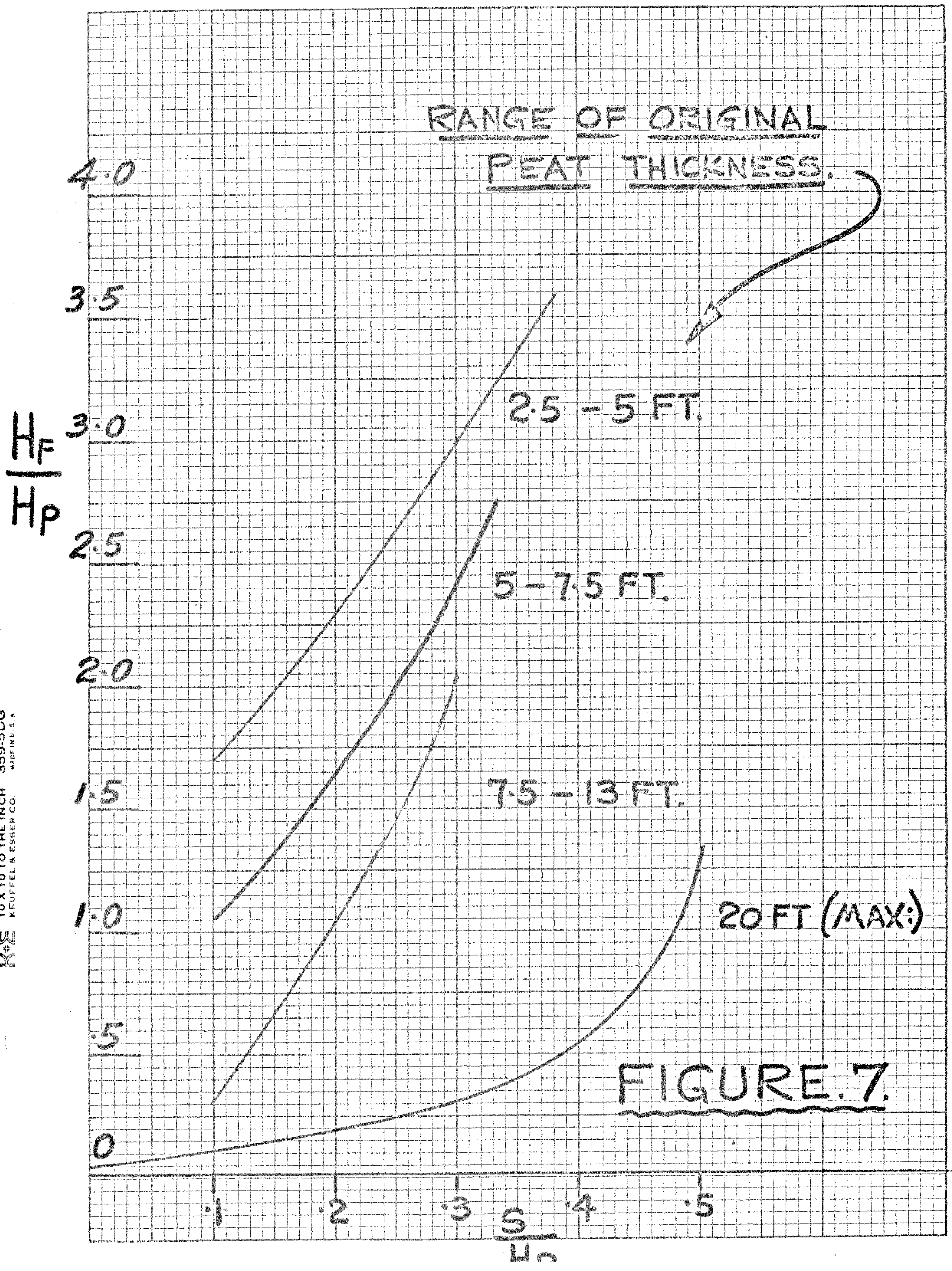


FIG. 5



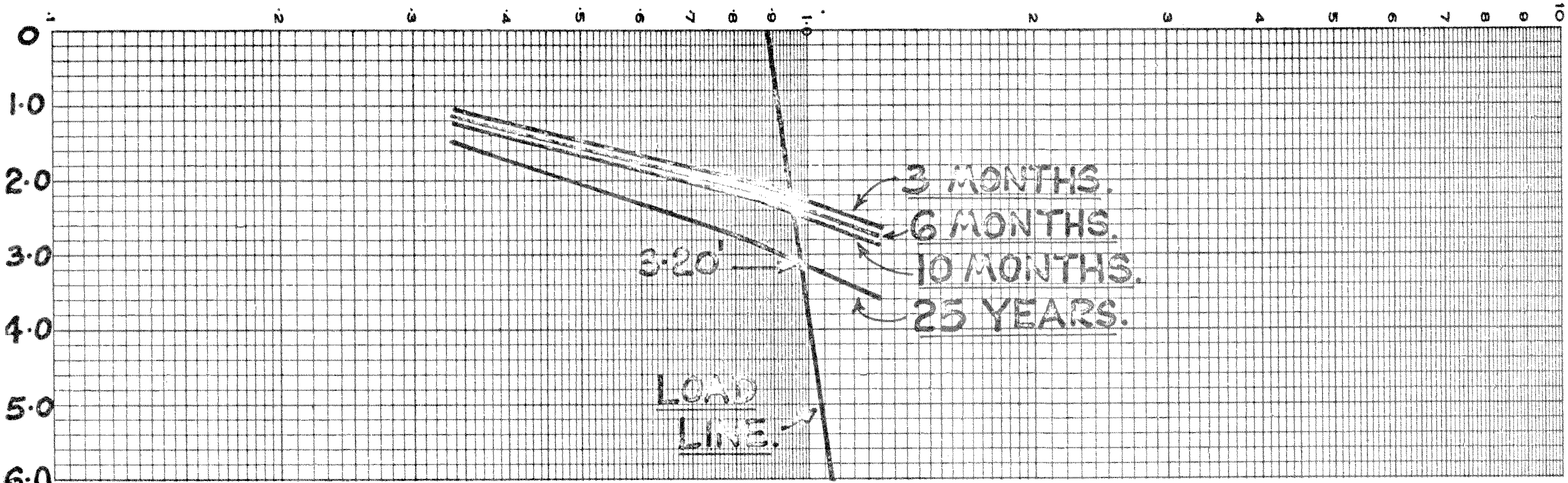
3.5
3.0
2.5
2.0
1.5
1.0
.5
0
$\frac{H_F}{H_P}$
0.1
0.2
0.3
0.4
0.5
$\frac{S}{H_P}$
4. CASES.
75%
16. CASES.
6. CASES.

FIGURE. 6.



RANGE OF ORIGINAL
PEAT THICKNESS.
2.5 - 5 FT.
5 - 7.5 FT.
7.5 - 13 FT.
20 FT (MAX:)
$\frac{H_F}{H_P}$
4.0
3.5
3.0
2.5
2.0
1.5
1.0
.5
0
.1
.2
.3
.4
.5
$\frac{S}{H_P}$

FIGURE. 7.



FILL 14·8' = 0·89 TSF.
INSTALLATION #2. STA: 454+50.

LOAD TSF.	TOTAL SETTLEMENT. 3 MO.	6 MO.	10 MO.	25 YR.
0·34.	1·10	1·15	1·18	1·50
0·84.	2·10	2·15	2·25	2·84
1·25	2·70	2·80	2·90	3·62

FIGURE. 9.

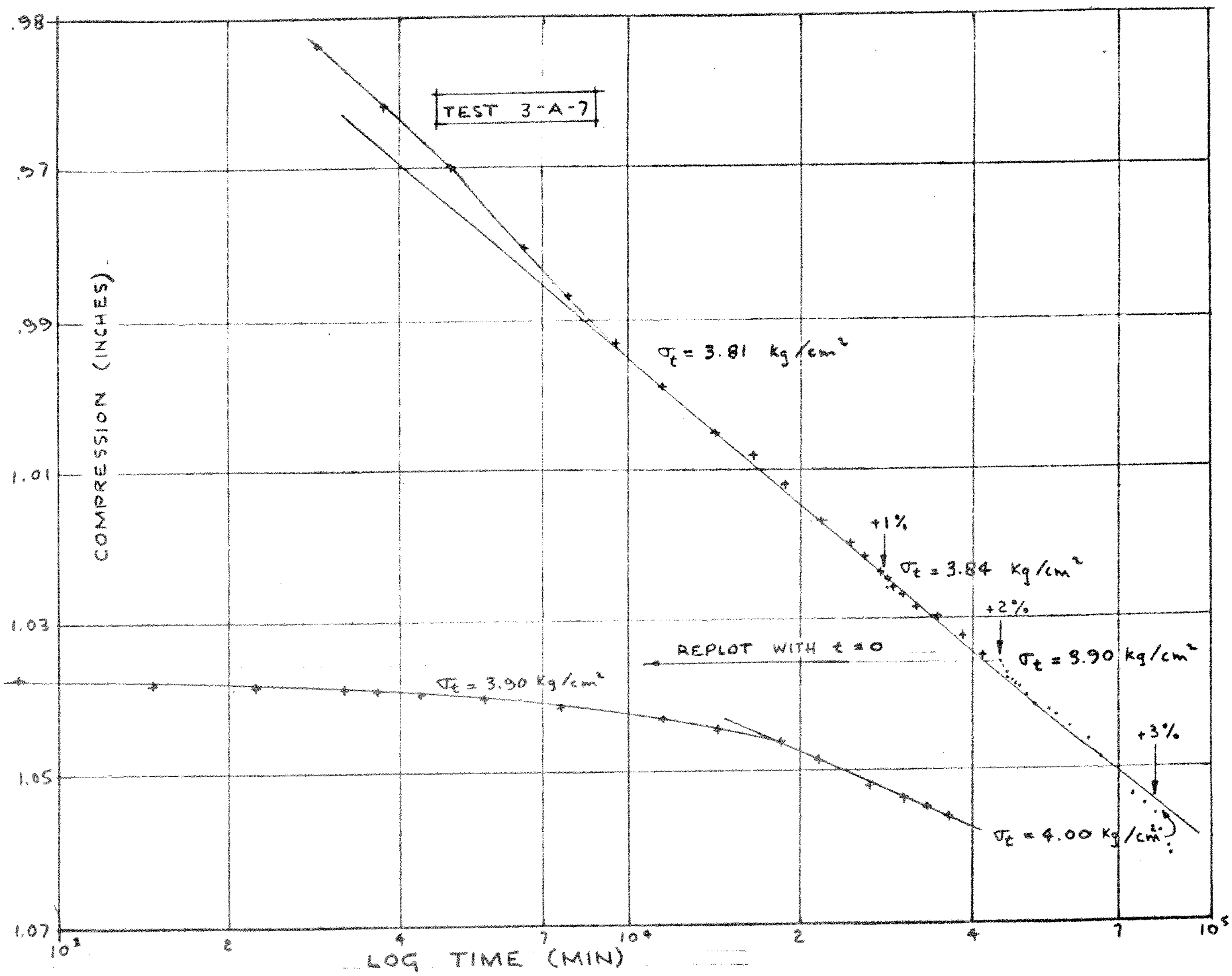


Figure 3 Compression vs log time, test 3-A-7