Proceedings of the Seventh Muskeg Research Conference

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NATIONAL RESEARCH COUNCIL OF CANADA

ASSOCIATE COMMITTEE ON SOIL AND SNOW MECHANICS

PROCEEDINGS

OF THE

SEVENTH MUSKEG RESEARCH CONFERENCE

APRIL 18 AND 19 1961

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and
Miss J. Butler

TECHNICAL MEMORANDUM NO. 71

OTTAWA

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FOREWORD

This is a record of the Seventh Annual Muskeg Research Conference which was held in the Engineering Building of McMaster University, Hamilton, Ontario, on 18 and 19 April 1961. The Conference was sponsored by the Associate Committee on Soil and Snow Mechanics of the National Research Council. A list of those in attendance is included as Appendix "A" of these proceedings.

The overall theme of the Conference was muskeg in relation to development of the North. A wide variety of topics was considered, including the environment and distribution of muskeg, road and railway construction in muskeg, petroleum exploration in organic terrain, reclamation of muskeg, corrosive effect of muskeg waters on concrete, and the compression characteristics of peat. In Session I, under the chairmanship of Mr. R.A. Hemstock, five papers were presented. Session II was chaired by Mr. C.O. Brawner and four papers were presented. The chairman of Session III was Mr. T.A. Harwood; five papers were presented. Session IV was a field trip to the nearby Copetown Bog. A dinner meeting was held on 18 April, at which the speaker was Mr. R.F. Legget, Director of the Division of Building Research, National Research Council.

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INTRODUCTORY REMARKS

Dean J. W. Hodgins of the Faculty of Engineering cordially welcomed delegates to Hamilton, to McMaster University, and to the Engineering Building. He expressed the hope that the Conference would be a successful one and that the visit to McMaster would be pleasant. Dr. Radforth, on behalf of the Associate Committee on Soil and Snow Mechanics, welcomed those present and expressed pleasure that Mr. R. F. Legget, Chairman of the Associate Committee, would be in attendance during most of the Conference. He drew attention to the photographic display which was set up adjacent to the cafeteria in the Students' Union Building and hoped that delegates would avail themselves of the opportunity to view these displays. Announcements were made pertinent to luncheon arrangements, dinner meeting, etc.

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I. 1 MUSKEG: ITS ENVIRONMENT AND USES

J. Terasmae*

"Muskeg" is a genuinely Canadian word with a reasonably long tradition. It has weathered many a criticism by specialists who have attempted to define and to use the word "muskeg" as a precise reference term. One such attempt was made by Radforth (13) in the first major report on muskeg research sponsored by the National Research Council. He came to the conclusion that "the results of the inquiry have been enlightening but bewildering." The writer is of the opinion that "muskeg" was never meant to be a precise scientific term, but rather became an everyday word out of necessity and tradition, first among Canadian Indians and then among the traders and explorers. "Muskeg" is as versatile a term as it is difficult to define. Perhaps it would be safest to define "muskeg" as a specific type of environment, caused and affected by a number of climatic, edaphic and biological factors.

Muskeg has always been associated with the Canadian North and for anybody who has travelled in that part of Canada it brings back memories of the wide expanse of bogs, lakes, rivers and spruce forest brcken by rock outcrop - as well as of blackflies, mosquitoes and wet feet! Any development in the Canadian North is likely to encounter problems related to the muskeg. Thus, it is important to know how to live with it, since there is no reasonable way of avoiding it.

One important aspect of muskeg, involving engineering factors such as access, trafficability and physical properties, will be the subject of other papers at this conference and is not discussed further here. There are, however, other aspects of muskeg research which are of more academic interest, perhaps, but nevertheless are important in understanding the context of muskeg.

One fundamental aspect is the development of muskeg. Obviously an excess of moisture is required. This condition is satisfied in areas of adequate precipitation, low evaporation, impervious soils or rock (or permanently frozen ground) and poor surface drainage

*See Appendix "A" for affiliation

owing to insufficient slopes or unorganized drainage systems. The temperature regime is also important; high temperature favours plant growth but increases the rate of decomposition. It seems that muskeg develops best in a boreal climate.

Under favourable conditions, the production of plant matter is higher than the decomposition, with a resulting accumulation of organic deposits beneath the growing plant cover. Hence, muskeg is composed of a living and a dead layer of plants. Circulation of water and air in the muskeg environment is largely inhibited and this generally brings about acid conditions which further favour preservation of dead plant matter. Both the living plant layer and the dead vegetable matter are of many kinds and have stimulated the construction of several classification systems for these layers. Such a classification of muskeg for engineering purposes has been worked out in Canada by Radforth (13). The living layer has been studied extensively by botanists and comprehensive classifications have been proposed in many parts of the world. The dead layer (peat and muck) has also been the subject of extensive investigations and several classifications are available (12).

By its biological nature muskeg is an extremely complex environment and may seem rather unorganized to the untrained eye. However, a closer examination will readily reveal orderly organization patterns in the muskeg environment.

The abundance of muskeg in some regions of the world has perhaps naturally directed man's curiosity towards its possible uses, particularly to those of the accumulation of peat. Numerous investigations have been made in this field (Canadian Department of Mines (4); Girard (6); Haanel (7); Leverin (9); Odell and Hood (11); Osvald (12); Sheridan and DeCarlo (17); Wilson and Staker (20); Wilson et al. (21). The use of peat as fuel and also as a soil conditioner is well known. Numerous other less important uses have been described. More recently, chemical studies of peat have shown some interesting results (15)(16). By various treatments, resins, waxes, carbohydrates, acids, lignin, adhesives, detergents, carbon, alcohol, etc., can be obtained from peat.

In suitable locations, peat and muck areas have been used with success for growing a variety of vegetables. It may be interesting to note that such "waste-land" has been bought for $10-15 per acre and after proper drainage and cultivation has been sold for up to $1,000 per acre. Risi et al. (16) estimate that there are some 700,000,000 acres of probably arable peatland in Quebec as well as some 200,000,000 tons of peat which can be utilized. Thus it is suggested that muskeg should be considered as a resource rather than as waste-land. With a better knowledge of the various aspects of muskeg, such areas can be definitely of economic value and assist further development instead of restricting it.

The accumulation of organic matter in the muskeg environment opens up an entirely new and different use for it. Each year a small amount is added to the accumulation of the deposit. Into its preserving environment many plants and parts of plants and animals are buried each season and become fossils. In this manner a continuous record of plant and animal life is preserved in the accumulating peat deposits. Besides the larger fossils representing plants which grew at or near the site of accumulation, the deposit also contains small microscopic fossils which could have been carried some distance by wind, water and insects before burial and hence represent the more regional plant cover. Pollen and spores belong in this category.

Each year large amounts of pollen are produced by plants during flowering, later to be blown about by wind or carried by insects from one plant to another. This rather wasteful way of pollen transport allows most of it to become deposited on land, lakes and muskeg where it can be preserved. When a peat deposit has been accumulating for several thousands of years, it contains a long record of past plant and animal life. A peat deposit can be compared to a reference library which may contain books in many languages and on many subjects. As in a library where the desired information must first be located then read and interpreted, so in a peat deposit the fossils must first be found then identified and interpreted. This may literally require a knowledge of many languages and fields of science. One such language is called palynology, which is a study of pollen and spores. It has been found that each plant species has its characteristic pollen. A study of fossil pollen can shed light on the plants of the past which produced it, and the past types of forest can actually be reconstructed.

The first step in such a palynological study is to locate a suitable deposit and then obtain a core through the sediments extending from surface to bottom where a non-fossiliferous floor is encountered. The location of suitable coring sites is by no means easy. Aerial photographs and surface reconnaissance are useful and general experience gives some guidance, but without numerous test borings across the bog there is little assurance that a suitable site has been selected. The present surface plant cover can be misleading as an indicator of the depth of bog and of the sedimentary sequence. In a study of more than a hundred separate peat deposits across Canada, the writer has observed stratification in all deposits. Several types of peat and lake beds generally compose the accumulation of organic deposits (2).

When a suitable site for coring is located, the core is taken with the appropriate sampler. A detailed description of the sedimentary sequence is made and samples collected at close intervals (1-3 inches) from the core. Since many thousands of microfossils are usually present in each cubic centimetre of peat, only small samples are necessary. In the case of an open exposure, a search is made for recognizable fossils and the vertical as well as horizontal changes in the deposit are examined and noted.

The collected samples are then subjected to necessary chemical treatments at the laboratory to separate and concentrate the microfossils for a microscopic study (Brown (3); Erdtman (5); Jones (8)). Under the microscope, some 200-300 or more microfossils are identified and counted by traversing the prepared slides. Each pollen or spore type is expressed in per cent of the total number counted or of the total number of tree pollen. This allows comparison of assemblages from different depths and a construction of pollen graphs for the identified species, which compose a pollen diagram (18). In brief, a pollen diagram illustrates the changes in relative abundance of pollen and spores through a certain episode of time. The assumption is made that these changes reflect corresponding changes in the plant associations. Thus the history, migrations and development of plant cover in a given area can be studied by palynological investigations, and changes in environmental conditions can be inferred from such studies. Past regional climatic changes can also be traced by palynological studies. This introduces another important use of muskeg. A knowledge of past environmental changes provides the possibility to predict what is happening and will happen in the biological and geological environment around us.

It has been established that the climatic changes in the last several thousands of years have been contemporaneous regionally and probably on a world-wide basis. Palynological studies, using that thesis, can be utilized for correlation and approximate dating of certain events. The accuracy of such dating has been much improved and put on a sound basis by the development of the radio-carbon dating method. This method is based on the principle that plants incorporate carbon dioxide from the atmosphere, where it contains in equilibrium a certain amount of the radioactive carbon isotope C^{14}. When a plant dies, the radioactive carbon in it begins to decay at a known rate and by measuring the activity in dead plant matter its age can be determined.

To illustrate the methods outlined above, some examples will be given. Palynological studies made in southern Ontario have shown that some 10,000-12,000 years ago a boreal forest grew in this region and was gradually replaced by a mixed hardwood-coniferous forest with abundant pine some 7,000-8,000 years ago. This forest type changed into a predominantly hardwood forest some 6,000-7,000 years ago and smaller changes have occurred since. These changes definitely indicate climatic changes in the past and the corresponding changes in the general environment. Similar studies made in northern Ontario have shown that when the glacier ice melted from the region both climatic and drainage conditions then were better than at the present time. The palynological studies, coupled with radiocarbon dating, have indicated that climate generally was warmer some 5,000-7,000 years ago and that several plant species then ranged north of their present distribution. This post-glacial warmer period was followed by a cooler and wetter episode which definitely favoured muskeg development on a large scale.

Going farther back in time, buried peat deposits show that muskeg formation took place in the interglacial intervals some 100,000 years ago under comparable climatic conditions to the present. A study of these buried organic deposits has helped to reconstruct the interglacial environments and the development of plant and animal associations and populations and the climatic changes. One such important occurrence of interglacial beds is at Toronto and the writer made a study of these deposits (19) which are being exposed further in the excavations for the Toronto Rapid Transit Subway extension.

The study of buried and surficial organic deposits provides a tool for the geologist which has been successfully used for correlating geological events and stratigraphic units. It further provides a means for approximately dating such events which are beyond the range of the radiocarbon dating method.

In conclusion, it should be emphasized that muskeg research can have many aspects, both practical and academic. The latter aspect provides a potential source of information for study and reconstruction of past environments and this alone may justify the recognition of muskeg among natural resources rather than simply classifying it as waste-land. Improved knowledge of the muskeg environment through further research will assist in coping with it successfully in areas of exploration in the Canadian North.

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I. 2. DISTRIBUTION OF ORGANIC TERRAIN IN NORTHERN CANADA

N. W. Radforth

(Summary)

A consideration of distribution of organic terrain must involve a recognition of differences in environment. Such differences must be recognized in problems of access and especially in vehicle development. Therefore, in muskeg research, pure scientists are involved as well as engineers. Engineers, who are primarily interested in application of principles, need to recognize the fundamental contributions of pure scientists.

The distribution map* is new and therefore has "bugs" in it which eventually will be ironed out. Actually, this map represents areas in Canada where muskeg problems will occur; within those regions shown on the map, muskeg will be a problem and cannot be avoided. All muskeg areas in Canada (e. g. the local "Copetown Bog") are not noted, for this is not the purpose of the map.

Muskeg classification types most predominant in various areas are shown. The dotted area represents that region where the frequency of occurrence of muskeg is least and consequently the problems are possibly the least. The cross hatched area represents

*The Muskeg Distribution Map is published by the Defence Research Board, Ottawa, and is available upon request.

that region where the frequency of occurrence of muskeg is highest. It is almost impossible to achieve access to the north without encountering this area. The black lines designate particular areas in which certain muskeg types occur predominantly. Several circles are shown on the map - with St. John's, Fort Churchill, Kapuskasing, The Pas, etc., at the centre. These circles centralize, in terms of Canadian proportions, different kinds of environment within the distribution pattern.

It should be pointed out that muskeg changes not so much in structure as it does with regard to water relationships; it changes also with reference to the ice factor and with topographic conditions. In continuous muskegs, there is no major change in the structure of peat with depth. However, this is not always so in bogs (i.e. confined muskegs).

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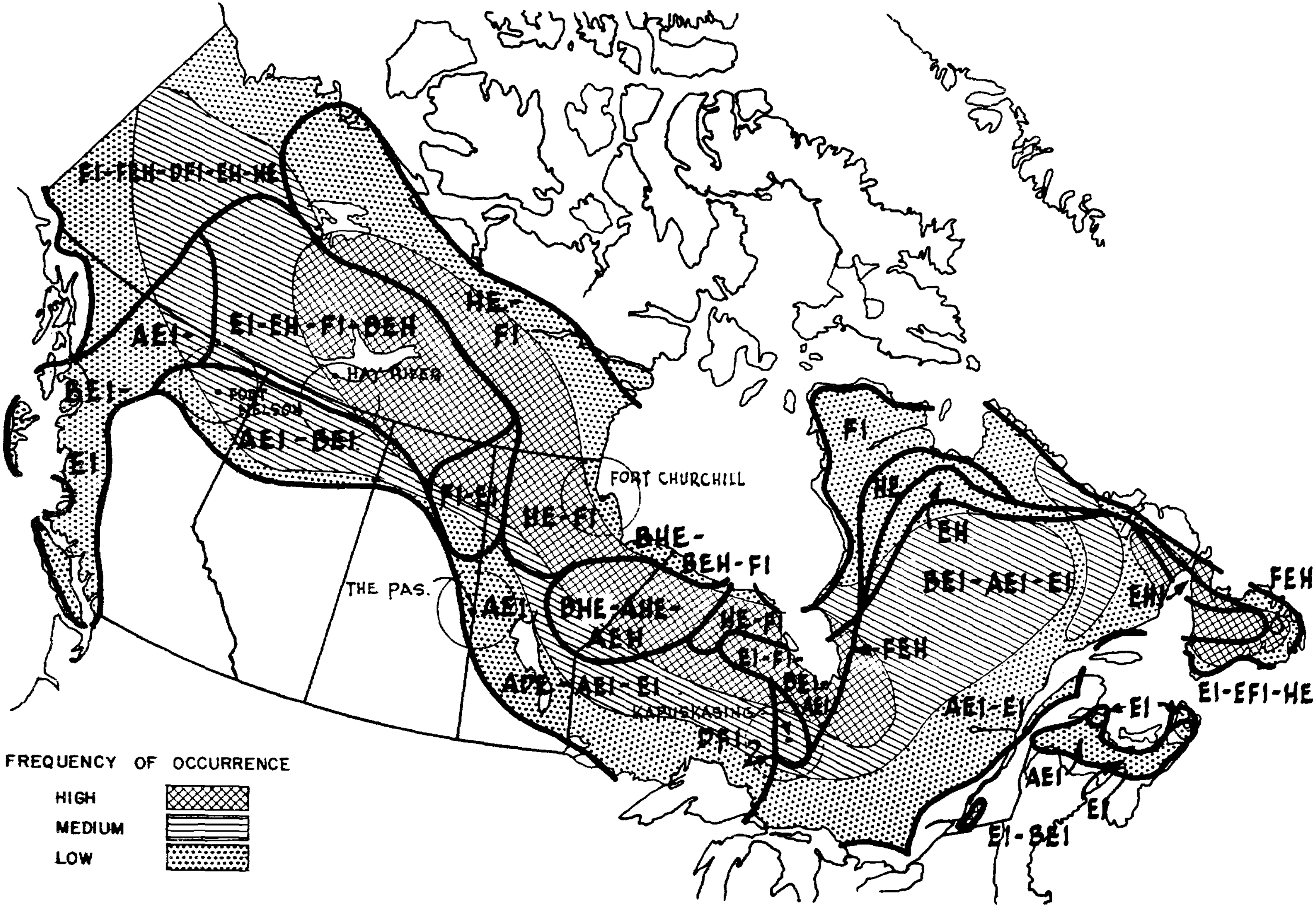

MUSKEG AREAS

BR 2515-3

ORIGINAL

Discussion

Mr. Hemstock (Imperial Oil) remarked that the three shadings represent different degrees, or percentages, of muskeg. He wondered if Dr. Radforth had given any numerical values to these areas, or are they purely relative. Dr. Radforth replied that an attempt is being made to give relative percentage figures. It is his intention that the map will appear again with the appropriate percentages noted on it. This will be in the third DRB Handbook.

Mr. Rula (Waterways Experiment Station) asked Dr. Radforth what type of sampler he used to obtain undisturbed samples and also to obtain depths of muskegs. Dr. Radforth said that it is very difficult to get adequate samples and also to know where to sample. The ice factor is particularly difficult; various devices have been used to get through organic terrain with ice in it. The thinking now is to obtain samples in winter by sawing them out. The problem then is to keep them frozen.

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I. 3 THE STRUCTURAL ASPECT OF PEAT

H. R. Eydt, J. Stewart and N. W. Radforth

(I) INTRODUCTION

N. W. Radforth

The accumulation of organic overburden on mineral terrain is largely affected by the persistence of exposed or high water table during periods when covering vegetation is actively growing. Thus water, not temperature, is the primary environmental factor basic to the generation of organic terrain. This is why one finds organic terrain on every continent and in all temperature zones, e. g., Tasmania, Uruguay, Nigeria, U. S. A., U. S. S. R., Britain, Scandinavia, British Guiana and Canada, etc., etc.

It is by reason of the water that peat - the dead component of organic terrain - is generated, fossilized and accumulated. If the water is removed the peat deteriorates, disintegrates and may eventually disappear

Because of the water factor (in its attendant physiographic circumstances) the associated vegetation is specially controlled and develops characteristic features which it readily reflects. These features, for both living and fossilized components of organic terrain, have been reported on elsewhere (2).

Often these features do not change significantly at given locations where peat is accumulating or, if they do, the change signifies a characteristic trend that can be demonstrated at other locations where conditions are similar. Relationship among the features is therefore ordered. This facilitates classification and prediction of organic terrain conditions, provided interpretative devices are appropriate and adequate.

Despite the prevailing cosmos in peat, details of its structural relationships are elusive. This is partly because the information on structure is needed on different bases for a wide variety of application; in agriculture, forestry, foundation and hydraulic engineering, aerial interpretation, vehicle development, botany, geography and military application.

Whatever method is used to designate peat structure, and for whatever purpose, to be appropriate it is desirable that it reflects the fundamentals of growth and organization in peat. To be adequate it should not only identify structural states; it should facilitate quantitative assessment of physical and mechanical conditions pertaining to those states.

The techniques which are employed in the examination of the natural organization of peat fall into two main groups. In the first place there are those methods that preserve the natural spatial arrangement of peat; and secondly, there are those that destroy the natural spatial distribution of the plant remains during analyses. To the first group belong such methods which reveal by bulk analyses the kinds of tissues present; while in the second group methods such as pollen analysis (palynology) and visual examination (stratigraphy) are used.

Two such methods of recent introduction are now described below.

(II) THE MACRO-CONSTRUCTION OF PEAT

H. R. Eydt

To reveal the macro-construction of peat it seemed desirable to examine the components of peat in their in situ arrangement because the mesh-work of the peat would be revealed and the tissues which form this would be shown. From an examination of the arrangement of the various tissues, it seemed that some of the principles governing the structure of the peat would be disclosed. If the arrangement of the tissues was seen at various levels, then a three dimensional view of the peat would lead to an assessment of its paleoecology. An appreciation of the tissues of the peat in their in situ arrangement which related to the field notes commenting on the water table of the peat, would allow some principles relating to the water relations of the peat.

Method

To obtain the in situ arrangement of the plant tissues in the peat, the method of embedding peat cubes in paraffin and then sectioning on a microtome proposed by Radforth and Eydt (4) was utilized.

Selection of Peat Samples

Peats from Ft. Churchill, Man. , were selected to represent a range of micro-environments from an area which is classified within one climatic ecological zone. These peat samples are referred to by the numbers which the cores were given when they were extracted.

Results

(I) Types of tissues identified

Examination of the slides of sectioned peat showed a variety of plant material which had to be classified into botanical entities. The tissues or associations of tissues identified were epidermis, parenchyma, xylem, moss leaf, and non-woody tissue which could not be placed in the above-mentioned groups.

(II) Analysis

When the mounts were completed and the tissues categorized, attention was turned to evaluation of in situ arrangement of fragments. To examine distribution of the fragments, sections were observed at a magnification of 320X. In the eyepiece of the microscope was a Whipple disc which divided the field of view into 100 equal squares. If 65 of the squares contained plant fragments, the density was said to be 65 per cent. Of these 65 squares, if 20 contained woody tissue, 20 parenchyma and 25 non-woody, these numbers were taken as the percentage cover for each of their corresponding botanical categories. Fifty areas on the slides were examined for plant tissues at each one inch interval. The density of each area in per cent, and the percentage of each tissue making up this density were recorded, and their averages taken as the density index and tissue indices at each depth.

(III) Evidence relating to paleoecological sequence

It was possible to obtain data on the density and the distribution of the plant tissues at selected depths in all the peats examined. The results of these analyses are shown on the following graphs (Figs. 1-8).

Discussion

Paleoecological Sequence

It is now possible to appreciate a three-dimensional picture of the peat and, by using the density readings and the tissue indices, to interpret the paleoecology of the various peats. Because the embedding of the peat has kept the in situ arrangement of the peat components, the indices reveal the structure of the peat and assist in determining the principles which control peat accumulation.

Peat samples B. 469 and B. 462 were both selected because they did not show structural change with depth and did represent two quite different micro-climates. Although no ecological sequence was expected from these samples, their indices when interpreted with the field notes will give two standards of reference for the other peats which are being examined.

In sample B. 469 (Fig. 3) it is seen that epidermis is dominant, with very low woody tissue. This peat was removed from a sedge meadow where the water filled the hole when the sample was removed. From the large amount of epidermis, and the presence of parenchyma and non-woody tissue, and the small amount of woody tissue, it is accepted that this peat has resulted from the accumulation of sedge plants, which have these tissues. The very small amount of woody tissue probably represents the vascular tissue in the sedge. Because the amount of non-woody tissue rose rather slowly, and never became the total tissue type seen, it is assumed that very little structural changes occurred as the peat accumulated.

The indices for the plant tissues seen from peat sample B. 462 (Fig. 1) are high in non-woody and woody tissue. The amounts of epidermis and parenchyma are low. This is a direct contrast to B. 469. Because the peat showed no change in type with depth, it is assumed that the vegetal cover has not changed significantly since peat started to accumulate.

The tissues from B. 464 (Fig. 2) show a high amount of non-woody tissue and some woody for the first three inches of depth associated with some epidermis and parenchyma. Below the three inch level, the woody tissue is not present, and a slight rise in parenchyma and a considerable rise in epidermis is seen. The upper three inches resemble the heath peat of B. 462 (Fig. 1) and the lower peat resembles the sedge peat of B. 469 (Fig. 3). This strongly indicates a difference in peat type with depth.

The moss tissue composed of Sphagnum dominates the upper six inches of B. 470 (Fig. 4). Only a trace of parenchyma, and some non-woody tissue are associated with the Sphagnum, but at the seven inch level, the non-woody tissue shows an increase and woody tissue appears. As depth increases, the woody tissue disappears and non-woody tissue becomes dominant. The large amount of Sphagnum over woody and non-woody tissues indicates that ecological succession occurred during the history of the peat.

<u>Water Relations in Peat</u>

Peat sample B. 469 was removed from a meadow predominantly of sedge where the water table was high. After the sample was removed, the hole filled rapidly with water. The density of plant tissues for this peat, as seen on Fig. 7, is never very high - only up to 44%. This means that there was more water than plant material in this peat. The tissues showed a high degree of preservation, which indicated that aerobic decay by micro-organisms had not taken place to any extent.

The density indices for B. 462 (Fig. 5) show higher readings than seen in B. 469. These higher indices are associated with a high amount of non-woody tissue, which is the result of tissue alteration. The poor degree of tissue preservation is due to either aerobic decay or to compaction due to the pressure of accumulated peat. This sample is not as deep. however, as B. 469; thus it seems that the poor tissue preservation and the high density indices are due to aerobic decay. Air can only enter the peat if the water level is low, at least part of the year.

Taking these two peat samples as standards, it seems that where the density readings are low, the water level is high, and where the density readings are high, the water level is low.

The density readings fluctuate for B. 464 (Fig. 6) but they always stay above 40%, and most often above 50% which indicates a low water level.

The density indices for B. 470 (Fig. 8) fluctuate between 35 and 53%, with most readings in the 40's. This suggests a high water table, with occasional fluctuations in the peat.

Conclusions

(1) There was no indication that succession took place during the history of certain peats (B. 469 and B. 462). This was revealed by analysis of the tissue indices.

(2) Most peats showed definite tissue difference so that it is stated that plant change occurred.

(3) Where the surface layer of the peat is not sedge peat there is most probably a high density in the peat and this density is made up of non-woody tissue and some woody tissue. However, if the surface vegetal cover is sedge, there is most probably a low density in the peat, and this is made up of epidermis, parenchyma, and only very little woody and non-woody tissue.

(4) Where the tissue is well preserved, the water table is high, and where the tissue is not well preserved, and appears as non-woody tissue, the water table is then assumed to fluctuate.

(III) CUTICULAR ANALYSIS: A NEW APPROACH TO THE ELUCIDATION OF THE STRUCTURE OF PEAT

J. Stewart

It has been demonstrated previously (5) that cuticle is present in peat and that it possesses sufficient taxonomic characteristics to ensure, at least generic and sometimes specific identification.

The objectives of this section of the paper center upon the following questions: (1) In what manner will cuticle lend itself to quantitative expression? (2) By what means do stratigraphic, palynological and cuticular analyses reflect structural changes in peat?

Cuticular Analysis of Peat Core

The first question investigated was whether cuticular analysis could be expressed in quantitative terms and thereby lend itself to elucidation of the structural characteristics of peat. For this purpose a peat core - 7.1 meters in depth - was procured by means of a Hiller borer from the X_1-X_2 transect of the Copetown Bog. (Fig. 9). This core was located near the center of the Bog which contained the greatest depth of peat and was overlain by the predominant group of vegetation - BEI (2). (Fig. 9). The overall depth of the peat core, from surface to the mineral sub-layer, was 7.45 meters, the upper 0.35 meters of which could not be collected because of the aqueous nature of the peat.

Every 2.5 cm. of the peat core was analyzed for cuticles by the method described (5). This resulted in a total of 283 slides being prepared and analyzed. Preceding every treatment a record was made of the actual weight of peat macerated.

After counting all cuticles for each slide, the numbers of each group were converted into percentages of the total cuticular count per slide and are represented graphically in Fig. 10. Here, the cuticular remains are divided as percentages of the following groups: Moss, Ericoid, Sedge-like, Conifer and Unknown. They are equivalent to the following coverage classes respectively: I, E, F, A & B, with the unknown possibly representing all 9 coverage classes (2).

The moss group, which has the largest numerical representation, consists of two genera; the first being Sphagnum and the second, while listed as unknown, has since been identified as Hypnum. Generally, Sphagnum predominates above the 3.45 meter level while Hypnum below. The disappearance of Hypnum from the center of the bog at the 3.45 meter level suggests a sudden change in environmental conditions. Thus, numerically, this group, despite the small size of the leaves, contributes significantly to the bulk of the peat.

The ericoid group is second in relative abundance and is present throughout the profile in varying concentrations.

The third group, consisting of cuticle with sedge-like characteristics, is concentrated mostly within the 2.5 to 5.0 meter levels and reaches a maximum at 3.2-3.6 meter levels. This maximum coincides with the level at which Hypnum disappears and it is possible that the increased numbers of sedge-like plants are a result of the change in environmental conditions postulated for this depth (5).

The conifer group is sparsely represented with a small increase at the 0.75-1.0 meter levels. The scarcity of cuticular remains from this group suggests that the leaves do not contribute very much to the structure of peat.

The unknown group is represented sporadically throughout the profile and includes all the unidentifiable cuticular remains.

It appears that the cuticles of the moss, ericoid, sedge-like, and to a lesser extent, the conifer groups, indicate the relative amounts of such plants within the structure of a given mass of peat. These relative amounts are estimated for every 2.5 cm. of profile and are illustrated in Fig. 10. This method does not reveal directly the identities nor quantities of such structural aspects as woodiness and fibrosity. With further research on this technique, there is every likelihood that a classification system of peat structure, which has so far been couched in qualitative terms, will lend itself to quantitative expression.

Assessment of Cuticular Analysis in Relation to Other Methods of Peat Analyses

The proposed method of cuticular analysis must be compared and contrasted with other recognized methods of peat study in order to gain a fuller picture of the natural organization of peat in toto, and also to establish the validity of this new approach.

In order to make this comparison, the stratigraphy, cuticular and palynological contents of the peat were examined. The section of the profile chosen from the same core to be examined was between 0.5-4.0 meter levels inclusive. This section with the results of analyses is graphically illustrated (Fig. 11).

The stratigraphy was recorded when the peat core was extracted. For the palynological comparisons and contrasts only those microfossils of plants known to be indigenous to bog habitats are included. These same microfossils are grouped in a manner similar to cuticle for interpretative purposes. The percentages of the pollen and spores are presented in the recognized manner for pollen analysis (1).

Stratigraphically, the moss components are visible throughout the section mixed with other constituents. Palynological analysis revealed only Sphagnum spores; those of Hypnum were not detected for, as yet, unknown reasons. In spite of a few local fluctuations, the Sphagnum spectra in both spores and cuticle decreases in representation with depth. Two genera of mosses were detected by both visual and cuticular means. However, it was the cuticular analysis that revealed the precise level at which Hypnum disappears as well as providing an estimate of the proportions of the moss group for a given mass of peat. If these estimates of the proportions of this group are acceptable, then it would appear that mosses contribute significantly to the structural bulk of peat (Fig. 11).

Visual examination revealed that the ericoids contribute to the woodiness of peat structure at various levels of the section. No ericoid leaves or other identifiable organs were found. Ericoid pollen predominates in the upper reaches of the spectrum and decreases with depth. Ericoid cuticle is represented in varying concentrations throughout the section (Fig. 11).

The sedge-like group is recognized throughout the section by its well-defined leafy remains. This group is well represented below the 2.0 meter level as is shown by stratigraphic, cuticular and, to a lesser extent, palynological analyses. Occasionally, relatively pure microstrata of sedge-like materials were observed; e.g., around the 3.0 meter level and 3.6-3.8 meter levels. This coincided with an increase in the cuticular remains of this group at these levels (Fig. 11). This group appears to be associated with the structural property of fibrosity. The degree of fibrosity is believed to result from the adventitious root system of this group. One may predict, then, that whenever there is an increase in the cuticle of the sedge-like group, an equivalent increase in fibrosity is likely.

The conifer group contributes a large portion of the woody component to the gross structure of peat. Woody erratics, e.g., tree stumps, fallen trunks, branches, etc., are the preserved remains of conifers that once grew on the bog. While cuticular analysis revealed little about this group's contribution to peat structure, it was noticed that following maceration of woody peat, quantities of separated tracheids and fibers were isolated that may, with further investigation, be significant in determining the degree of woodiness in peat. Pollen analysis indicates the presence of conifers throughout this section with a marked concentration within the 0.5-0.8 meter levels. This corresponds to a slight increase in cuticular representation and suggests a correlation between cuticular and palynological evidence as far as successive trends in the vegetation are concerned. The unknown group is represented sporadically throughout the section and appear to have little influence on the structure of peat.

In addition, it is shown that the stratigraphy can adequately be expressed within the limits of the 16 categories of the Radforth system (3) so illustrated in Fig. 11. In this system, the descriptive terms of the profile, such as fibrous, woody and pulpy peat are replaced by terms defining such structural qualities as non-woody, fine-fibrous, woody and non-woody particles in fine-fibrous, woody coarse-fibrous with scattered woody erratus and amorphous granular in fine-fibrous (Fig. 11). The stratigraphy also revealed the presence of thin layers of charcoal at various depths, indicative of past fires in the area.

Pollen analysis reveals a feature not recorded by the other methods of investigation, namely, the presence of such aquatic plants as Caltha, Potamogeton, Nymphaea and Nuphar. These aquatic plants, as a result of their aquatic habitat, have, in general, poorly developed cuticular and are thus rarely preserved in recognizable form. The well-preserved pollen of these plants appears below the 3.0 meter level in appreciable numbers and are indicative of an aquatic environment not revealed by the other methods of analysis.

It appears then that the stratigraphy yields information on peat structure in terms of woodiness, fibrosity, etc. Palynology tells of the floral history of the region and also indicates the succession of the vegetation of the peat area in general terms. It yields, at the most, only indirect information on the structural components of peat. Cuticular analysis appears to complement both stratigraphical and palynological examination. By this method, the principal plants that are directly involved in peat structure are ascertained upon identification of the cuticularized leaf remains. Only those plants present in appreciable numbers are registered. Other classes of surface vegetation, such as C, D, G and H, known to frequent bog habitats, are probably not concentrated enough in any volume to register in the peat. It is probable that the unknown group contains representatives of these classes. This method does not yield any information structurally on those plant remains whose contribution from cuticularized leaves is negligible.

In summing up, the structural aspects of peat are best studied by first classifying the profile using categories (3), secondly, noting the approximate botanical identity with macroscopic examination, and, thirdly, subjecting a given weight of peat to cuticular analyses. Should a more complete picture be required of the succession of vegetational types in a profile, pollen analysis should also be included.

Therefore, it appears that cuticular analysis along with the other recognized methods of peat study can be used to elucidate the many structural aspects of peat.

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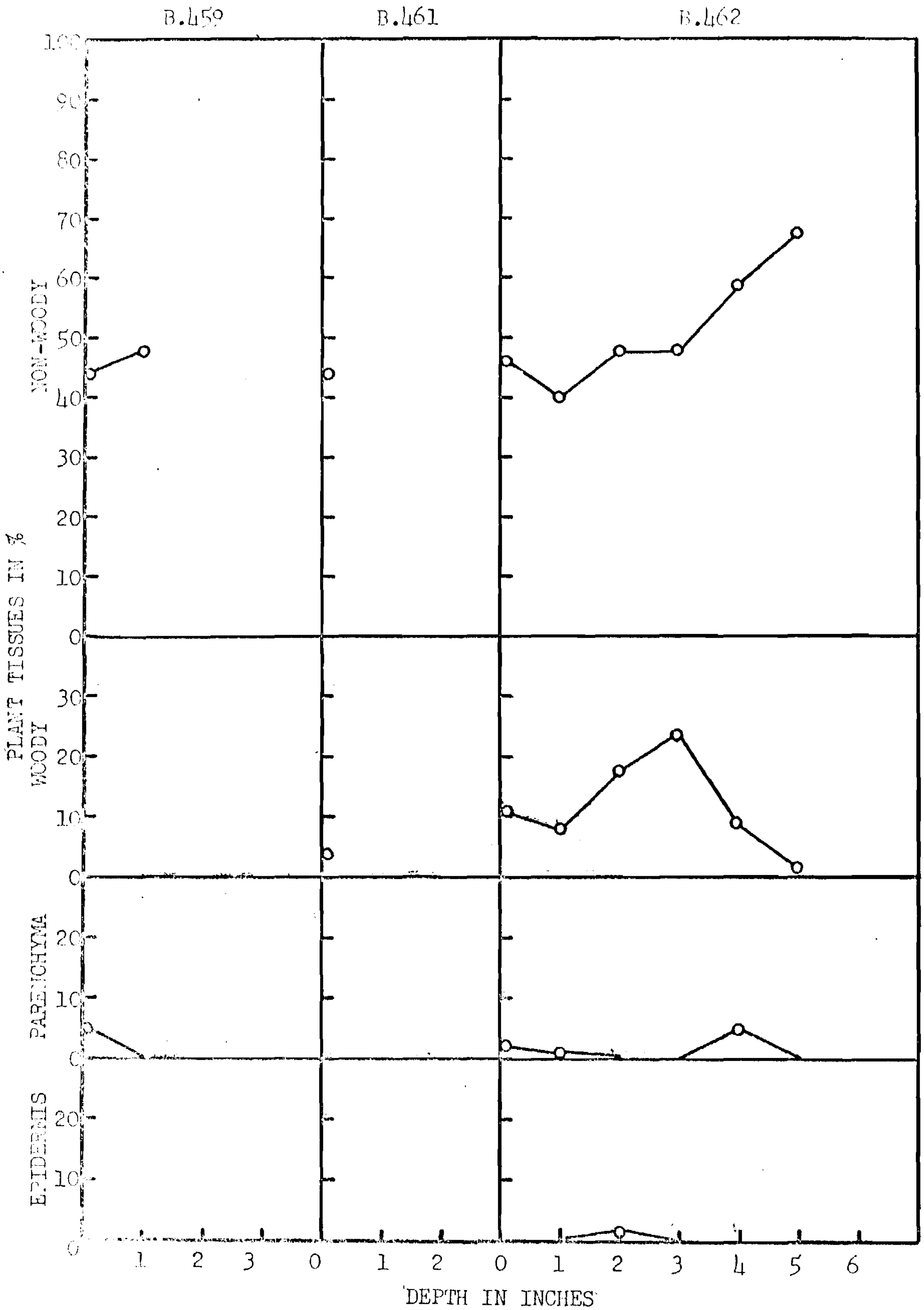

B.459
B.461
B.462
NON-WOODY
WOODY
PARENCHYMA
EPIDERMIS
PLANT TISSUES IN %
100
90
80
70
60
50
40
30
20
10
0
DEPTH IN INCHES
1
2
3
4
5
6

FIG. 1

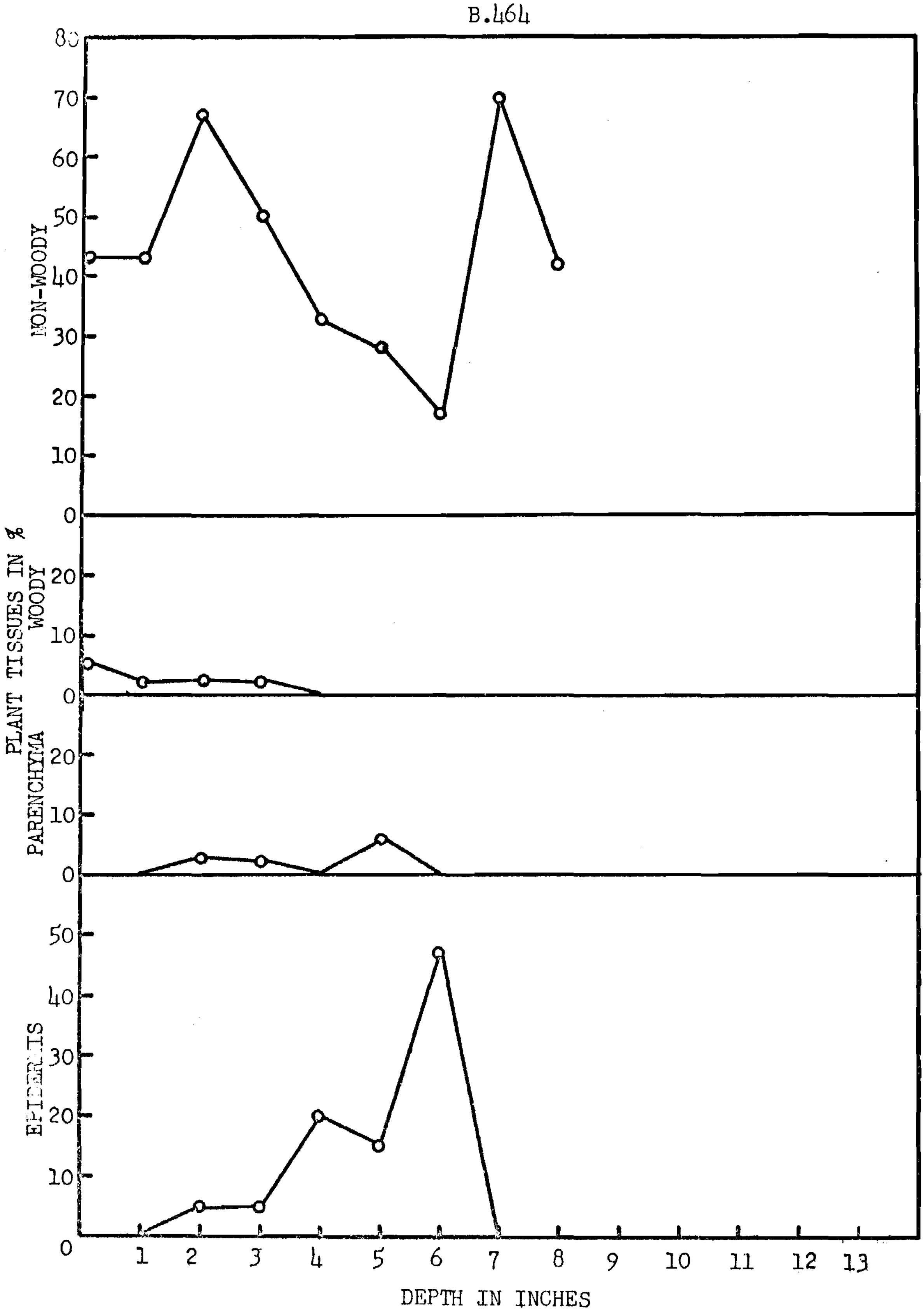

B.464
80
70
60
50
40
30
20
10
0
NON-WOODY
PLANT TISSUES IN %
WOODY
PARENCHYMA
EPIDERMIS
1
2
3
4
5
6
7
8
9
10
11
12
13
DEPTH IN INCHES

FIG. 2

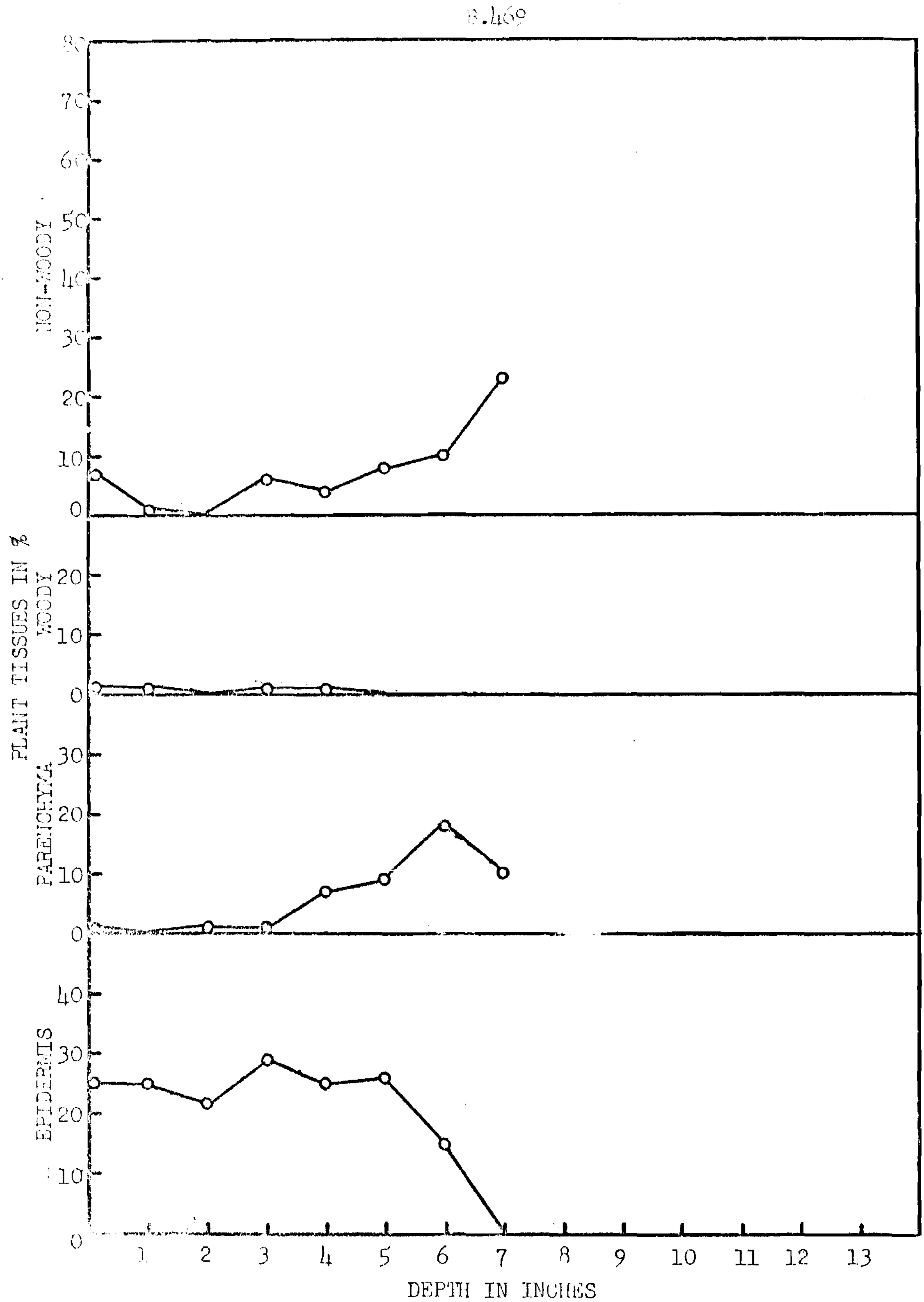

B.469
NON-WOODY
WOODY
PARENCHYMA
EPIDERMIS
PLANT TISSUES IN %
DEPTH IN INCHES

FIG. 3

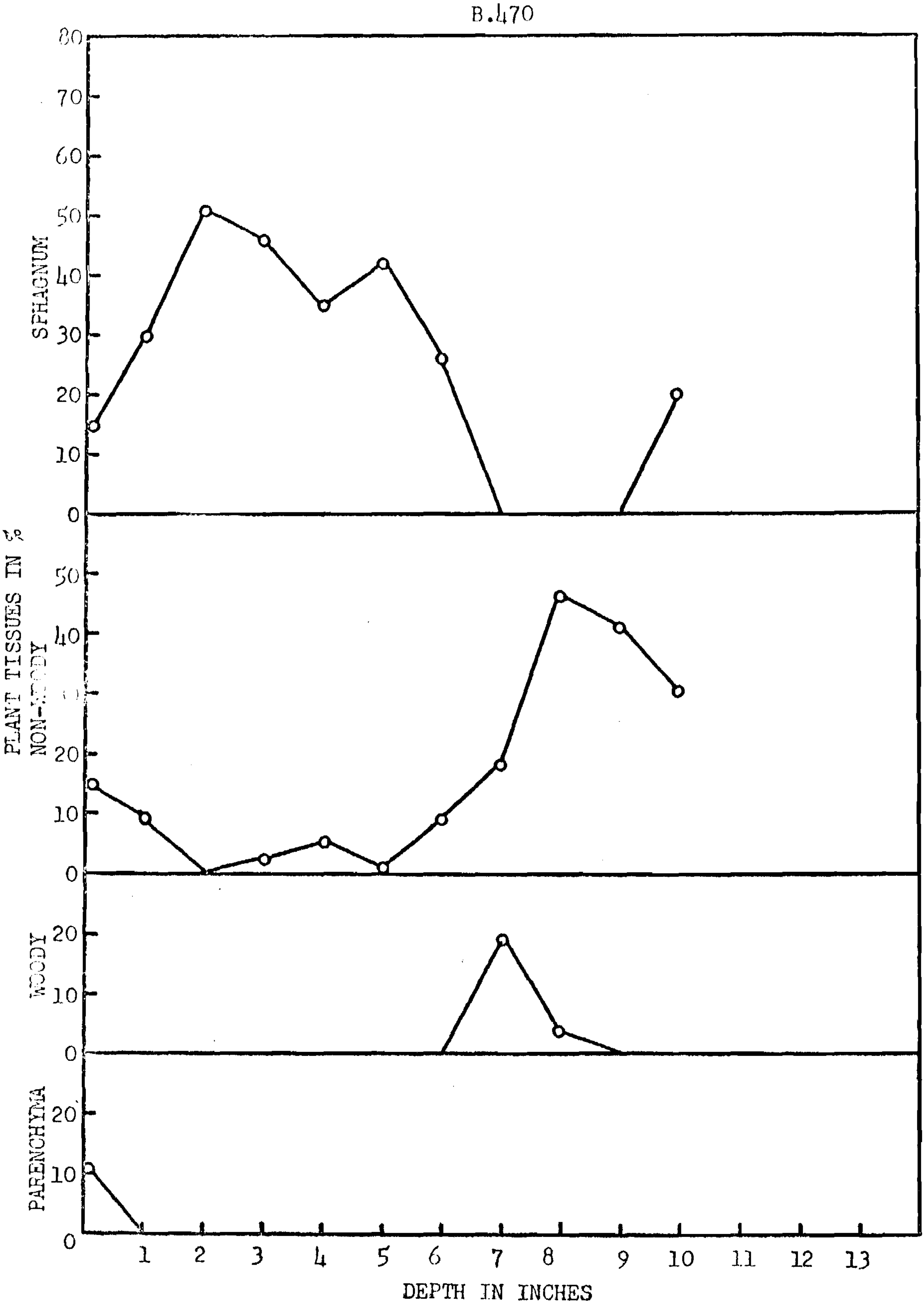
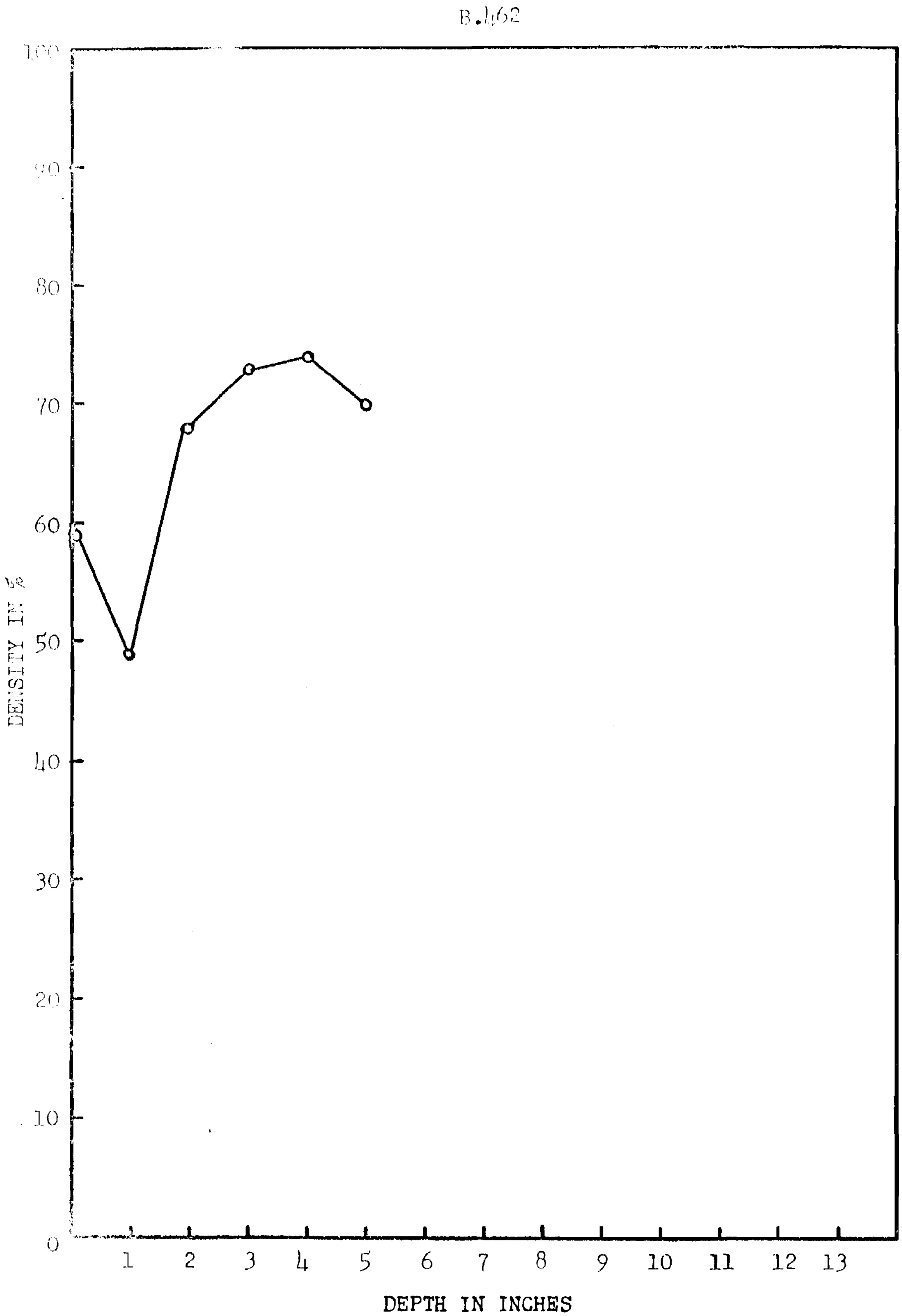

B.470
PLANT TISSUES IN %
SPHAGNUM
NON-WOODY
WOODY
PARENCHYMA
80
70
60
50
40
30
20
10
0
DEPTH IN INCHES
1
2
3
4
5
6
7
8
9
10
11
12
13

FIG. 4


B.462
DENSITY IN %
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90
80
70
60
50
40
30
20
10
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1
2
3
4
5
6
7
8
9
10
11
12
13
DEPTH IN INCHES

FIG. 5

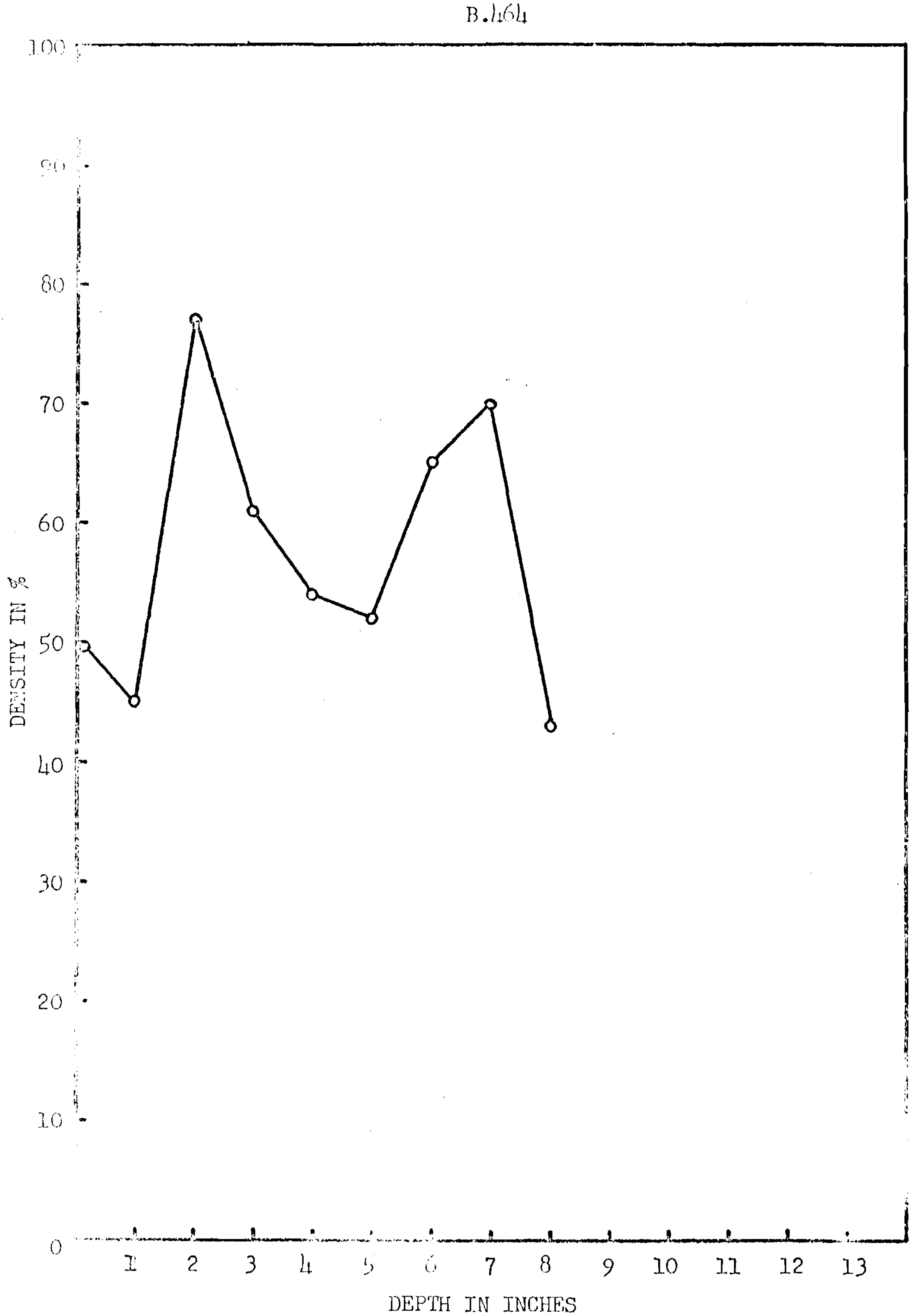
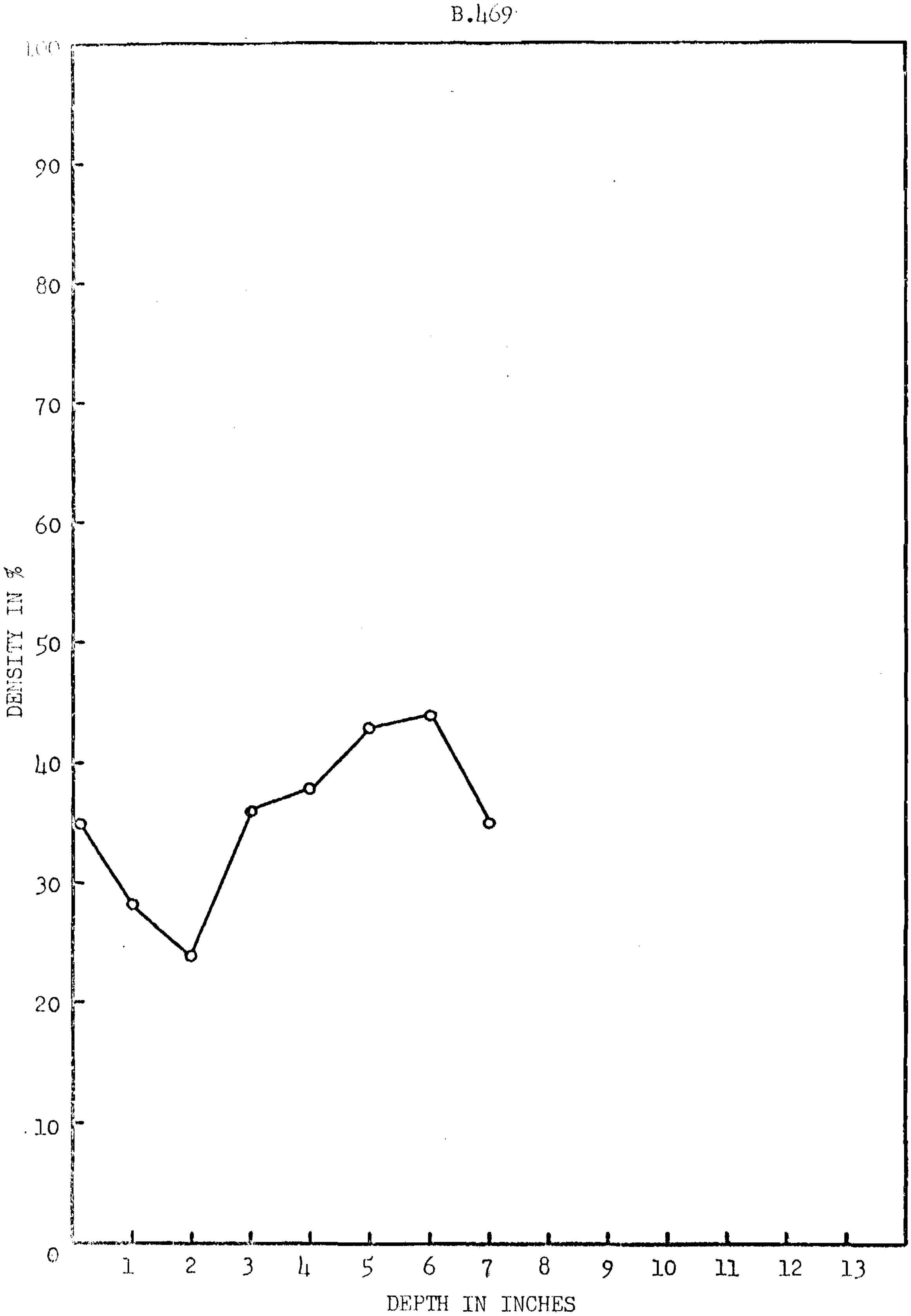

B.464
100
90
80
70
60
50
40
30
20
10
0
DENSITY IN %
1
2
3
4
5
6
7
8
9
10
11
12
13
DEPTH IN INCHES

FIG. 6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100
90
80
70
60
50
40
30
20
10
0
DENSITY IN %
1
2
3
4
5
6
7
8
9
10
11
12
13
DEPTH IN INCHES

FIG. 7

B.470

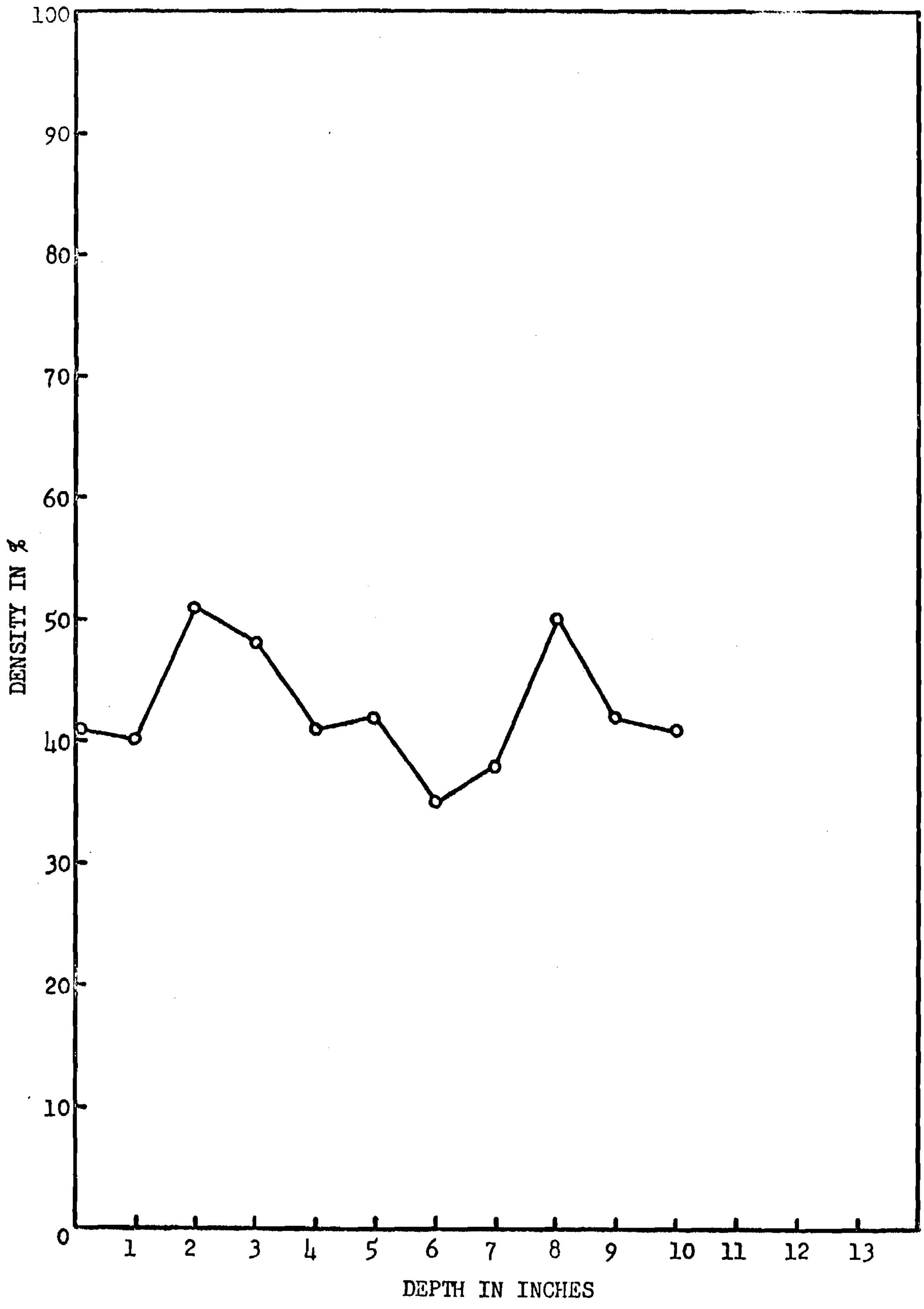

100
90
80
70
60
50
40
30
20
10
0
DENSITY IN %
1
2
3
4
5
6
7
8
9
10
11
12
13
DEPTH IN INCHES

FIG. 8

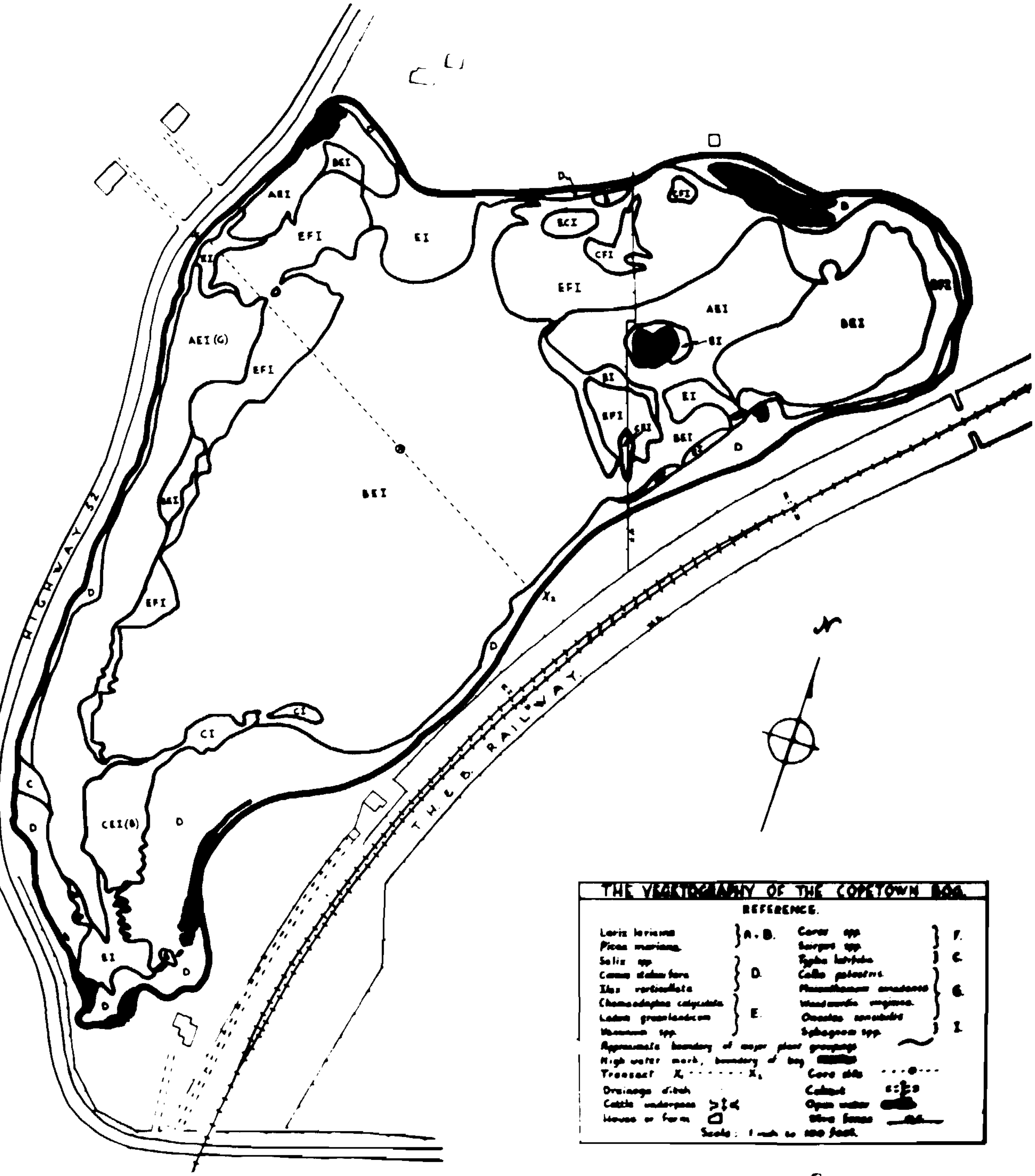


FIG. 9

Carex spp.
Scirpus spp } F
Typha latifolia } C
Calla palustris
Malanthemum canadensis
Woodwardia virginica
Onoclea sensibilis } G
Sphagnum spp. } I
Core site - - - ● - - -
Culvert
Open water
WIRE FENCE

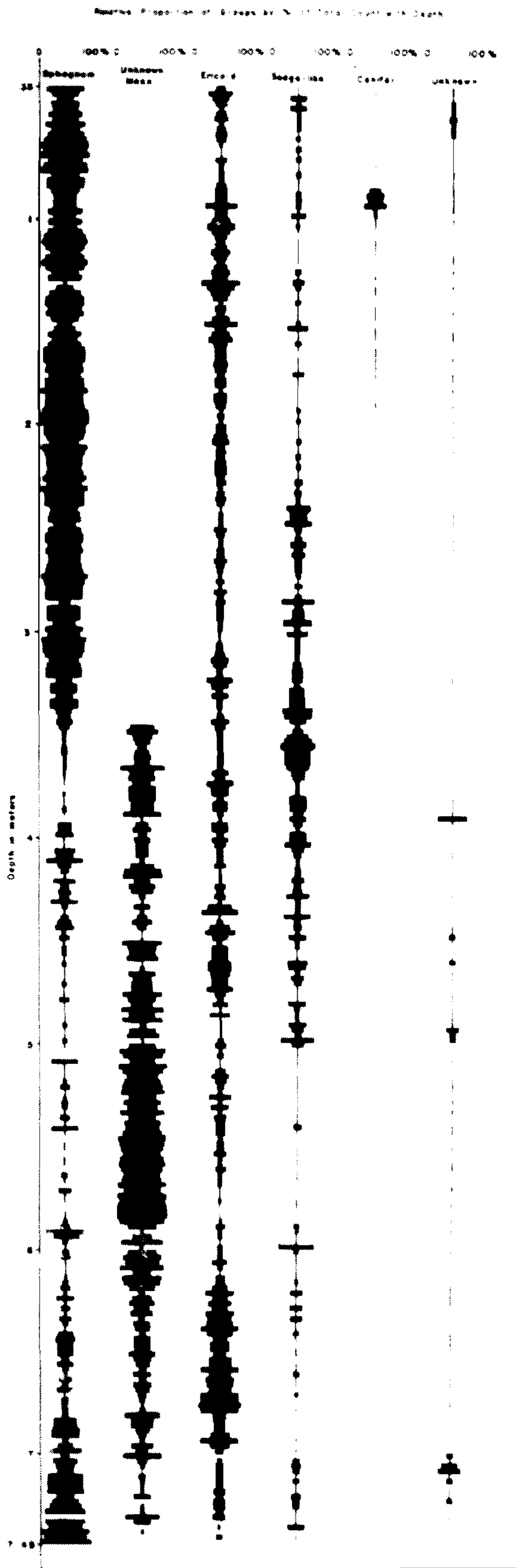
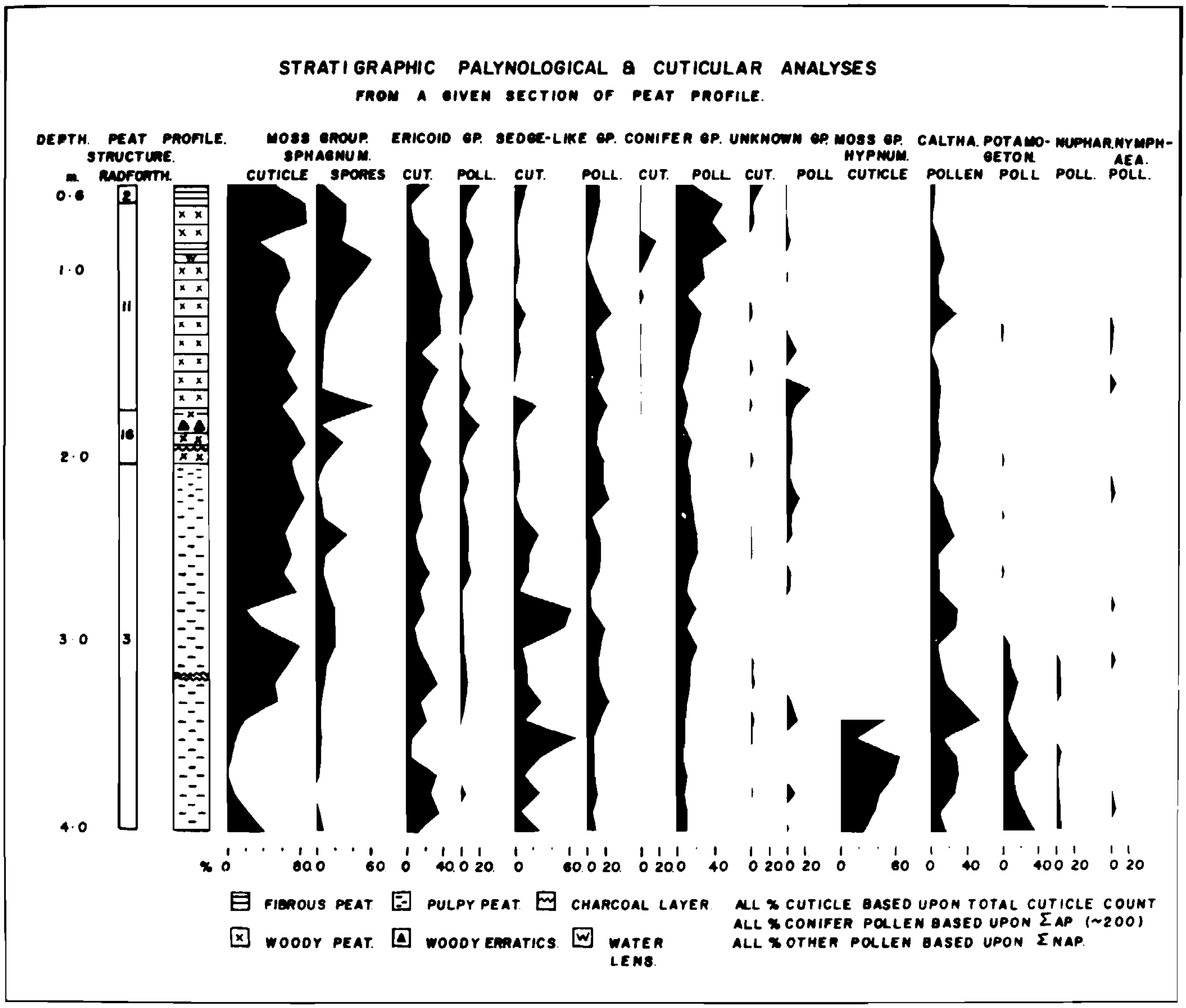

0
100%
Sphagnum
Unknown Moss
Ericoid
Sedge-like
Conifer
Unknown
Depth in meters
1
2
3
4
5
6
7

FIG. 10



STRATIGRAPHIC PALYNOLOGICAL & CUTICULAR ANALYSES
FROM A GIVEN SECTION OF PEAT PROFILE.
DEPTH. m.
PEAT STRUCTURE. RADFORTH.
PROFILE.
MOSS GROUP. SPHAGNUM.
CUTICLE
SPORES
ERICOID GP.
CUT.
POLL.
SEDGE-LIKE GP.
CUT.
POLL.
CONIFER GP.
CUT.
POLL.
UNKNOWN GP.
CUT.
POLL.
MOSS GP. HYPNUM.
CUTICLE
CALTHA.
POLLEN
POTAMOGETON.
POLL.
NUPHAR.
POLL.
NYMPHAEA.
POLL.
0·6
1·0
2·0
3·0
4·0
%
FIBROUS PEAT
PULPY PEAT
CHARCOAL LAYER
WOODY PEAT
WOODY ERRATICS
WATER LENS
ALL % CUTICLE BASED UPON TOTAL CUTICLE COUNT
ALL % CONIFER POLLEN BASED UPON ΣAP (~200)
ALL % OTHER POLLEN BASED UPON ΣNAP

FIG. 11

Discussion

Mr. Hemstock (Imperial Oil) asked if there was any evidence to show that there is a change in the type of living plants after burn-over by forest fire. Dr. Radforth said that fire seems to be casual; that is, the evolution of the surface cover back to what it was before the fire can be observed in one generation. It is not possible to see the complete return, but a trend is evident. For a short time after a fire there is a drying out of the peat on the top, and consequently a change in the chemical constitution. But soon -- geologically speaking -- there is a reversion to the kind of organization basic to the mass of peat itself.

Mr. Tomlinson (Spartan Air Services) commented that it has been found that in the Northwest Territories it takes one hundred years for the surface cover to come back to the original vegetation -- for example, to a full blossoming stand of caribou moss. In contrast, in Central Quebec, it takes only thirty years for a full stand to come back.

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I. 4 LABORATORY COMPRESSION TESTS ON PEAT

J. I. Adams

ABSTRACT

Laboratory tests on samples of peat obtained from the James Bay area in Northern Ontario are presented. Consolidation tests consisting of one-dimensional tests of both short term and long term duration and triaxial consolidation tests with measured pore pressures were carried. Triaxial strength tests were performed consisting of undrained compression tests with measured pore pressure and drained compression tests. The results indicate that the consolidation of the peat under load is mainly the expulsion of pore water under excess hydrostatic pressure which is extended to relatively long duration by a large reduction in permeability. The triaxial compression tests show an exceptionally high drained strength but the undrained strength is shown to be very sensitive to pore pressure development.

****

The testing of peat by techniques commonly used for determining the engineering properties of soil have been reported in many papers and publications and the results in many cases are conflicting and difficult to interpret. Questions have arisen as to whether or not peat can be expected to behave in a similar manner to mineral soil when subjected to external loads and whether or not any of the now accepted soil testing methods are appropriate for testing peat.

The presence of muskeg and the associated deposits of peat in Northern Ontario have a marked influence on the feasibility of hydro-electric schemes and on the design and construction of access roads and transmission lines. Reliable means of testing peat and analyzing results are needed therefore if the engineering behaviour of the material is to be predicted. Some preliminary testing of peat has been carried out in Ontario Hydro's soils laboratory in order to obtain some understanding of the engineering behaviour of this material and to determine, if possible, what tests will be useful in predicting field behaviour. This paper describes some of the testing which has been done and comments on the significance of some of the results and also on the experience of others who have conducted similar work.

The samples of peat tested were obtained from sites in the Cochrane-Kapuskasing area, at locations near (a) the Abitibi Canyon Generating Station (Abitibi River) and (b) the Little Long Rapids Generating Station (Mattagami River). The cover classification at both sites according to Radforth (7) was chiefly AI. The peat classification was predominantly woody fine-fibrous (Categories 9 and 11).

Sampling of the peat was accomplished with 2-inch OD Shelby tube samplers pushed into the peat. No particular measures were used to obtain even relatively undisturbed samples and considerable compression of the peat took place during insertion of the sampling tubes. In addition bulk soil samples were obtained for remoulded tests.

The results of typical classification tests are given below:-

	Range
Moisture Content (as sampled) % of dry weight	375 - 430
pH	4.8 - 6.3
Specific Gravity	1.62 - 1.70
Ash Content % of dry weight (Based on loss on ignition)	12.2 - 22.5

CONSOLIDATION TESTS

Probably the most obvious characteristic of all peat types is the exceptionally high compression under load. Many conventional and non-conventional tests have been reported on various types of peat and there appears to be divided opinion as to what type of consolidation occurs. The shape of the time-settlement curves in most of the reported work indicates similar behaviour, namely, a large amount of initial compression occurring very rapidly, then a progressively lower rate of settlement approaching a straight line relationship with time. The interpretation of the results has suggested in many cases that the consolidation of peat is largely of a secondary nature. See for example, Lewis (6), Buisman (1) and Thompson (8). On the other hand, the work published by Hanrahan (3) and Colley (2) indicated that the rate of consolidation of peat was largely dependent upon the length of drainage path, suggesting that an appreciable portion of the consolidation was of a primary nature. Primary consolidation will be considered as that occurring under excess hydrostatic pressure and secondary consolidation will be considered that occurring under zero or negligible excess hydrostatic pressure.

The following tests were carried out in hope of resolving some of these uncertainties:-

(a) Settlement - Permeability Tests

A consolidation test was carried out on a sample of remoulded peat in a settlement-permeability consolidometer. The initial thickness of the sample was approximately 3.5 inches and the diameter was 8 inches. The load was raised in increments to 5900 p.s.f. and then unloaded. Each increment was left on approximately 19 hours and dial readings of settlement were taken at frequent intervals. At the end of each loading period a falling head permeability test was carried out. Time-settlement curves and a void ratio vs. log of pressure curve are shown in Fig. 1.

The permeability plotted against the change in volume is shown in Fig. 2. It should be noted that the permeability is reduced significantly with increase in load, changing from approximately 1×10^{-1} ft/min.to 1×10^{-6} ft/min. which is equivalent to that of a relatively pervious sand and a relatively impervious silt, respectively.

(b) Triaxial Consolidation Tests

Consolidation tests with measured pore pressure were run on a 4-inch by 1.9-inch sample of peat. The sample was tested in the triaxial machine at 10 p. s. i. and then at 20 p. s. i. all around pressures and measurement of water discharge was made by burette readings. The sample was allowed to consolidate under the confining pressures for 5 days and 14 days, respectively. Pore pressures were measured at the bottom of the sample periodically. An additional test was carried out under a 10 p. s. i. confining pressure in which total volume change indicated by cell water measurements and volume change due to water discharge were compared. The curves of pore pressure and volume change with time are shown in Fig. 3.

In these tests, the pore pressure reduced with time from a value equal to the consolidating pressure to zero at a point on the time-settlement curve where the settlement becomes linear with time. It will be noted that the compression under excess pore pressure represents the majority of the total compression measured. In the test in which the total compression is compared with the total volume of water discharged, it is indicated that approximately 90 per cent of the consolidation is due to expulsion of water.*

(c) Long Term Compression Test

A long term consolidation test was performed on a sample of peat 3.0 inches thick and 8.0 inches in diameter. A load of 2180 p. s. f. was applied and left on for a period of approximately one year. The settlement plotted against the square root of time is shown in Fig. 4, on scales showing both short term and long term settlement. On completion of the test the sample was unloaded and the final rebound as indicated on Fig. 4 was approximately 7 per cent.

Discussion of Consolidation Tests

The long term consolidation test showed continuing settlement, indicating that in the field consolidation of peat will continue for an extremely long period of time. The majority of the settlement however did occur rapidly within the first week of the test. It was also shown that the permeability of peat is reduced significantly during consolidation. This reduction is most pronounced under initial loading. During

*Prior to consolidating the sample, the cell pressure was raised in increments to 50 p. s. i. with the drainage valve closed; no measurable change in volume occurred indicating that the sample was saturated.

the pore pressure dissipation tests the pore pressure period was 1 day for the sample consolidated under 10 p. s. i. and 3 days for the sample consolidated from 10 to 20 p. s. i. In both cases the majority of the settlement occurred during the excess pore pressure period. It is thought that the dramatic reduction in permeability extends the pore pressure period of consolidation with the result that the major proportion of the settlement occurs under excess hydrostatic conditions.

Hanrahan (3) and Lea (5) have reported results in which the rate of consolidation of peat was accurately predicted using a simplified version of Terzaghi's primary consolidation theory, i. e. :-

$$t \text{ field} = \frac{H^2 \text{ field} \times t \text{ lab}}{H^2 \text{ lab}}$$

and

$$s \text{ field} = \frac{H \text{ field} \times s \text{ lab}}{H \text{ lab}}$$

in which t = time for consolidation

H = thickness of layer

s = settlement

These results would substantiate that the consolidation measured occurred under an excess hydrostatic condition. It is thought that this relationship is a convenient method of computing field settlement and our present knowledge of the consolidation of peat probably warrants no further refinement. It may be noted that Lake (4) has reported results of tests on peat which do not support the above behaviour.

SHEAR TESTING

The shear testing of peat presents many problems both in place and in the laboratory. Securing of undisturbed samples is exceedingly difficult, if not impossible. The fibrous nature of the material, presence of woody erratics and the high compressibility all contribute to problems in testing. The published tests on peat indicate conflicting behaviour and opinion regarding the strength of peat varies from entirely cohesive to entirely frictional.

The shear tests carried out by Hanrahan (3) are particularly interesting and led to some of the tests which follow. Included in his work were consolidated undrained triaxial tests carried out with measured pore pressure. The pore pressures in the consolidated samples developed very rapidly during shear and came close to the confining pressures at failure.

(a) Consolidated Undrained Triaxial Tests with Measured Pore Pressures

Undrained tests with measured pore pressure were carried out on saturated samples 4.5 x 1.9 inches consolidated to 2, 10, 20 and 30 p.s.i. The samples were stressed to failure at a rate of approximately 0.005 inch per minute. In each test the build-up of pore pressure was rapid, coming within a few pounds per square inch of the confining pressure at failure.

The results in terms of effective stresses, the stress-strain relationships and the pore pressures developed are shown in Fig. 5. A plot of total strength to consolidation pressure is included in Fig. 5. It will be noted that due to the high pore pressure development in the undrained tests the corrected Mohr circles in terms of effective stress indicate a very high drained strength. ($\phi' = 50^{\circ}$).

(b) Drained Triaxial Tests

Two drained tests were carried out on samples consolidated initially under 10 p.s.i. and 20 p.s.i. respectively. The tests were carried out by dead loading i.e., applying a small increment of load and allowing the sample to consolidate completely under each increment. It was found convenient to apply the load in 3 and 6 p.s.i. increments for the 10 and 20 p.s.i. consolidated samples, respectively. Measurement of consolidation was recorded by making periodic burrette discharge readings. The tests were carried out over a period of approximately 3 months and failure of the samples finally occurred after an extremely high strength development accompanied by large axial deformation.

Stress-strain curves and Mohr stress circles are shown in Fig. 6. Although the final length to diameter ratios were slightly less than one, the stresses measured are generally close to the Mohr envelope obtained from the undrained tests.

(c) K_o Test

The K_o value of soil is the ratio of the horizontal effective stress (σ_3^{o}') to the vertical effective stress (σ_1') for a condition of zero lateral strain during compression. The consolidation of natural soil due to its own weight is generally considered to occur in this manner. The K_o value for soil has been reported to vary between 0.4 to 0.8. During the drained compression tests on peat, very little lateral deformation was observed during the entire loading period and it was thought that the K_o value of the peat must be remarkably low.

A K_o test was carried out by dead loading a sample of peat in a manner similar to the drained triaxial tests. The lateral pressure was adjusted to maintain the K_o condition of zero lateral deformation. This was accomplished by measuring the sample volume change by cell water displacement and allowing volume change in the vertical direction only by manipulation of the vertical and lateral loads. The sample used for this test was that used for comparing volume change and water discharge under a 10 p.s.i. confining pressure. The K_o test was started on the sample after it had been unloaded to zero pressure and had come to equilibrium.

The measured K_o value plotted against deformation is given in Fig. 7. It will be noted that the K_o value is initially of the order of 0.5 and reduces with increased load to a value of the approximate order of 0.175 at a deformation of 0.4 inches. The initial higher values are thought to be due to the pre-consolidation caused by the initial 10 p.s.i. consolidating pressure.

Discussion of Strength Tests

The triaxial compression tests on isotropically consolidated peat show a relatively high pore pressure development under undrained conditions and results corrected for pore pressure show a high angle of shearing resistance. The drained tests show an exceedingly high strength development and in general substantiate the undrained tests in terms of effective stress. A low value of K_o was measured which confirmed the behaviour of a small lateral strain observed during the drained compression tests. It is thought that the low K_o value of peat is a significant factor in determining the behaviour of peat under compressive loads.

The low K_o behaviour of peat indicates that under anisotropic consolidation the horizontal effective stress is small in comparison to the vertical effective stress and consequently appreciable shearing stresses are set up which increase with the effective consolidating pressure. The stress envelope for $K_o = 0.175$ consolidation as shown in Fig. 8. It will be seen that a relatively high angle of shearing resistance is required for equilibrium during consolidation. It will be further noted that, for the case cited, a relatively small positive pore pressure would result in failure or overstressing. The tests show therefore that a high strength can be developed by consolidating the peat providing the loading is not accompanied by appreciable pore pressure development.

CONCLUSIONS

The consolidation of the peat tested is predominantly due to the expulsion of water under excess hydrostatic pressure. A large proportion of the settlement takes place very rapidly when the material is relatively pervious. With time the permeability is reduced with a corresponding reduction in the rate of settlement. It is argued that the consolidation occurring under excess pore pressure is extended to relatively long term compression by the large reduction in permeability.

The compression tests on peat show an exceptionally high drained strength on axial loading. Consolidated undrained tests corrected for pore pressure are in general agreement with the drained tests. The peat was found to have a low K_o value which indicates that under anisotropic consolidation high shear stresses are developed. The total strength of the peat can vary therefore from very low values to relatively high values depending on the pore pressures set up during consolidation. It is evident therefore that a consideration of pore pressure is very important in analyzing the strength of peat.

ACKNOWLEDGMENT

The author is indebted to Mr. D. Bazett and Mr. S. Smotrych for their criticisms and helpful suggestions. The co-operation of Mr. T. R. Allan and Mr. Straghan who conducted the laboratory work and prepared the data is appreciated.

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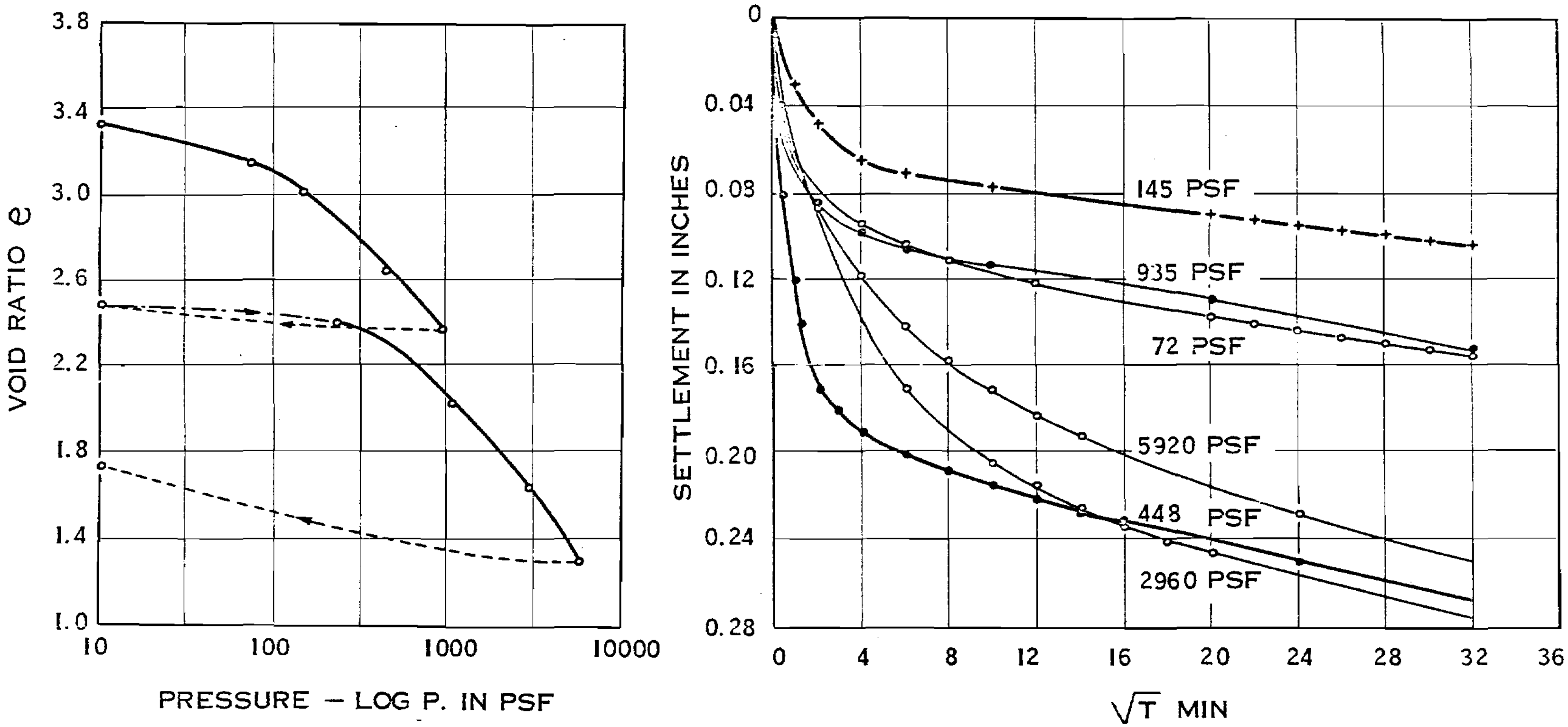


[A] PRESSURE — VOID RATIO

[B] TIME — SETTLEMENT

SAMPLE THICKNESS [INITIAL] — 3.5 INCHES
SAMPLE DIAMETER — 8.0 INCHES

FIGURE I

CONSOLIDATION TESTS ON PEAT

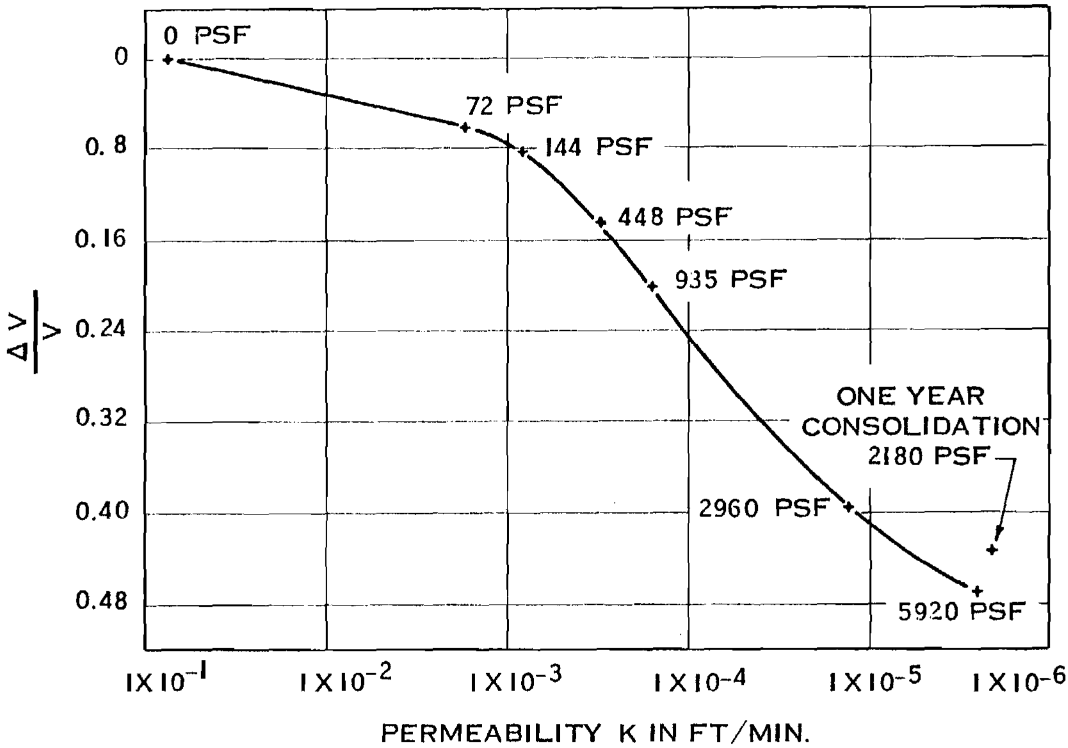
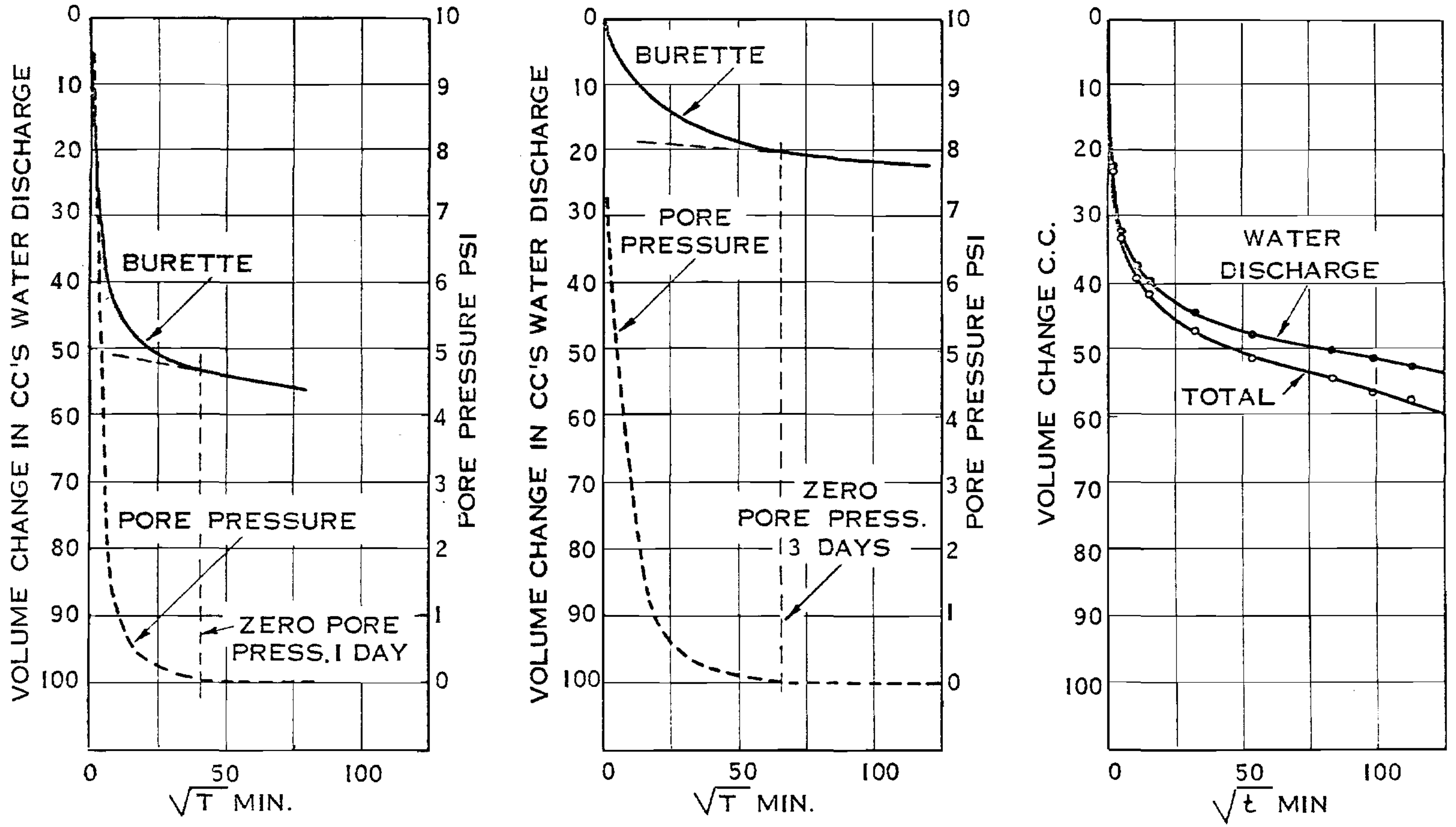


FIGURE 2

PERMEABILITY VS. VOLUME CHANGE DUE TO CONSOLIDATION



PORE PRESSURE AND VOLUME CHANGE p — 0 — 10 PSI [A]

PORE PRESSURE AND VOLUME CHANGE p — 10 — 20 PSI [B]

VOLUME CHANGE [TOTAL] AND VOLUME CHANGE [WATER] p — 0 — 10 PSI [C]

FIGURE 3

TRI—AXIAL CONSOLIDATION TESTS ON PEAT SHOWING PORE PRESSURE AND VOLUME CHANGE

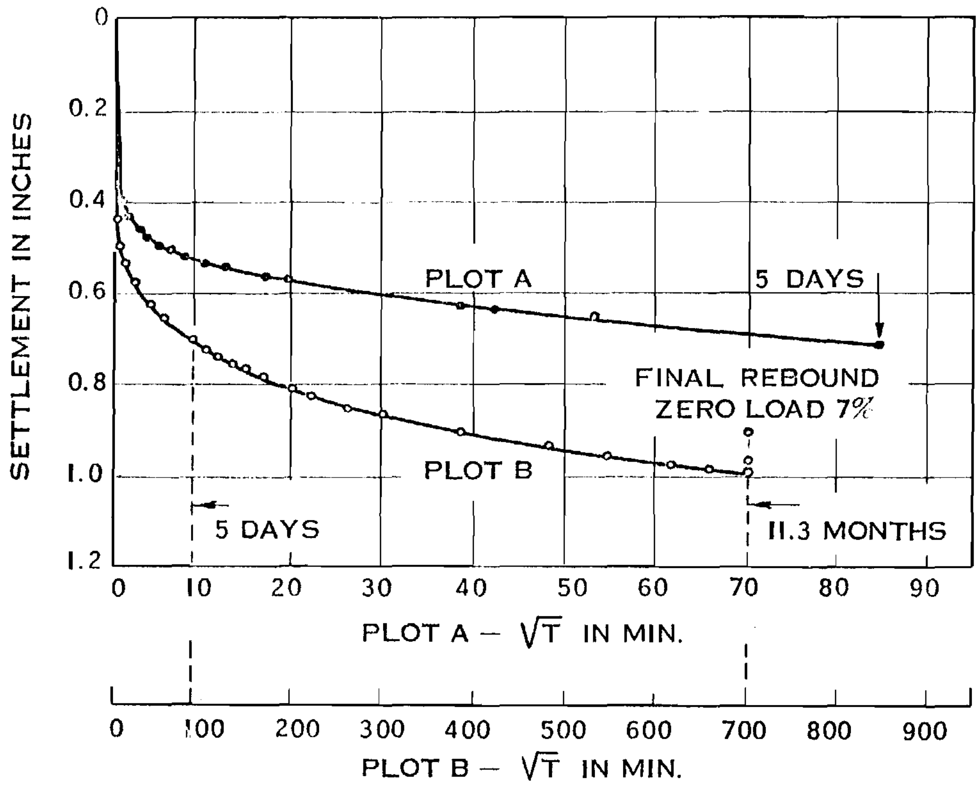


FIGURE 4

LONG TERM CONSOLIDATION TEST ON PEAT

LOAD 2180 PSF
SAMPLE THICKNESS — 3.0 INCHES
SAMPLE DIAMETER — 8.0 INCHES

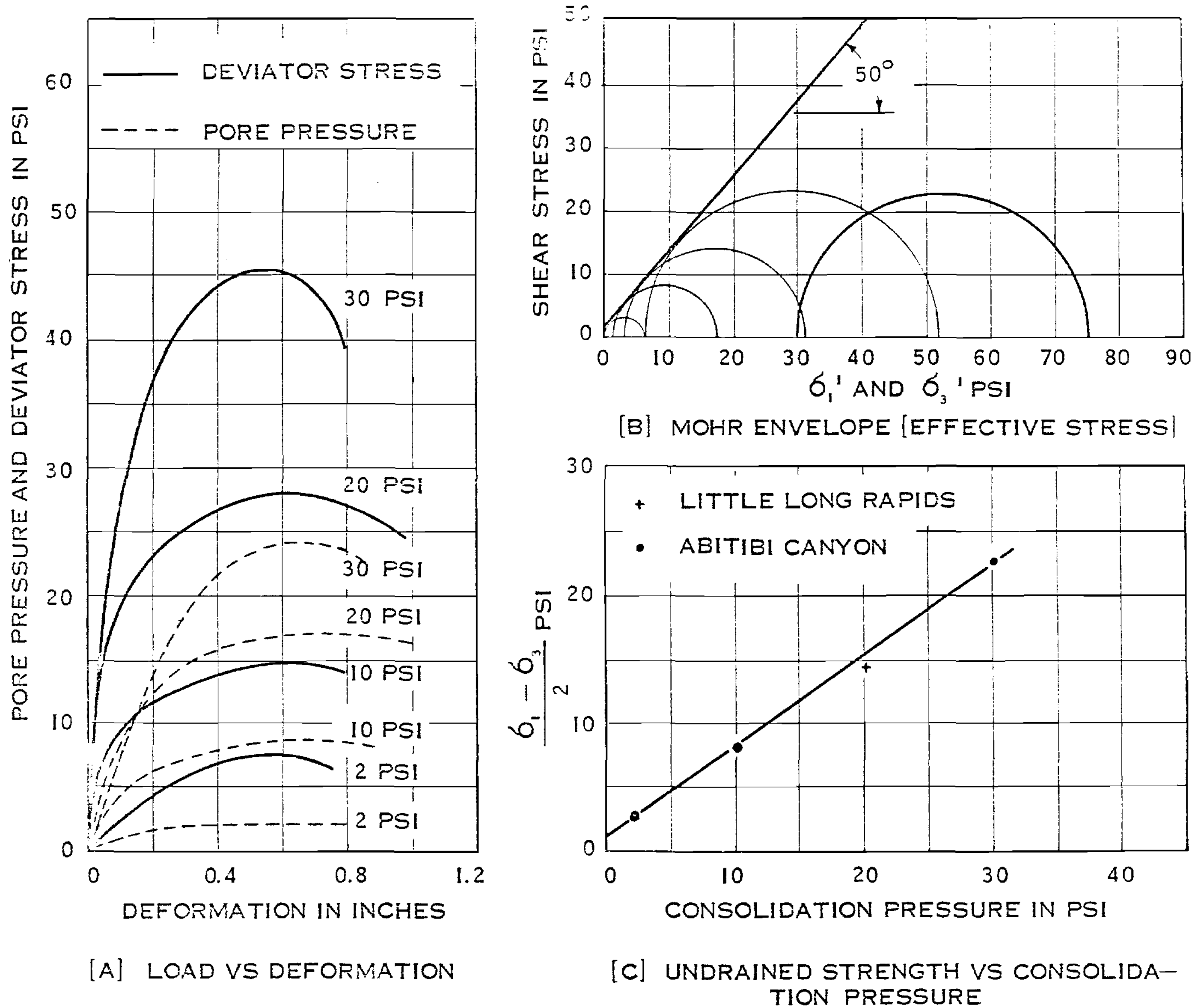


FIGURE 5

CONSOLIDATED UNDRAINED TRIAXIAL TESTS ON PEAT

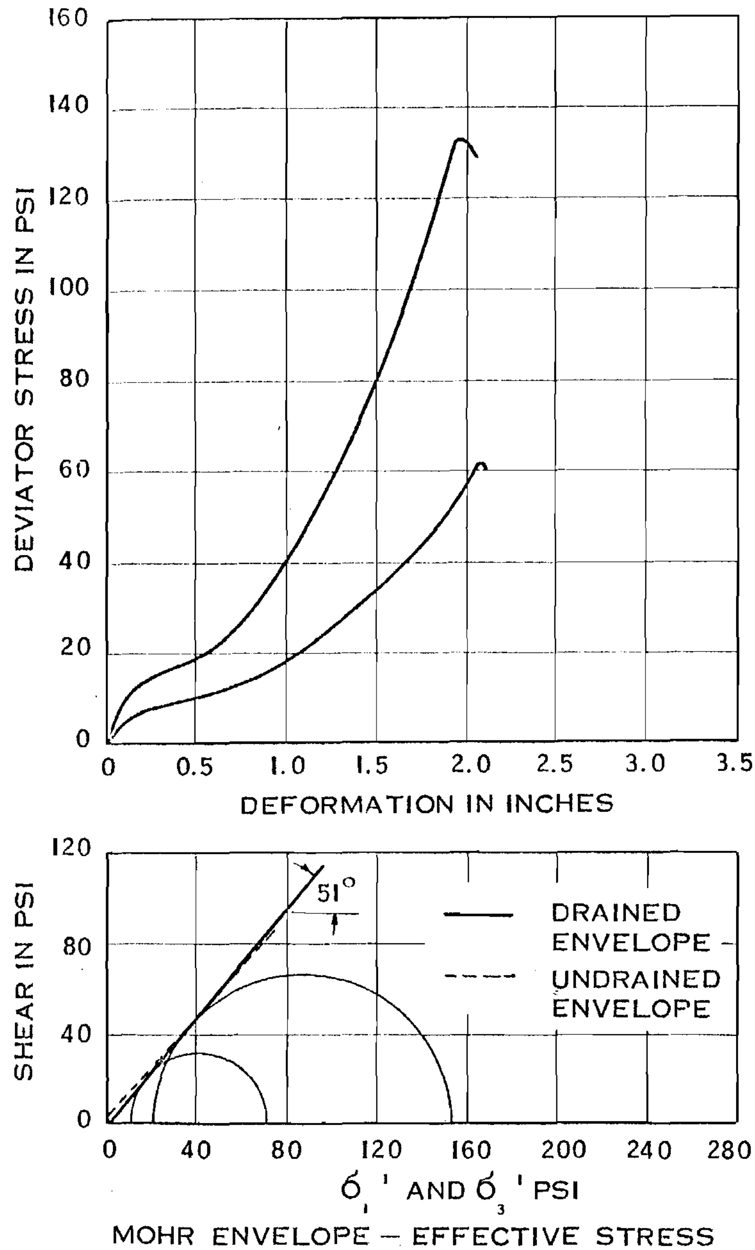


FIGURE 6

DRAINED COMPRESSION TESTS ON PEAT

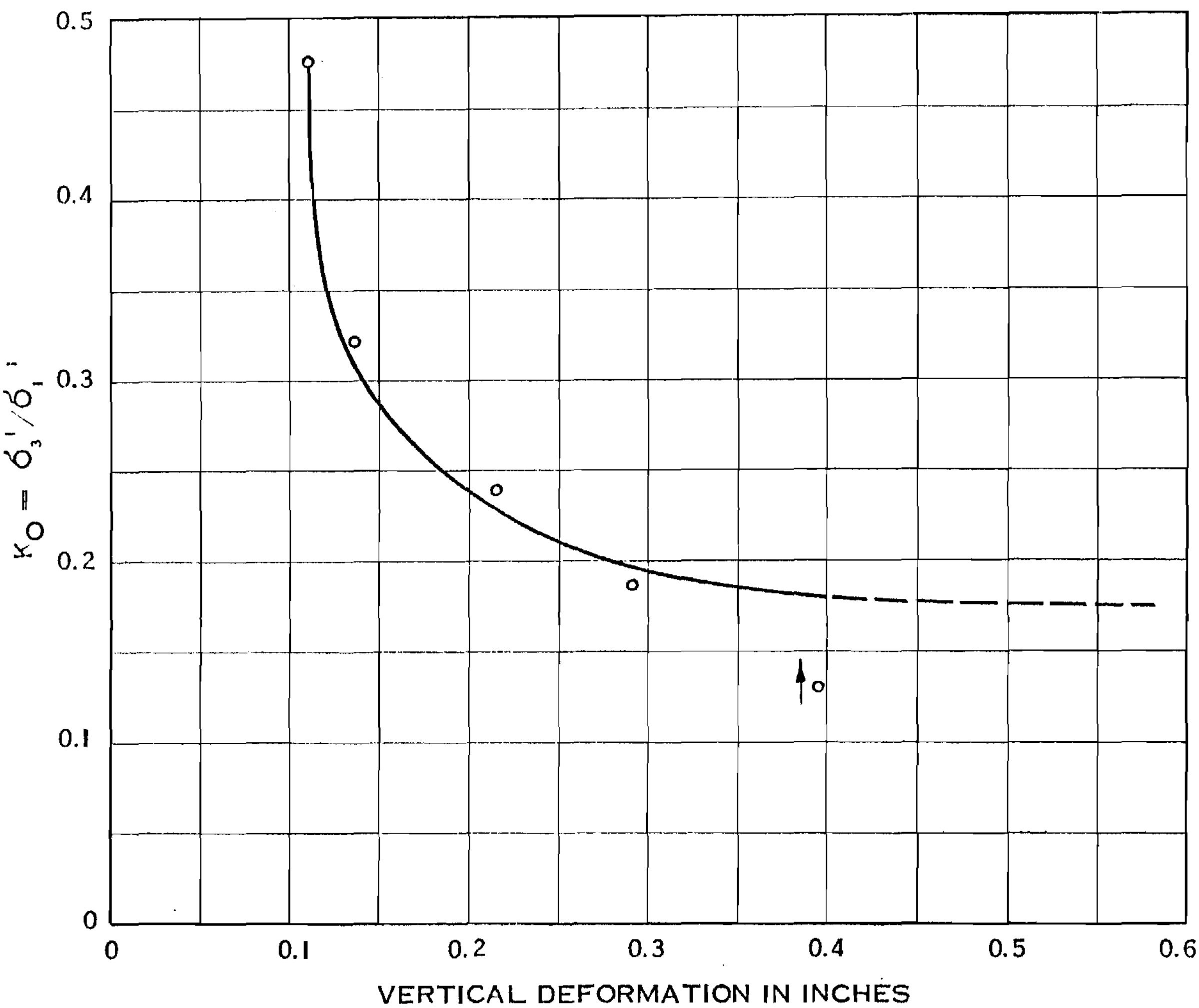


FIGURE 7

RESULTS OF K_O TEST

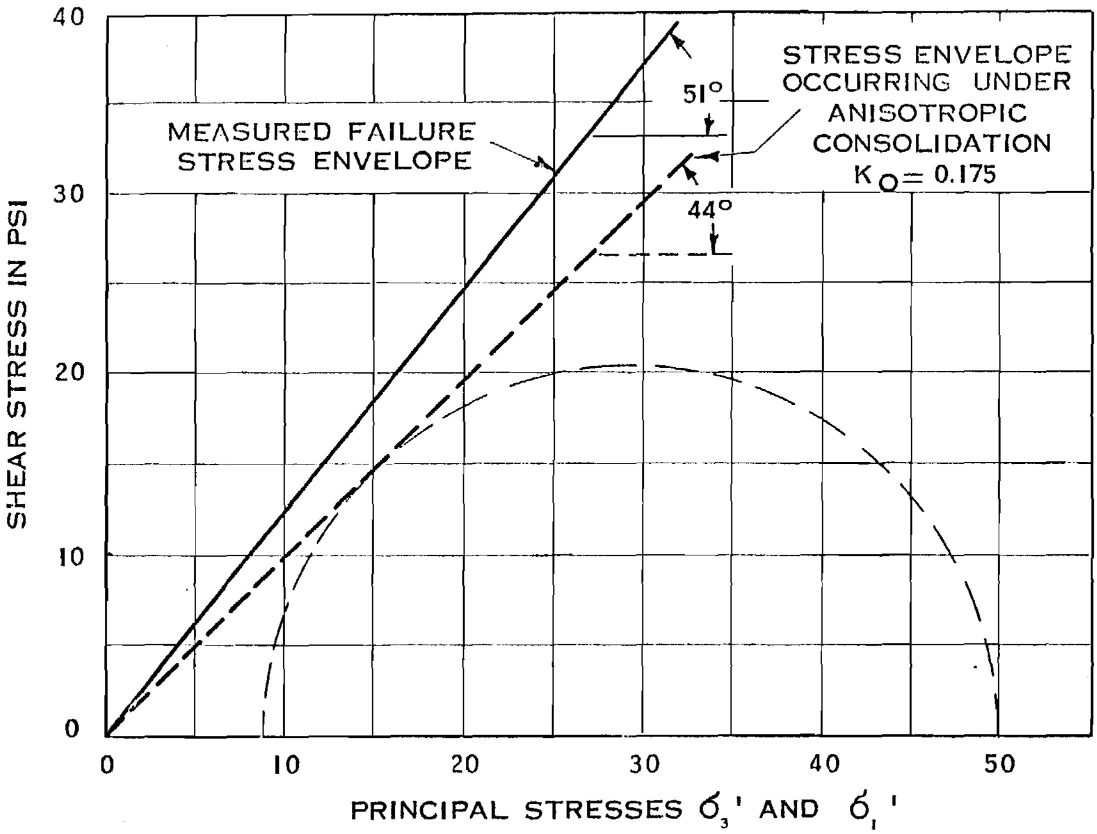


FIGURE 8

STRESS CONDITION DURING K_O CONSOLIDATION

Discussion

Dr. Chu (Tippetts-Abbott-McCarthy-Stratton) questioned Mr. Adams on his analysis of consolidation of peat. He wondered if settlement had been plotted against log-time. Mr. Adams replied that some results were plotted on the log-time scale. However, there appeared to be no advantage to using this scale for peat. Dr. Chu further queried if any triaxial tests had been carried out in the direction perpendicular to sampling (i.e. in the horizontal plane). Mr. Adams stated that all testing was carried out in the vertical plane of the peat. Dr. Golder asked if any tensile tests had been carried out; Mr. Adams said that a suitable method could not be found to do this test on peat.

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I. 5. CORROSION OF CONCRETE STRUCTURES BY MUSKEG WATERS

I. C. MacFarlane

Corrosion of concrete has long been a matter of considerable economic importance. Consequently, in various countries extensive studies of chemical attack on this material have been carried out. The great bulk of this research, however, has concerned itself with the problem of sulphate attack on concrete -- doubtless because in practice the most common cause of chemical deterioration is by sulphates. Relatively little detailed information is available on the corrosion of Portland cement concrete by acids -- and in particular by acidic muskeg waters.

A few years ago, the Division of Building Research undertook a literature survey on the subject of corrosion of concrete by aggressive muskeg waters, at the request of the Trans-Canada Telephone System which was engaged at the time in the construction of the Mid-Canada Radar Line. The subsequent report was of a semiprivate nature, however, and did not receive wide distribution.

Increased construction activities in Northern Canada have given rise to questions regarding the effect of the water which is always encountered in muskeg upon the concrete with which it may be in contact. In recent months, this concern has been indicated by the increasing number of enquiries on the subject which have been directed to the Division of Building Research.

Much of the work that has been done in this field to date has been concerned with the effect of the attack of acidic peaty waters on concrete drain tile; these investigations have been carried out chiefly in the United States. Most of the references to the effects of such aggressive waters on large concrete structures (such as dams) originate in the United Kingdom and particularly in Scotland. Some field observations as well as laboratory investigations have been carried out in Scandinavia, Germany and South Africa. No active research of this type is known to have been carried out in Canada. There is, however, no reason to doubt that the effect of muskeg waters on structures is basically any different in Canada from that in other countries.

2. The Nature of Aggressive Muskeg Waters

A lucid account of the nature of aggressive waters is presented by Lea (8) (9); only a brief resume will be given here. Waters found in areas with a considerable peat accumulation tend to have an acid reaction owing to the presence of carbon dioxide and humic acid arising from the decay of the peat. Humic acid is probably a mixture of acids of fairly high molecular weight and relatively low solubility in water. Some of the simpler organic acids -- such as acetic acids -- may also be produced in small quantities. A saturated solution of humic acid in water has a hydrogen-ion concentration (pH) of 3.6 to 4.1, the strength of this acid lying between that of carbonic and acetic acids. Owing to its low solubility of about 0.9 gm./litre, the quantity of free acid which can be carried in water is small. In practice, however, humic acid -- though having some action -- is much less aggressive to concrete than pure, or carbon dioxide containing water.

Peaty waters, which are practically free from salts, show pH values generally between 4 and 7 (9). Occasional lower values of pH are due to the presence of small amounts of free mineral acids, mainly sulphuric. According to Lea, acidity of peaty waters fluctuates with the seasons and weather conditions and is usually greatest after heavy rain following a warm, dry period. An investigation of some Puget Sound bogs (17), however, indicated that the time at which water samples were collected for analysis appeared to be of no consequence. The dominating factor influencing the degree of acidity in this region was concluded to be the stage of succession of a bog, the acidity being directly proportional to the stage of plant succession.

In general, acid solutions attack cement mortars or concretes by dissolving part of the set cement, thereby progressively weakening the material by removal of the cementing constituents. Waters which are acid owing to the presence of organic acids and carbon dioxide do not normally produce more than a surface attack on mass Portland Cement concretes, although with the more acid waters over a long period of time, the action may penetrate to a depth of several inches. The action will become much more serious if there is a considerable hydrostatic pressure tending to force the water through the concrete mass.

Pure water (as well as soft waters from rain and melted ice or snow) dissolves lime to an extent of about 1.2 gm./litre (9) and is also capable of leaching out lime from set cement compounds. Initially, it reacts with lime to form insoluble calcium carbonate, but on further action the much more soluble calcium bicarbonate (temporary hardness) is formed. The attack of water on Portland cement depends partly on the content of bicarbonate and free carbon dioxide in the water and partly on the carbonation of the cement surface. For a constant content of bicarbonate in the water, the lime removing potentiality increases with the content of free carbon dioxide. When the content of free carbon dioxide remains unchanged, the dissolving power increases as the content of bicarbonates in the water decreases.

To distinguish between "free" and "aggressive" carbon dioxide in water: the "free" carbon dioxide is that present over and above the amount required to form calcium bicarbonate. The "aggressive" carbon dioxide, capable of dissolving more calcium carbonate, is the free carbon dioxide present in excess of that required to maintain the existing bicarbonate equilibrium, less that portion of it which will be required to stabilize the additional calcium carbonate in the solution.

According to Werner and Giertz-Hedstrom (21), the degree of acidity (pH) of the water does not give a simple measure of the aggressive action but bears a relation to it only insofar as it is an expression of its softness and content of aggressive carbon dioxide. In general, however, water may be aggressive at pH values up to 7.0 -- or even 7.5 -- if the temporary hardness (calcium bicarbonate) is below about 0.25 parts per 100,000, and at pH values up to 6.0 or 6.5 for higher values of temporary hardness.

In the investigation of the acidity of the waters of some Puget Sound bogs (17), an attempt was made to correlate colour of the water and its acidity. There appeared to be only a rather indefinite relationship, although the intensity of colour seemed to increase with acidity. Furthermore, the later the stage of plant succession, the more intense was the colour of the water.

An investigation of the reaction and calcium content of drainage water from peat deposits in New York (22) indicated that bog waters are not always acidic. Whether a deposit rests on calcareous or non-calcareous material appears to determine -- to some degree at least -- the relative acidity of the upper layers of the deposit and the reaction of the drainage water from the upper few feet of the soil. Furthermore, if the peat deposits are relatively shallow and underlain with a calcareous material, the drainage water from the soils is likely to contain comparatively large quantities of calcium and consequently is not aggressive.

Radforth (12) states that in many large areas of the North, for example, in the vicinity of Churchill, the peaty member of muskeg is alkaline rather than acidic in reaction. Due to flooding conditions which often occur in some of the muskegs at least once a year, dissolved materials are brought up into the organic material from the mineral soil below, therby charging the peat with agents which are frequently alkaline in reaction. Where plateaus of peat occur, the top portion is often acidic (pH 5 - 7) while the bottom is alkaline (pH 8 - 8.5). Where the terrain is flat and poorly drained, pH of all the peaty material is frequently alkaline (pH 8). Where the terrain is uneven and drainage is better, the reaction may be predominantly acidic (pH 4 - 7).

In the course of carrying out muskeg investigations in Northern Canada in 1955, some eight samples of water from various muskeg classification types were obtained by the writer for the Division of Building Research. These samples were subsequently analyzed to determine those properties which might have an adverse effect on concrete. The analyses were carried out in accordance with the procedure outlined by Rodt (13). The results are shown in Table I. Graphs showing the relationship between hydrogen-ion concentration (pH) of the water and free carbon dioxide, corrosive (or aggressive) carbon dioxide and carbonate carbon dioxide are shown in Figs. 1, 2 and 3 respectively.

There is scant reference to the use of muskeg water for the mixing of concrete. In Germany, the practice of using acidic water for mixing concrete is specifically advised against (16). Lea (9), however, points out that as a rough rule any water that is fit to drink can be used in mixing concrete. This is confirmed by tests carried out by the U. S. Waterways Experiment Station to determine the effects of pH values of water on the strength of concrete (23). The tests indicated that pH does not provide a basis for specifying quality of mixing water.

3. Investigations of Mass Concrete Structures

The papers by Halcrow (5) and by Halcrow, Brook and Preston (6) as well as subsequent discussions (24) are concerned with the effect of peaty waters on concrete dams in Scotland. The object of the papers was to describe a particular instance of the deterioration of concrete surfaces due to contact with moorland waters; to note its effect on the design and construction of hydro-electric installations; and also to give some particulars of a series of tests carried out to ascertain the best materials for resisting such action. The research was carried out at the Kinlochleven works of the British Aluminium Company.

It was found that in the course of time all concrete surfaces exposed to water, whether still or running, were affected. The smooth surface of the concrete became rough; the cement disappeared from the faces of concrete structures exposed to aggressive water, leaving the grains of sand and small aggregates exposed. The depth to which the action took place was small, probably not in excess of one-eighth of an inch; therefore the stability of the structure was not affected. In conduits, however, the deterioration of the surface increased the coefficient of friction thereby reducing the carrying capacities with a consequent decrease in the maximum possible output from the power station.

Analyses of the water and laboratory tests showed that the acids in the peaty moorland water dissolved the lime and calcium aluminate in the cement and somewhat rapidly removed the face of the concrete, the action becoming much slower as the area of cement exposed was lessened by the presence of sand grains. In an effort to determine the best cement or other material to be used to resurface the conduit when the necessity should arise, and also with a view to dealing with new construction, experiments were carried out on the resistance of various cements and protective coatings to this corrosive action. Some 113 possible materials were tested; they consisted of varying mixtures of portland, slag and aluminous cement concretes, tars, bitumens, bricks, renderings and chemical surface treatment.

In the testing programme, a small conduit 4 ft. wide by 4 ft. deep was utilized. Internal linings 4 1/2 ins. thick as well as loose concrete slabs 2 ft. by 2 ft. by 4 ins. were exposed to the water. The linings and slabs were formed of the various concretes to be tested. The majority of the tests showed definite deterioration at the end of one year and only a few were unaffected after a period of several years; of these, aluminous cements were outstandingly good. Some of the materials under test resisted well for 8 or 9 years, then began to fall off. With most of the surface treatments tested, there was doubtless a protective agent present and in many cases this concrete was in a better state of preservation after 4 or 5 years than ordinary Portland cement concrete. After this period, however, the effect of treatment disappears and more rapid deterioration sets in. It is pointed out that with a life of only 4 years, the surface treatment methods discussed are not suitable for application to the lining of an aqueduct or to the face of a dam; it is apparent that something more permanent is required.

In the subsequent discussions to the above investigations (24) it was suggested that it might be more economical to treat the water with silicate of soda or hydrate of lime, thereby neutralizing the acidic effect. It was pointed out, however, that in large hydro-electric schemes which involve great quantities of water, such a solution to the problem is not altogether feasible.

A thorough investigation was made of the Blackwater dam of the Kinlochleven works some 25 years after it was built (5). Wherever seepage through the dam (a mass concrete structure) had occurred, a deposit of lime in the form of $CaCO_3$ had accumulated on the downstream face of the dam; this deposit varied in thickness from a mere wash of lime to a crust 3 ins. thick. Apart from these lime encrustations, the downstream face of the dam was found to be in good condition. Core samples of the concrete were taken and analyzed for cement content. Leakage water was also analyzed. It was estimated that some 52 1/2 tons of free lime were lost (washed out) in 25 years, equivalent to a loss of 85 tons of cement out of the forty thousand tons used in construction of the dam. It was pointed out that the carrying away of lime in cement by percolating water is a very slow action, but which may result ultimately in complete disintegration of the cement and concrete. The need was stressed for making dams completely watertight to withstand the aggressive effect of acidic and soft moorland waters.

Hellstrom (7) reports on investigations of, and construction work on, mass concrete and masonry structures, chiefly in Scandinavia. He deals with the reasons for deterioration of concrete and discusses the solubility in water of different types of cements. He also considers the practical side of the repair work to dams which have been subjected to aggressive waters. From his investigations he concludes that it is evident that water which is acidic in reaction will dissolve the lime when percolating through concrete, and also that it is evident that water already containing lime and carbonates will affect concrete to a lesser degree.

Walsh (20) refers to the complete destruction by peaty water, in less than 5 years, of masses of concrete that were not subjected to unilateral pressure but rather which were frequently exposed to wave action with an amplitude of 2 to 4 ft. Impermeable concrete in the same exposure was merely etched to a depth of about 1/8 in. whereas any permeable concrete was rapidly attacked and destroyed. The pH value of the water was only 6.5. He stresses the vital importance of impermeability of a hydraulic concrete structure.

4. Investigations of Concrete Drain Tile

Thin-walled concrete units, such as concrete pipe, require rather special consideration with regard to their resistance to aggressive agencies. A degree of attack which would be relatively unimportant in a mass concrete structure may be sufficient to destroy a pipe with a wall thickness of only 1 or 2 ins.

One of the earlier investigators was Elliot (3) who carried out rather extensive laboratory and field tests on the effect of peat and peaty water on concrete tile. Some of the more interesting conclusions of his investigations were:-

(1) Peat increases in acidity with degree of decomposition.

(2) A high percentage of lime, even the presence of marl, is no guarantee against corrosion.

(3) An acid subsoil aids in the disintegration of tile.

(4) The more porous the tile, the more rapid the disintegration.

(5) The presence of water is necessary for disintegration to take place.

(6) It is not necessary for the tile to be actually in the peat for disintegration to take place, but merely that the peaty waters have free access to it.

(7) Most of the tile then available commercially was much too porous to withstand the effects of acidic waters.

Miller and Manson (10) (11) report on tests made on some 1100 commercial and experimental drain tile and on 10,000 2 in. by 4 in. experimental cylinders. Installation of experimental specimens were made in three peats in Minnesota, three peats in Wisconsin and in an acidic mineral soil in North Carolina. For purposes of comparison, specimens were also installed in one mineral soil in Minnesota and one in Wisconsin. Field and laboratory studies extended over a period of 24 years, from 1923 to 1947. Significant results of this most comprehensive investigation were as follows:-

(1) Practically all concrete and mortar specimens examined showed evidence of corrosive action where exposure periods in peat had been about 20 years.

(2) Behaviour of the experimental tile was essentially the same as that of the commercial tile. The loss in supporting strengths of all tile after an average period of 15 1/2 years in peat was slightly more than 400 pounds per lineal foot.

(3) The strength trends of the experimental cylinders of plain concrete and mortars were very similar to those of the drain tile. The strength loss of the concrete cylinders was 18% of the strength at 5 years, whereas the mortar cylinders lost 45% of their strength at 5 years.

(4) In general, the degree of corrosion varied with the acidity of the peat and the unit strength of the concrete in the test specimen (whether drain tile or cylinder) when installed. Acidities of the three mineral soils in which cylinders were installed did not furnish as reliable an indicator of corrosive action as they did for the peats.

(5) Poor quality concrete tile did not give satisfactory service in even low acid grass and sedge peats with a pH of 6.0 or higher.

(6) None of the 17 Portland cements used differed in resistance to soil acids. The concretes made with high alumina cements, however, were slightly more resistant than those made with Portland cements.

(7) None of the various types of admixtures used in the tests improved the resistance of the concrete to the action of soil acids.

(8) Rich mixes of high strength and low permeability gave the greatest resistance to the action of soil acids.

(9) None of the various special materials and procedures employed improved the resistance of high strength concrete, with the possible exception of steam curing at 345°F or coating the surface with linseed oil or with a cut-back bituminous product.

Werner and Giertz-Hedstrom have reported on tests on concrete pipes carried out in Sweden (21). These pipes, of 4 ins. internal diameter and manufactured by various methods, were continuously dipped in waters of varying chemical composition. Each specimen was immersed 2 million times per year. Aggressive action on the pipe was checked from time to time by weighing. On the whole, the aggressive action of the water was found to increase with increasing degrees of acidity (decreasing pH).

5. <u>General Field and Laboratory Studies</u>

One of the earliest studies of the effect of acidic waters on concrete was that of Dykerhoff (2), who carried out tests on concrete made with four kinds of sand and who showed that all concrete is more or less attacked by marsh water. He found that sea sands and marsh sands gave the poorest results and also that the greater the amount of silica in the cement, the better it resisted corrosive attack.

Fraser (4) discusses a search for cheaper alternatives to the usual physical (such as painting of surfaces) and chemical (such as use of high alumina and waterproof cements) methods of protecting concrete from peaty waters, since both these measures are expensive. The problem was approached from both the physical and chemical standpoints in tests carried out at the Gaur Project in Northern Scotland. At this particular location, the pH of the run-off is sometimes as low as 4.0.

Three types of concrete were tested:-

(1) Ordinary Portland cement concrete.

(2) Ordinary Portland cement concrete with an agent to entrain 3.0 to 3.5 per cent air for improved physical resistance.

(3) Ordinary Portland cement concrete with a minute quantity of water softener for improved chemical resistance.

Both additives were combined with the mixing water. Thirty cubes of 6 in. sides were made of each type of concrete, cured and airdried then exposed to aggressive waters. Cubes of each type were removed at various intervals and tested. Although the tests were of a relatively short duration, it was found that concrete types 2 and 3 were superior to type 1 in resistance to peaty water attack, and also that type 2 concrete behaved more consistently than did type 3. It was recommended that a more detailed investigation be made of chemical treatment of ordinary cement and it was also suggested that as an immediate practical measure all concretes exposed to acid water attack should be of the air-entrained variety.

Tremper (18) presents the results of an extensive investigation into the corrosive effect of acid waters on concrete laboratory specimens. This investigation shows that the setting and hardening of Portland cement does not result in compounds of lime that are absolutely insoluble in water, and also that the solubility of lime increases with the acidity of the water. Tests are described of intact concrete specimens exposed to a flow of artificially acidified water, the pH value of which was comparable to that of natural waters. A few specimens were exposed in a small creek which was naturally slightly acid. Although the water was made to flow past the surface of the concrete and was not forced through it, it was concluded that the results would probably apply in a general way to concrete through which water is percolating.

Results are presented of tests of about 3000 specimens ranging from 1 1/2 in. by 1 1/2 in by 4 in. prisms to 8 in. by 16 in. cylinders. Data were obtained on the loss of lime and the relative loss in strength resulting from acid exposure. The information thus obtained was used to estimate the probable life of concrete structures under various conditions of acid exposure. Significant conclusions drawn from this investigation were:-

(1) For the range of acidity expressed by pH values between 6.0 and 7.0 the degree of attack was inversely proportional to the pH number; if the pH of the water was above 7.0 the attack in general was too mild to be of any practical consequence.

(2) The volume of water and its velocity must be very low before a decrease in the rate of attack is observed.

(3) Waters of high velocity, especially if they carry suspended matter, may accelerate the rate of attack.

(4) Exposure to a naturally acid creek gave results comparable to those obtained with artificially acidified water in the laboratory.

(5) Of the four types of Portland cement studied, none were particularly superior in resistance; concretes made of some special cements were attacked to a lesser degree than those made of Portland cement.

(6) The character of the aggregate probably had little effect on resistance; if there is a difference between aggregates it could not be connected with results of usual inspection or tests.

(7) Allowing the concrete to become dry before exposure, or curing for 48 hours in water vapour at any temperature between 70 and 212 ^{o}F., had no effect on resistance.

(8) Although admixtures were found which increased the resistance of concrete somewhat, none afforded complete protection.

(9) Surface coatings in general reduced the rate of attack for two years; the performance of such protection depended upon the durability of the coating or the frequency of its renewal.

(10) Concrete of high quality was markedly more resistant than poor concrete.

(11) The strength of concrete at any stage of attack was dependent upon the ratio of the mixing water to the amount of lime remaining in the concrete; the relationship may be expressed by a curve similar to the cement-water ratio curve.

(12) Loss of lime caused a decrease in strength of concrete; when about 50% of the original lime has been removed from average concrete, complete loss of strength and of coherence results.

(13) The logarithm of the quantity of lime removed varied directly with the logarithm of the period of exposure.

(14) Apart from a severe attack of about 1/16 in. at the surface, loss in lime with accompanying loss in strength was uniform throughout the body of the

concrete for a depth of at least 15 ins. from the surface.

(15) Due to a high surface-volume ratio, small members such as 3 in. or 4 in. drain tile may have a very short life in acid soils.

(16) Standard culvert pipe, 12 ins. or more in diameter, or other structures having an equal or smaller surface-volume ratio, if made of reasonably good concrete, may be expected to have a life running into the centuries, under all probable conditions of natural acidity in water.

In a recent South African contribution to concrete corrosion studies, van Aardt (19) conducted some laboratory experiments aimed at assessing the resistance of Portland cement mortars to aggressive agents. It was concluded from these studies that whenever concrete is to be subjected to an aggressive environment, a well designed plastic mix with a low water-cement ratio should be used and care taken to obtain full compaction in order to ensure low permeability. Fairly rich mixes are shown to be more resistant to corrosion than lean ones, although the mix should not be too rich because trouble will then be experienced from crazing and cracking which will increase the permeability. Pervious concrete is not only subject to corrosion itself but also will allow corrosion of the reinforcing steel to take place.

It was pointed out that Portland cement products are very often inadequately cured and many failures are due to this cause. Green concrete is much more vulnerable to aggressive agents than is well matured concrete. In some instances it may be necessary to provide special curing; it was recommended that in any case moist conditions be maintained for a sufficient length of time after placing if the concrete is to attain its maximum strength and resistance to an aggressive environment.

Corrosion of Metals

The aggressive action of acidic muskeg waters on metals is in itself a study of considerable scope and magnitude. Scott (14) presents a review of the literature on mechanism and evaluation of corrosion in soils. It is pointed out that -- as in the case of concrete -- the presence of water in a soil is absolutely necessary if corrosion is to take place; this requirement is well taken care of in muskeg. The corrosive character of a soil depends upon the presence of dissolved materials and bacteria in the soil water. The low pH of some muskeg waters, therefore, would indicate a potential source of corrosion to metals, particularly to the ferrous metals.

7. Conclusions

The research that has been carried out in various parts of the world on the effect of peaty waters on concrete structures permits a number of general conclusions to be drawn which are applicable to the muskeg areas of Canada:-

(1) Muskeg waters are usually - but not always -- acidic and are therefore potentially aggressive to concrete and to metals.

(2) The hydrogen-ion concentration (pH) alone is not necessarily the only criterion for aggressiveness of muskeg waters. Of equal importance is the amount of dissolved salts in the water. Soft waters can be highly aggressive to concrete, even if the acidity is low. The analyses of the muskeg waters obtained in Northern Canada (Table I) indicate an increase in the acidity as the surface cover changes from tall woody growth to low non-woody growth. On the other hand, corrosive CO_2 appears to be highest for muskeg types with tall woody cover and least for muskeg types characterized by low non-woody cover. Nevertheless, a low pH value would appear to be some indication of potential aggressive action against concrete and appropriate precautions would be advisable.

(3) The colour of muskeg water is not necessarily an indication of the degree of its aggressive effect.

(4) Running water is more dangerous than stagnant water since harmful substances are constantly renewed.

(5) Attack at a constantly changing water level is usually stronger than attack on parts of the structure constantly under water.

(6) Fresh concrete is more susceptible to attack than is old, hardened concrete. Consequently, precast concrete is more resistant to corrosive action than is cast-in-place concrete.

(7) If concrete has ample strength to fulfill the purpose for which it was designed, it does not automatically follow that it will resist attack and possible disintegration in an aggressive environment. Mechanical strength is not necessarily related to resistance to attack.

(8) For any given cement, the less permeable the concrete produced, the greater will be the resistance to attack. It cannot be too strongly emphasized that concrete subjected to an aggressive environment should be of high quality and dense. The best cure for aggressive action is prevention.

(9) High alumina cements, as well as air-entraining agents introduced into the concrete, have produced concretes which are more resistant to attack than ordinary Portland cement concrete.

(10) Most surface treatments have a life considerably less than the design life of the structure and generally have not been too successful over a long period of time.

(11) Thin-walled concrete structures, such as drain tile, are much more susceptible to aggressive action than are mass concrete structures.

(12) In general, muskeg waters can be used for the mixing of concrete.

(13) Most of the investigations reported herein were carried out some years ago when concrete technology was not as advanced as it is now. With the use of high quality materials currently available and utilizing good workmanship to produce a reasonably impermeable concrete, together with any extra precautions which circumstances may dictate, there is no reason to expect that a concrete structure will not be highly resistant to any aggressive action of muskeg waters.

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TABLE I

Analysis of Bog Waters

Sample No.	M1-1W	M2-1W	M3-1W	M4-3W	M4-4W	M4-5W	M5-1W	M5-3W
Location	Cran. Port., Man.	Flin Flon, Man.	Lynn Lake, Man.	Bird, Man.	Bird, Man.	Bird, Man.	Knob Lake, P.Q.	Knob Lake, P.Q.
Muskeg Type*	BEI	AEI	CDI	DFB	CI	BI	FEI	FI
pH	6.5	6.5	5.1	6.4	7.2	5.2	5.0	5.5
Free CO_2 (ppm)	7	10	11	7	14	8	4	2
Corrosive CO_2 (ppm)	19	20	23	19	12	23	15	12
Carbonate CO_2 (ppm)	21	22	15	10	55	nil	4	2
Free H_2SO_4	---	nil	---	nil	---	nil	---	nil
Free Organic Acids	---	nil	---	nil	---	nil	---	nil

*Radforth, N. W. A Suggested Classification of Muskeg for the Engineer. The Engineering Journal, 35:11. Nov. 1952

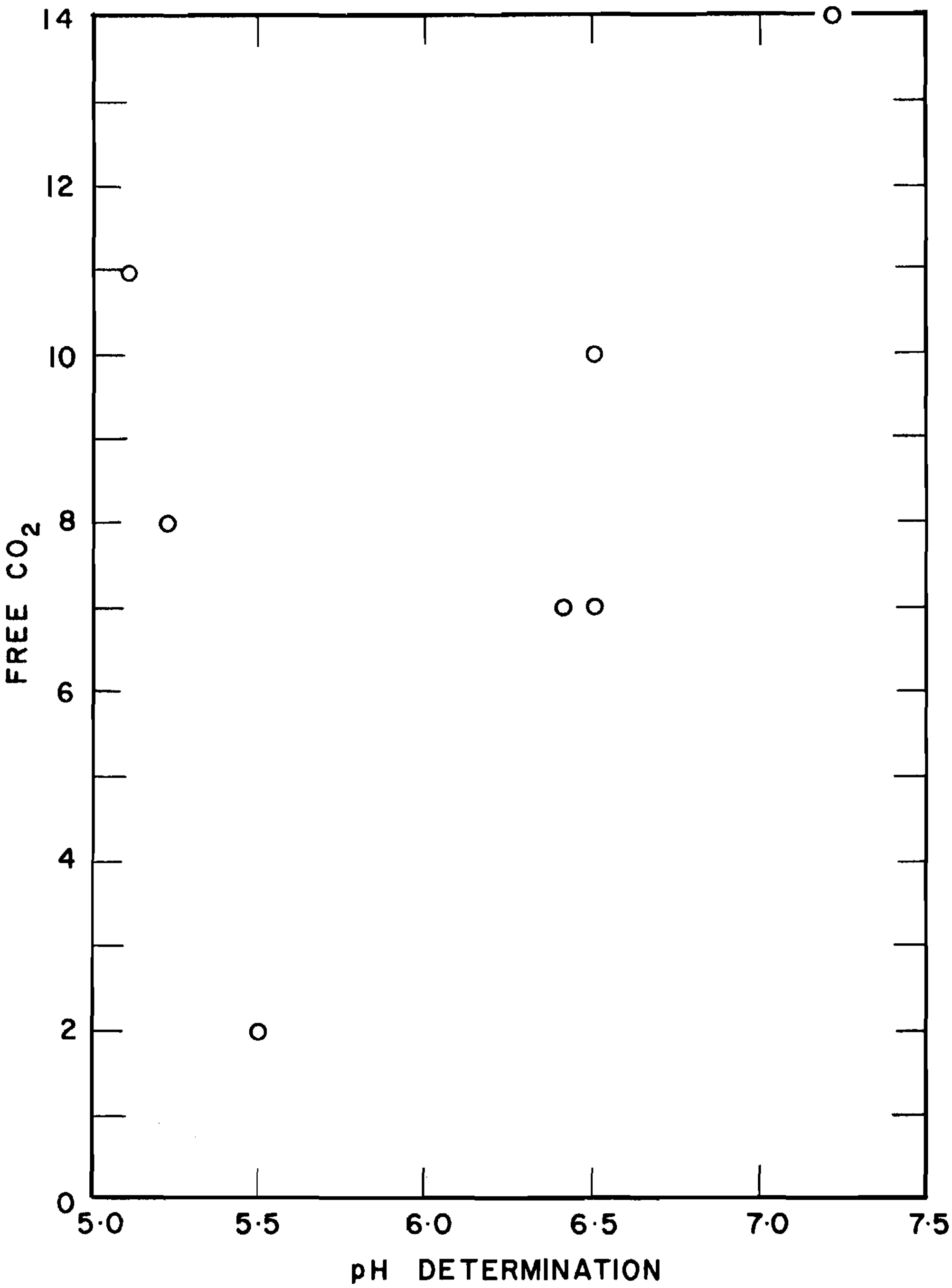

FIGURE I

GRAPH OF FREE CO_2 VS. pH

BR 1669-1

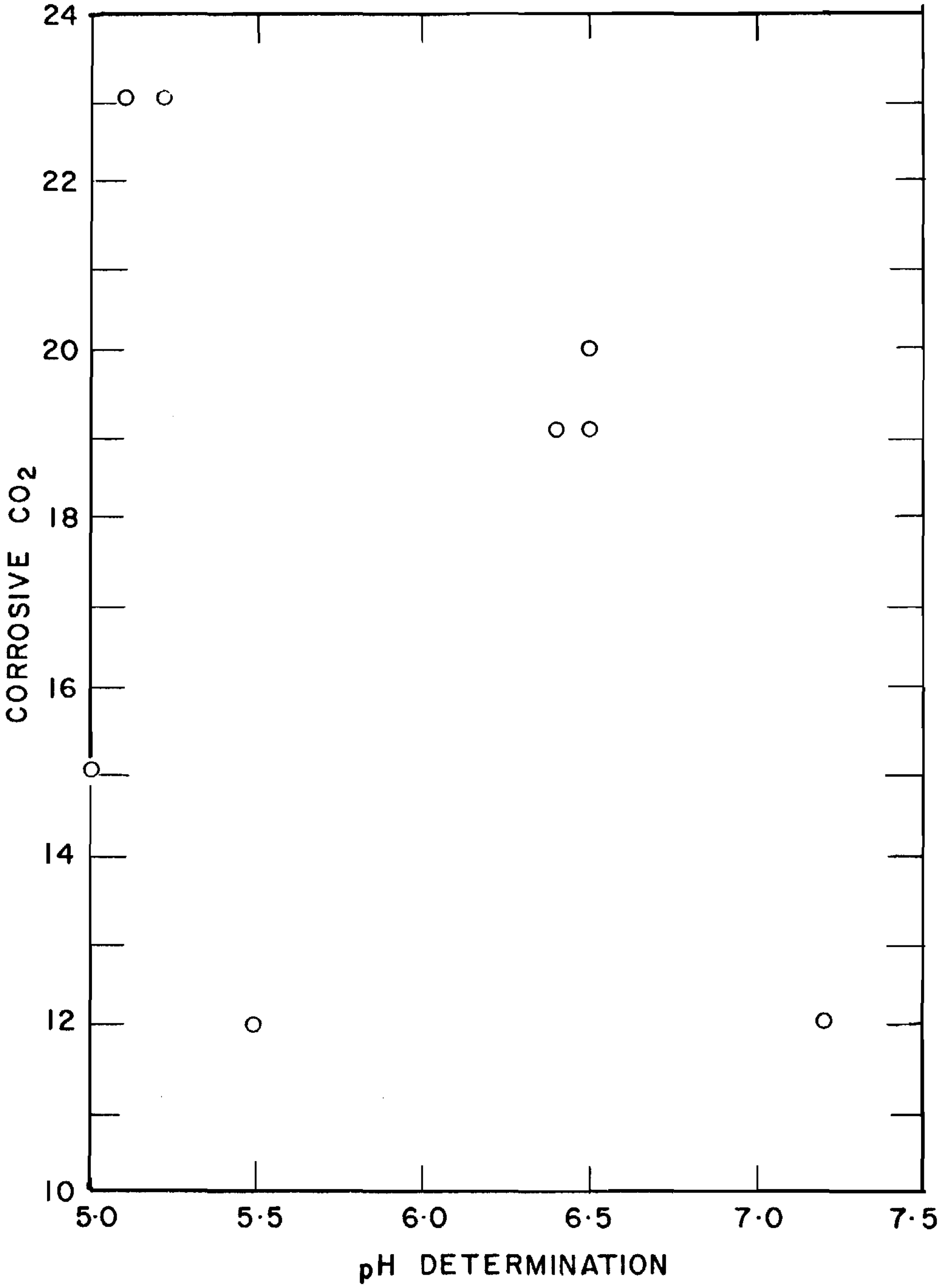

FIGURE 2

GRAPH OF CORROSIVE CO_2 VS. pH

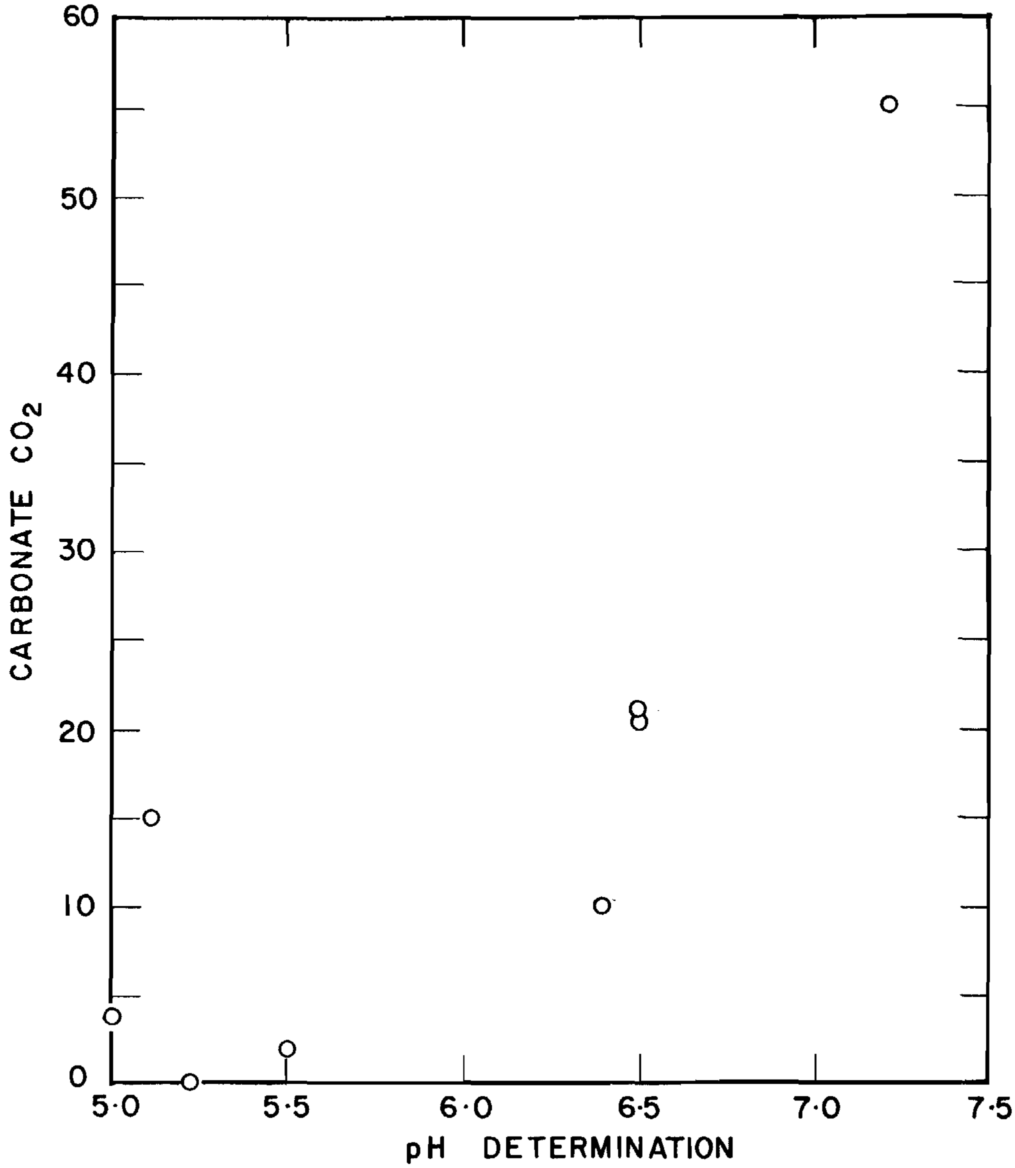

FIGURE 3

GRAPH OF CARBONATE CO_2 VS. pH

Discussion

Mr. Rutka (Ontario Department of Highways) asked if there were any particular case records in Canada of chemical deterioration of concrete by bog waters. Mr. Adams (HEPCO) also wondered if Mr. MacFarlane knew of any cases in Canada of corrosion of metals by bog waters. In reply, Mr. MacFarlane stated that there is practically nothing in print with regard to Canadian case records of concrete and metal deterioration from bog waters. Mr. Brawner (B.C. Department of Highways) commented that there are known cases in the Vancouver area where houses built in peaty areas have suffered a 3/4 in. deterioration of the concrete basement walls over a period of twenty years. Professor Irwin (Ontario Agricultural College) pointed out that concrete drain tile in peat bogs is no longer the problem it used to be, due to the high quality of concrete, and they do not hesitate to recommend concrete drain tile for this type of soil. In commenting on the use of muskeg waters for mixing concrete, Mr. Hemstock (Imperial Oil) observed that this water is used in drilling cement and is found to be quite satisfactory. However, they have run into considerable corrosion of steel pipe lines from some muskeg waters, particularly at the junction of organic and mineral soils. This is quite a serious problem in some areas.

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II. 1 ORGANIC TERRAIN RECLAMATION IN NEWFOUNDLAND

J. V. Healy

The need for organic soil reclamation in provinces other than Newfoundland is not yet immediate but undoubtedly will become so within the foreseeable future. Population expansion and other factors are now beginning to, or will shortly, affect the acreage of mineral soils available for agricultural use. The problem of reclaiming these organic soils in Newfoundland is much more urgent. Factors creating this situation are:-

1. The total acreage of suitable mineral soils available is small and usually occurs in pockets too small and isolated to permit large-scale economic farming practices. All mineral soils with the exception of some very small alluvial deposits are of recent glacial origin and consequently are extremely stony. The Wisconsin glacier covered the island until approximately 7500 years ago and, as a result of the low average temperature, little topsoil formation has taken place since. The time of glacial break-up was established by taking core samples from the deepest sections of several bogs and submitting them to carbon-14 tests. It seems reasonable to assume that the typical bog vegetative types with their low living requirements would be the first to establish following glaciation.

2. Freight rates from mainland ports to Newfoundland are fantastically high, resulting in an abnormal retail price. These rates almost prohibit the import of winter feed for cattle; hence, the Newfoundland farmer who has not enough acreage to produce his own hay or silage also cannot import it. At the moment, approximately 90 per cent of agricultural produce consumed in Newfoundland comes from other provinces or seasonally from the United States. The population of Newfoundland is nearing the half-million mark. Taking the Dominion Bureau of Statistics figure (80.2 lbs) for domestic disappearance of beef and veal (1955), it is seen that there is an annual import of approximately 30 million pounds of beef and veal. Very much the same applies for several other foodstuffs. Imports of this order are a constant drain on the economy of the province.

3. Any large expansion of farming on mineral soils would have to be at the expense of forest since all mineral soil areas are densely wooded. The wisdom of clearing timber is obviously doubtful in a province where pulp and paper production looms so large in the economy.

The Newfoundland Royal Commission on Agriculture (1955) dealt in detail with these problems and suggested, inter alia, that the possibilities of organic soil reclamation should be explored. On the instructions of the Hon. W. J. Keough, Minister for Mines, Agriculture and Resources, the experiment was started in 1956. The directive required initially that the following information be obtained:-

1. Determine the suitability of Newfoundland organic deposits for grass and hay crops.

2. Determine the average unit cost of reclamation.

As in all organic soil operations, whether transportation, peat fuel production or agricultural reclamation, the procurement of specialized machines is imperative. It was realized initially that European machines would not, in most cases, suit our proposed operations as most reclamation schemes in Europe have been preceded by peat fuel or peat moss operations. The economics are quite obvious of first removing 90 per cent of the deposit for peat fuel (average calorific value: 9000 BTU/lb - dry basis) or for readily saleable peat moss (German exports to U.S. (1955) $4,000,000), then reclaiming the remainder for agricultural purposes. It is regrettable that total utilization cannot as yet be employed in Newfoundland. The rapidly expanding peat moss market, however, may soon change the situation. Thus, European reclamations not only inherit a complex drainage system which would be far too expensive to install for agricultural use alone, but also the reclamation machines work on a maximum organic material depth of 3 ft. and so avoid most flotation problems. Since peat fuel or peat moss schemes are invariably of large areas, advantage of this is taken to use reclamation units of as large a size as possible. None of these desirable situations existed in Newfoundland. Equipment had to be considered from an entirely different viewpoint, obtained from what was commercially available and, in most cases, altered to suit organic soil conditions.

The factors controlling choice of machines in Newfoundland are as follows:

1. Areas of approximately 50 acres would have to be reclaimed for individual farmers, thus necessitating frequent machine movement. Hence, all machines would have to be light and compact enough to be transported by orthodox means without even partial dismantling.

2. The majority of Newfoundland bogs are not well humified and humification is the deciding factor for suitability or otherwise of any type of drainage equipment.

3. Even had suitable European units existed, their purchase would not have been considered good policy due to the inevitable service difficulties.

In agricultural reclamation of organic soils the following sequence of operations is generally employed. Any single operation may be performed in different cases by different types of machines but rarely is an operation omitted:

1. Survey.
2. Drainage.
3. Correction of acidity.
4. Seed-bed preparation.
5. Fertilization and seeding.
6. Rolling.

Survey includes the establishment of top and bottom contours and a physical examination of the upper two foot layer. While it is true that crops can be grown on any bog ranging from raw to completely decomposed (H-1 to H-10 von Post scale)* for reasons of economy in producing a good seed-bed, etc., a bog in the H-5 to H-7 bracket is desirable. Top and bottom contours are fixed on a grid which will range from 100 to 200 ft. side as cost and ultimate use

*See Appendix I

dictate. Bogs intended to produce high priced vegetable crops naturally can afford a more expensive drainage plan. The establishment of the bottom contours is far more important in laying out the drainage network than fixing the top contours. This arises from the continual shrinkage which begins immediately after drainage. It is clear that a section of the bog area with a depth of 20 ft. will have a much greater absolute shrinkage over the years than a section with a depth of 5 ft. If this vital factor is ignored in laying out the drainage plan, it is quite likely that accumulated shrinkage will reverse the drain gradients with disastrous results to the project.

Fig. 1 shows what might be regarded as a typical drainage network under Newfoundland conditions. The small ponds and gravel or rock intrusions are a very common feature. It should be noted that the field drains do not form a continuous pattern but are sited to make use of the important bottom contours. As these various groups of drains discharge into several separate collectors, the expense of excavating large collector or outfall drains is avoided. Initially, the drains were installed at a centre-to-centre distance of 50 ft. and were approximately 2 ft. in depth. Later tabulation of ground water table levels has shown that this distance could be increased to 75 ft. centres with no significant increase in g.w.t. level.

Initially, a plough-type ditcher pulled by a large tractor was bought as a drainage unit. It was realized before purchase that at best its general suitability would be low but no other machine was commercially available. As expected, its performance was extremely erratic. On well-humified bog, it was reasonably satisfactory but on bogs of low humification the fibre mass invariably choked the plough. This condition is shown in Fig. 3. Even when producing a reasonably good ditch, the cost of breaking up and spreading the plough soil makes the operation expensive in total. Fig. 4 shows the spoil being deposited by the plough. It is essential to remove the spoil as its concentrated weight inhibits drainage (see Fig. 2).

To overcome the difficulties encountered with the plough, recourse was made to the rotary disc type ditcher. European models of this type are available but since they are of relatively enormous size, they were unacceptable for this reason, quite apart from the very high cost involved. Accordingly, the small compact machine and associated towing tractors (shown in Fig. 5) were designed and built locally. The ditcher has performed very adequately and is the subject of a detailed report which will be issued shortly. Figs. 6 and 7 show the disc ditcher in action. It is seen that the spoil is finely broken up and well scattered over the surface area.

All Newfoundland bogs are acidic (pH values 3.5 to 4) and this condition is corrected by applying two tons per acre of finely ground limestone (70 per cent passing 100 mesh sieve). A standard lime and fertilizer spreader is used for this purpose. Extreme care must be taken to ensure that no area misses receiving its proper quantity of lime as vertical or horizontal movement is very small in the early years of reclamation.

To admix the lime thoroughly through the top six inches of the organic soil, a power-driven rotary cultivator is used. Several of these machines are on the market and on organic soil can usually obtain the required depth in one pass. Besides mixing in the lime, the rotary cultivator promotes a very rapid drying out of the top layer by increasing the exposed surface area. Also, shearing of the fibre mass at the 6 in. level prevents a large amount of capillary action. All this obviously permits a rapid warming up of the seed bed layer.

With regard to grass seed and fertilizer treatments, no final conclusions have yet been arrived at and experimental work is continuing under the Federal Department of Agriculture. Adequate response has been obtained from 500 lbs. per acre of one of the commercial fertilizer mixes such as 6-12-2 or 9-9-7. For organic soils, it is essential that trace elements be added to fertilizers. While 500 lbs. has yielded satisfactory crops, amounts as high as 1500 lbs/acre have been tried and indications are that even at that level the point of diminishing returns had not been reached.

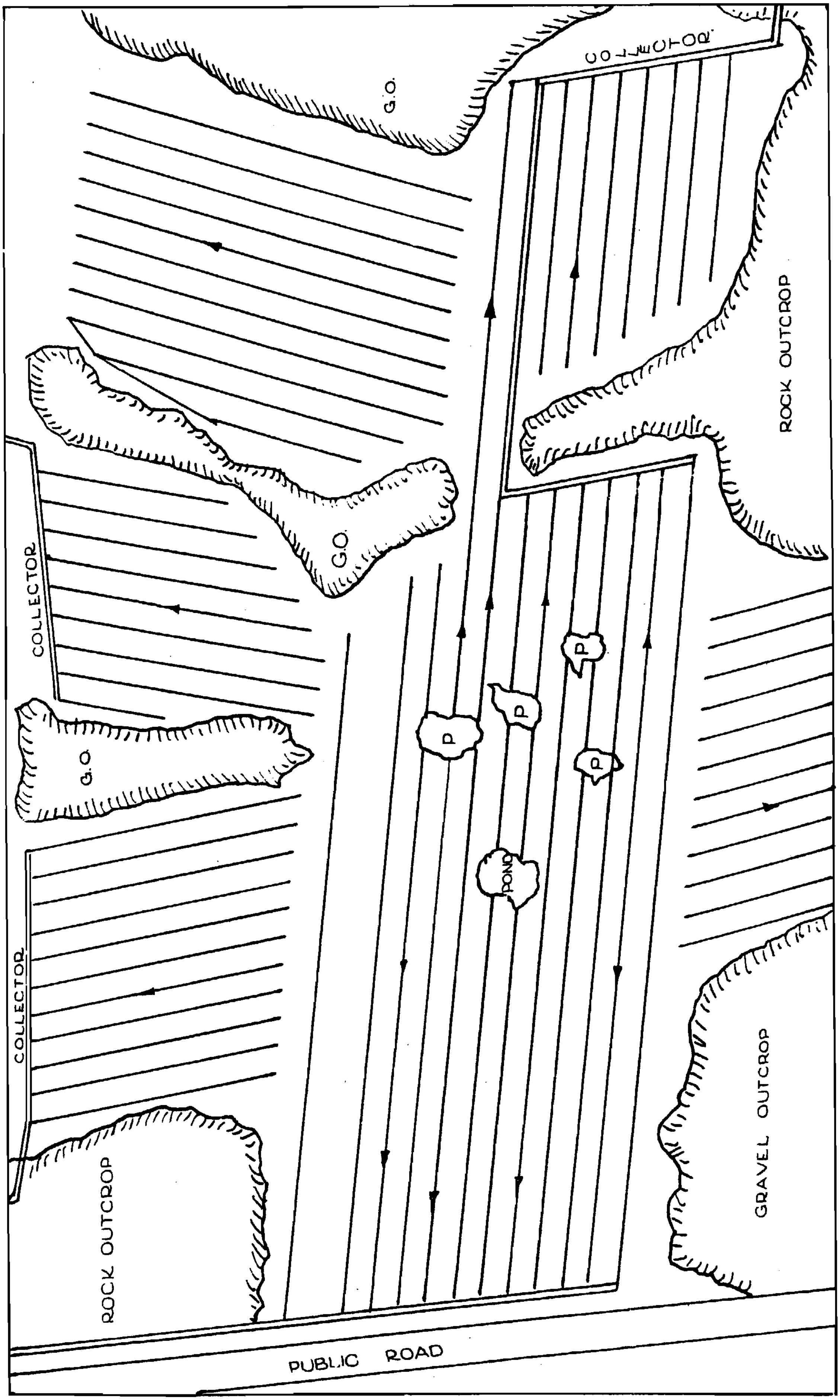

FIGURE I
TYPICAL DRAINAGE NETWORK

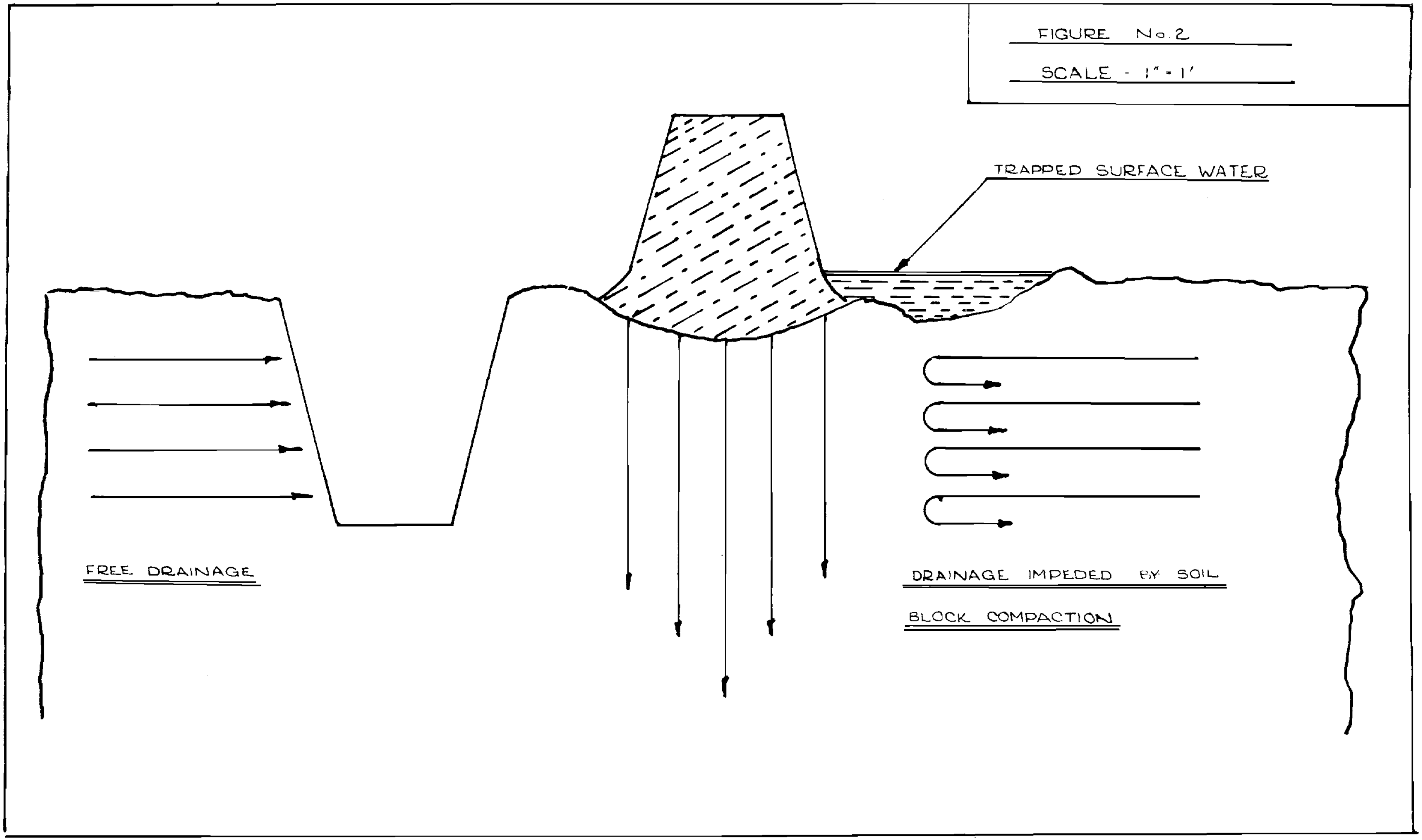

FIGURE 2
DRAINAGE INHIBITED BY SPOIL BANKS

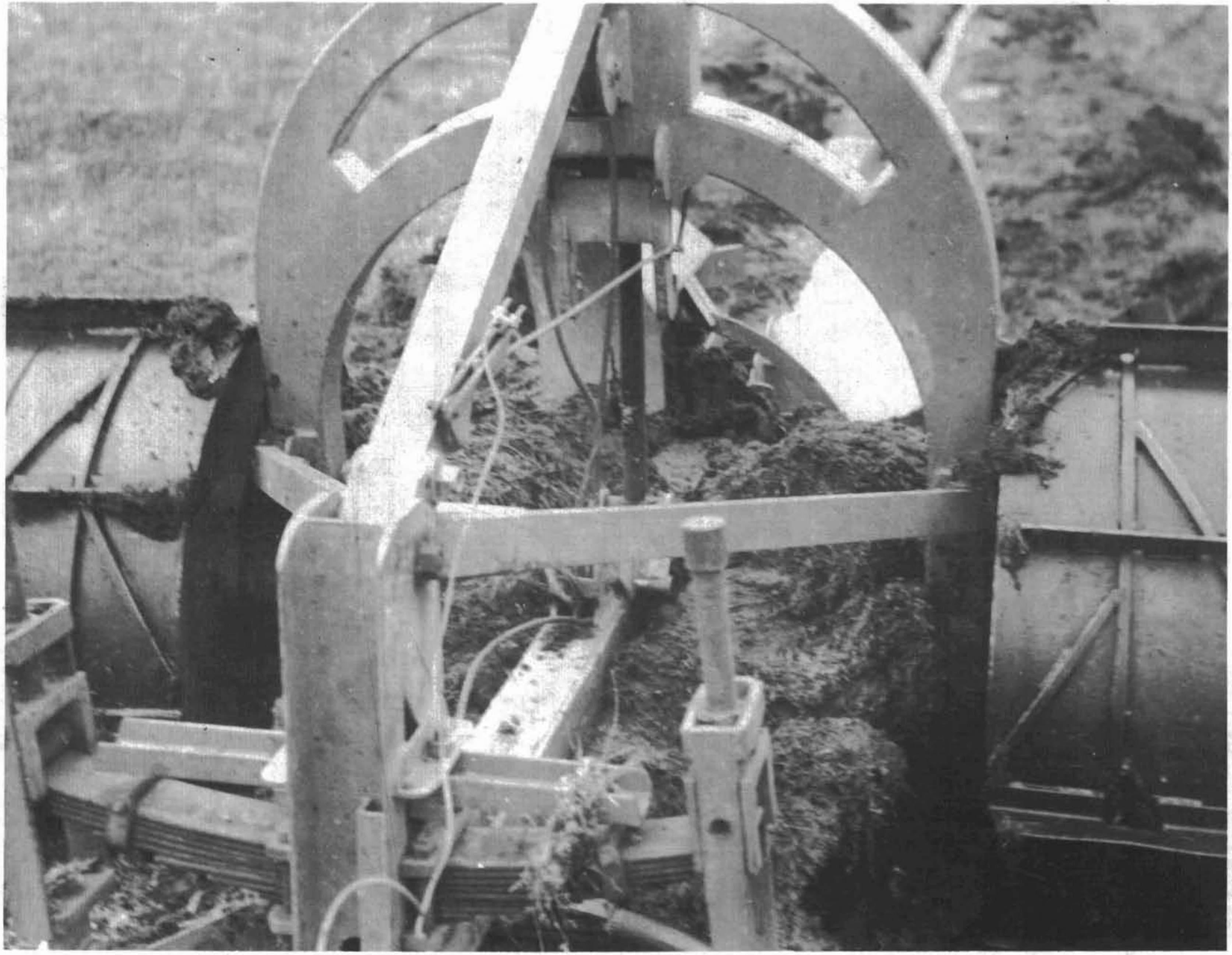

FIGURE 3
PEAT CLOGGING PLOUGH-TYPE DITCHER

FIGURE 4
SPOIL DEPOSITED BY PLOUGH-TYPE DITCHER

FIGURE 5
ROTARY DITCHER AND TRACTOR

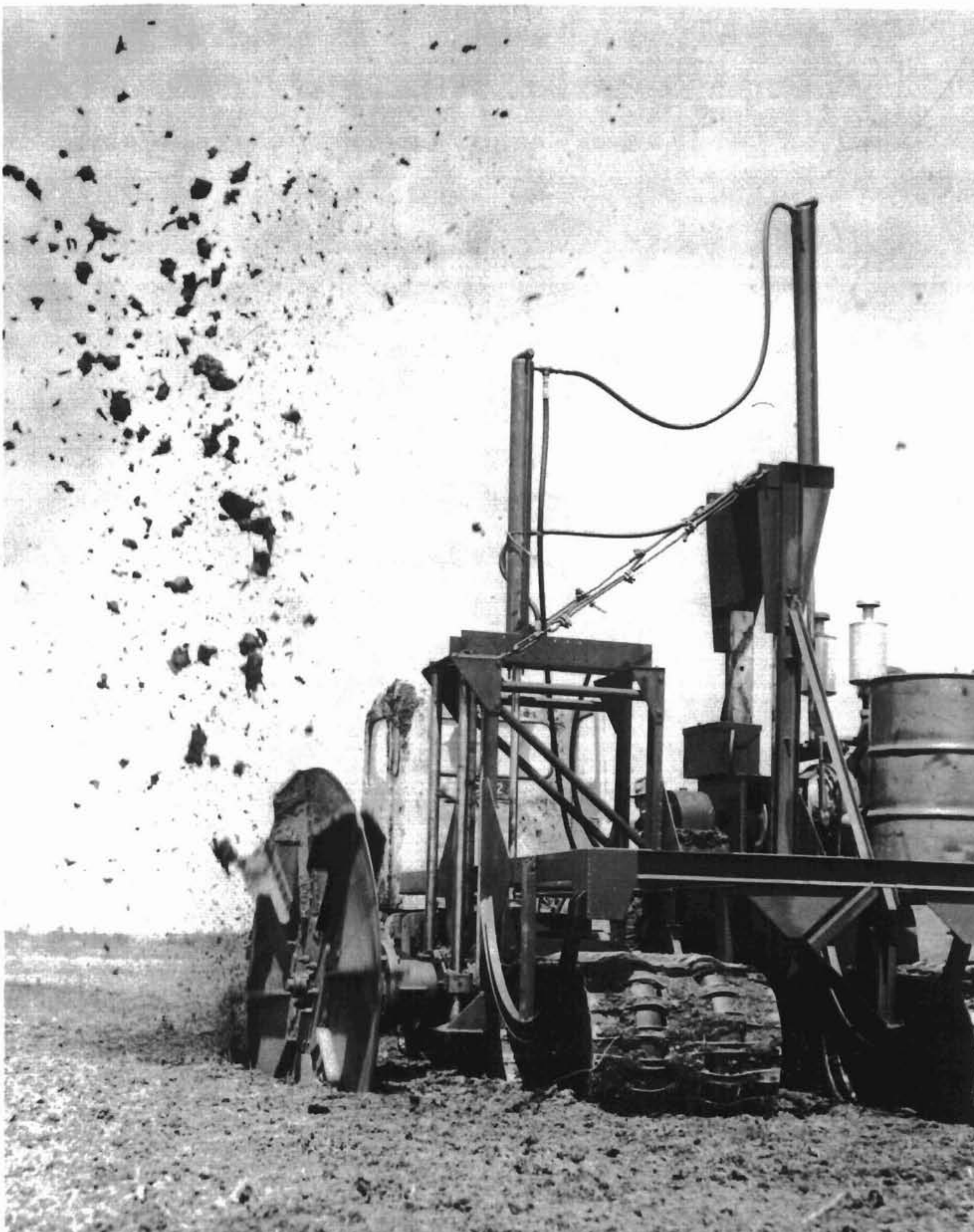

FIGURE 6
SPOIL DEPOSITED BY ROTARY DITCHER

FIGURE 7
DRAINAGE DITCH EXCAVATED BY ROTARY DITCHER

APPENDIX "I"

LENNANT VON POST HUMIFICATION SCALE

H 1: Completely unconverted and mud-free peat which when pressed in the hand only gives off clear water.

H 2: Practically completely unconverted and mud-free peat which when pressed in the hand gives off almost clear, colourless water.

H 3: Little converted or very slightly muddy peat which when pressed in the hand gives off marked muddy water but no peat substance passes through the fingers. The press residue is not thick.

H 4: Slightly converted or somewhat muddy peat which when pressed in the hand gives off marked muddy water. The press residue is somewhat thick.

H 5: Fairly converted or rather muddy peat. Growth structure quite evident but somewhat obliterated. When pressed some peat substance passes through the fingers but mostly very muddy water. The press residue is very thick.

H 6: Fairly converted or rather muddy peat with indistinct growth structure. When pressed, at most 1/3 of the peat substance passes through the fingers. The remainder extremely thick, but with more obvious growth structure than in the case of unpressed peat.

H 7: Fairly well converted or marked muddy peat but the growth structure can be seen however. When pressed, about half the peat substance passes through the fingers. If water is also given off this has the nature of porridge.

H 8: Well converted or very muddy peat with very indistinct growth structure. When pressed, about 2/3 of the peat substance passes through the fingers and at times a somewhat porridge-like liquid. The remainder consists mainly of more resistant fibres and roots.

H 9: Practically completely converted or almost mud-like peat in which almost no growth structure is evident. Almost all the peat substance passes through the fingers as a homogeneous porridge when pressed.

H 10: Completely converted or absolutely muddy peat where no growth structure can be seen. The entire peat substance passes through the fingers when pressed.

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Discussion

Mr. Davis (Spruce Falls Power and Paper Company) asked about the unit cost of muskeg reclamation. Mr. Healy replied that it was $80.00 per acre, including fertilizer and seeding. In answer to a question regarding the type of peat in the reclaimed areas, Mr. Healy said that it was largely non-woody. Mr. Rowe (Department of Forestry) wondered if there were any frost or ice problems. Mr. Healy replied in the negative, pointing out that the average temperature in Newfoundland is higher than that on the mainland. In Newfoundland, there is heavy snowfall from November onwards. The snow is blown off the bog surface and collects in the ditches. Consequently, the surface of the bog freezes but the ditches do not, so that a flow of water continues all winter.

Mr. Brawner (B. C. Department of Highways) enquired about the rate of reclamation, to which Mr. Healy replied that in the past year -- the first year of commercial production -- it was eight hundred acres. Dr. Leahey (Department of Agriculture) remarked that in Newfoundland they do not have to worry about drainage, due to the rolling topography. For muskegs on the mainland, one of the chief problems is getting an outlet for drainage ditches. Also, Newfoundland gets much more rainfall than does the mainland. One great problem in reclamation is water control, as it is easy to overdrain. With its high precipitation, this is no problem in Newfoundland. Mr. Keene (Connecticut State Highway Department) suggested that shrinkage of a peat bog results from subsidence of the underlying peat due to drying of the top layer, which is buoyant peat no longer and consequently the effective weight is increased.

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II.2 ROAD CONSTRUCTION FOR FORESTRY PRACTICE IN NORTHERN ONTARIO

A. E. Davis

In Northern Ontario, there are road construction problems which, although troublesome, are not entirely insurmountable. At present, the only pressing problem is not how or where to build the roads but rather the lack of roads. Specifically, the Spruce Falls Power and Paper Company, Limited, has about 200 miles of permanent or semi-permanent winter roads, and about 215 miles of gravel surfaced roads. Of the latter, 15% are built over muskeg.

The geomorphology of the Company's timber limit is small humps of heavy lacustrine clays surrounded by extensive areas of muskeg. Very widely scattered throughout are displaced remnants of eskers, generally running in a north-south direction. Eskers are the main source of gravel deposits. For the construction and maintenance of the entire mileage of gravel roads there are only twelve workable gravel pits within the limit. Muskeg constitutes 70% of the total land area and has peat depths ranging from 1 ft. to 10 ft. The peat generally overlays heavy silt and gumbo clays. The vegetal cover of the muskegs fall mainly in the ADI - AEI - DFI and BEI classes. Peat structures are in categories 10 to 14. Variations of the common types are frequent. (See Appendix I).

From a forestry point of view the Company's roads are chiefly used as:

1. Access routes for logging operations, forest protection, planting, etc.

2. Haul routes for transporting the harvested pulpwood crop.

Roads, suitable for rubber-tired traffic, are divided into two main divisions; the first called "winter" and the second "gravelled surfaced". Within these two broad divisions there are several classes based upon the intended use of the road.

I. Winter Roads

Winter roads take advantage of the frost. In muskeg areas the roadbed can be prepared in one of two fashions. First, the right-of-way can be hand cut, with the large stumps grubbed out or dynamited down to the level of the surrounding ground. This practice is fast disappearing and giving way to the second method which is used at present for preparing new muskeg winter roads. The merchantable timber is first removed. When the cold weather comes, the right-of-way (generally cut 66 ft. wide) is tramped with small tractors or the large-type snowmobile. When there has been sufficient frost penetration into the peat, large bulldozers (generally in the 16 or 26 ton class) equipped with shearing blades, bulldoze a swath about 50 ft. wide down the middle of the right-of-way. Shearing is merely the action of cutting or shearing off the stumps, unmerchantable trees and debris at ground level. This leaves an excellent root mat for future use. The secret of shearing is adequate "tramping down" of the snow to ensure sufficient frost penetration into the peat. The stumps and trees, if not frozen firmly in the peat, tend to tear or lift out rather than shear.

Shearing is usually done the winter before the road is required. When it becomes very cold the following winter, the snow covering the sheared muskeg is packed down with snowmobiles and later with tractors of the 10-ton class. When the snow has been sufficiently packed and the frost has penetrated the peat, a large "V" plow pulled by 3 or 4 10-ton tractors is drawn over the road. The packed snow, the slightly frozen peat, and the supporting action of the root mat permit relatively trouble-free passing of the plow and tractors.

After exposure to sub-zero temperatures for two or three nights, the road is "tanked". Successive layers of water (hauled by tandem trucks equipped with piggy-back tanks) are poured on until a smooth, strong road surface is obtained. On the average, the ice thickness is about 2 ins. to 3 ins. Such a road easily handles 50-75 ton loads. This type of muskeg road costs about $600 per mile, exclusive of shearing.

It is obvious that a winter road over muskeg has limitations. The most serious, of course, are its temporary nature and yearly preparation cost. On some of the Company's main winter roads, in an effort to prolong their seasonal use and to eliminate the yearly preparation cost, the Company has gone one step further after shearing. Wherever and whenever practical, a shallow lift of fill (usually clay) is laid down over the sheared muskeg road. This fill is about 1 ft. in depth and laid down 30-40 ft. wide. If the sheared section of road is less than 400 ft. long and has good borrow pit possibilities on each end, then the cushion is made by a straight tractor push. Sections over 400 ft. long with clay fill available are done with tractor-drawn scrapers. On these longer sections, if good clay fill is not readily available, it has been found necessary to truck in fill. This is usually of a granular nature. Costs vary from 60¢ per lineal foot for tractor fill to $1.00 per lineal foot for truck fill.

Where it is necessary to protect these shallow cushions from Spring washout, cross drainage in the form of corrugated steel culverts is provided. To date, these cushions have been put on about 3 miles of muskeg road, the longest section being almost a mile in length.

Some advantages of the winter road are:-

1. This type of road is primarily used for the transportation of the harvested wood crops.

2. While relatively inexpensive to make, it is dependent solely on cold weather.

3. Its load-carrying capacity is very high.

4. Specifications are simple with regard to construction, drainage and surfacing.

5. Except for snow plowing, maintenance is negligible.

6. Roads of this class can be built across any type of muskeg.

II. Gravel Surfaced Roads

In this division there are four main classes of roads:-

1. Haul Road.

2. Class I.

3. Class II.

4. Camp Service Road.

Each class of road is designed for a different type of traffic. As its name suggests, the haul road is built to accommodate a pulpwood haul. It is designed to be used by wide heavily loaded semi-trailers. To date, there has been a very limited need for this type of road as most of the pulpwood haul is done during the winter months and generally on winter roads.

Class I and Class II are access roads for logging operations, forest protection, planting, etc., Class I being built for heavier traffic.

Camp Service Roads, the lowest class of gravel road, are used primarily in connection with cutting camps. From new camps it is expected to cut 500,000 or more cords over a period of up to twenty years. This means that a much larger area than before must be cut from any one camp. To do this it has been found necessary to transport men up to 7 miles from the camp to cutting areas. Some of this can be done during the winter months on winter roads. However, for summer cutting, a network of secondary gravel-surfaced roads is essential.

Surveys are a prime requisite for all classes of roads. The higher the class the more intensive and informative the survey required. Generally speaking, the gravel roads are laid out to take advantage of the displaced remnants of eskers. Try lines are first plotted photogrammetrically and later field checked for suitability. Muskegs are avoided whenever possible, especially those with the BEI and FI vegetal cover types. However, with 70% of the land area of the limit consisting of muskeg, this is practically impossible. When muskeg areas are unavoidable, the shortest route possible across them is chosen. There are limitations to this though, as there has been considerable difficulty experienced in constructing roads over muskegs with peat depths exceeding 6 ft. If encountered, these sections require specialized treatment.

The Company's usual method of crossing muskegs with a gravel surfaced road is by "floating" the fill. As the first step in construction, a brush mat or brush corduroy is laid down across the muskeg section. This mat is used to support or "float" the fill on top of the organic terrain and is made by cutting the trees on the right-of-way and carefully hand placing them at right angles to the axis of the road. This is a specialized job. Great care must be taken that full-length sections of coniferous trees are used and that for the sake of ease of handling or the desire to salvage too much pulpwood the cutters do not use short, unsatisfactory pieces of trees and brush.

The mats, depending upon the road class, vary in width from 18 ft. for camp service roads to 36 ft. for haul roads and cost about 20¢ to 30¢ per lineal foot. Wherever possible, a mat is built to be 18-30 ins. thick when compressed. A general rule is the deeper the peat the thicker the mat. Care is taken that the brush mat is not cut too soon before the fill is placed. A fresh, green mat is much more desirable for fill support than is a dry defoliated one.

Although normally the fill is "floated" over the muskeg, there are two other methods of crossing short stretches of muskeg used by the Company. They are "fill by displacement" and "complete excavation and backfill". The fill by displacement method is used where there is a decided lack of brush or standing timber to make a mat and the cost of hauling in brush is prohibitive.

In the summer of 1955 it was found necessary to cross a muskeg with a DFI type of vegetal cover. It was approximately 800 ft. in length with peat depths averaging 4 ft. Rather than fill by displacement it was decided to truck in some coniferous trees to make the mat. A standard 5-ton truck was used. It had been so dry that summer that it was possible to drive the truck on an old winter road paralleling the right-of-way. This old winter road had no brush or root mat at all - just bare peat - yet it supported a loaded 5-ton truck throughout its entire length.

Fill by displacement has never been attempted in any muskeg of a depth of peat greater than 5 ft, at least not intentionally. An embarrassing incident developed four years ago. A short stretch of muskeg, about 500 ft. long had 6 ft. depth of peat, according to the survey profile. A brush mat was laid down and end filling operations began in a normal fashion. The bulldozers started building a causeway about 3 ft. in depth out over the mat, and by the end of the first day had about 200 ft. of mat covered. Upon returning to work the following morning, the construction crew was greeted by a strange sight. The fill was level with the muskeg ground elevation, the ends of the brush mat, not covered by fill, were tilted upward at approximately 45° and on each side of the road were two large mounds in the ground. The explanation was very simple. Due to fading in the survey profile print the peat depths appearing to be 6 ft. were in reality 6 ft. plus. Upon resounding it was discovered that the fill was suspended over 12-15 ft. of peat. One week later the subgrade over this ADE cover type was considered finished. Because of an inadequate mat it had been necessary to continue fill operations until the fill (about 10 ft. of it) had reached a state of equilibrium. In this particular instance there was about 2 ft. of clay showing above the general ground level.

The third method of crossing muskegs is by complete excavation and backfill. Although draglines on mats have been used to do a limited amount of peat excavation during the summer months, the most successful results have been obtained during the late winter months using a large bulldozer of the 26-ton class. The peat at this time of year is quite dry and peat depths of 4-6 ft. have been successfully excavated by this method. The bulldozer works at right angles to the road axis and humps the peat up on the edges of the right-of-way. The peat is later levelled for aesthetic purposes. Excavation by this method costs about 22¢ per cubic yard. Backfill, depending upon the availability of fill, is 20-30¢ per cubic yard. This method is employed mainly on the haul road class where there is a strong possibility of heavy summer pulpwood hauling.

Placement of fill in muskeg areas has been done for all three methods by straight tractor pushing, tractor drawn scrapers, and by trucked-in fill. In the brush mat or "floating" method, draglines have also been used to cover the mat. The method used depends upon existing physiographic and economic conditions.

So much for gravel road classes and methods of traversing muskeg areas with a gravel surfaced road. For any class and any particular method there are certain recurring problems which must be faced in a muskeg crossing.

The first and foremost is drainage. Due to the very nature of the area, offtake ditches can be a very costly affair. To have a 2 1/2 in. fall-off in 100 ft. is considered fortunate indeed. It is most advisable not to stint on the length of the offtake. If the peat depths are constant and there is a relatively even fall-off in the land, dynamiting by propagation does the job very well. If, however, there are uneven peat depths and it is necessary to cut through the rim of the muskeg for adequate drainage, then the only solution is a dragline. In putting in a dragline offtake, the proposed line is surveyed with advantage taken of any natural topographical features. The final line is cut 16 ft. wide and the fallen trees cut into 12-16 ft. lengths and laid crossways on the offtake line. This permits the dragline (usually in the 3/4 yd. class) to walk down to the end of the offtake. The dragline then ditches to a predetermined grade out to the culvert position, casting the fallen trees, peat, brush and mineral soil, if any, to the side.

In muskeg sections of road it is advisable to have a 4 ft. differential in elevation between the crown of the subgrade and the water table line. This is done preferably by lowering the water table in the muskeg with graded ditches at least 1 ft. in depth, and laying down on the mat at least 2 ft. of compacted fill.

With longitudinal drainage as described and with offtake drainage adequate, the muskeg sections on Company roads exhibit the following noticeable features: -

1. There has been no displacement or settling where an adequate mat was made.

2. Gravel surfacing lasts longer than on a comparable highland section of road.

3. There is a complete absence of frost boils in the Spring.

4. Wash boarding and pot holes are in much less evidence on muskeg sections than on highland sections.

The second problem encountered in a muskeg area is culvert installation. If the offtake permits and the peat depth is not excessive, an effort is made to excavate the peat (by dynamite or dragline) and the culvert set on mineral soil. However, if it is necessary to set the culvert in peat, there are two possible solutions. First, an excavation can be made in the peat to a predetermined elevation. (That is, the final culvert elevation). Log corduroy covered with coniferous brush is then placed on the bottom to provide a bed on which to assemble and place the culvert. When ready, backfill operations are started. As the fill falls into the excavation, labourers hand tamp and pack the fill under and around the culvert. When the culvert is set, the remaining fill is pushed on with extreme care by the machines.

The second method is very similar to the fill by displacement method. The culvert site is cleared of brush, large stumps and debris and then backfilled. When the fill stabilizes - that is it ceases to settle - a new excavation is made in this fill and the culvert placed in position. Backfill by machinery, as before, is done with caution. In all culvert installations an inverse camber of about 9 ins. per 44 ft. culvert is used.

A third problem in organic terrain crossings is one of "communication". Although an effort is being made to "carry a finished road", that is, be completely finished as close as possible to the construction unit, there are times when certain pieces of machinery become isolated. To fuel and serve these, rubber tracked vehicles with bearing pressures of 1.5 to 3.0 p.s.i., have been used. Nevertheless, isolated machines can develop into quite a problem. It has been found that bulldozers of the 16 and 26-ton class can cross muskegs for fuel and servicing. A safe rule of thumb is "one machine will pass in one place one time". Walking the bulldozers over muskeg, of course, does not apply to those which have FI and EI cover classes.

In connection with the policy of "carrying a finished road", this Spring saw the completion of a rather interesting experiment. On the road programme this year, there is a muskeg of the ADI class to cross. It is exactly 5,280 ft. long. It is also 4 1/2 miles away from the start-off point this summer. To tackle this muskeg during the summer months would have been a mammoth undertaking. Backfilling of this muskeg by truck fill was completed early in April. Having the muskeg covered when the road construction reaches there this summer will more than offset the cost of bulldozing the access trail 4 1/2 miles, the cost of cutting the brush mat now by commuting, and the cost of taking all the equipment in to do the job. Backfilling by truck, even in isolated areas, becomes feasible and economical when one can take advantage of the frost. There was approximately 12,000 cubic yards laid down over the mat for a cost of 86¢ per cubic yard or about $1.90 per lineal foot.

Costs for gravel roads over organic terrain vary directly as to class of road and length and depth of muskeg. The variance is great but experience has shown that a completed road will cost anywhere from $10,000-$20,000 per mile. Improvements in technique, engineering skill and taking full advantage of the weather can help only so much. Unless someone can find some hitherto unknown solution for consolidating peat into a hard usable material, gravel roads over muskeg will continue to be expensive.

APPENDIX I

The letters used to describe the vegetal cover and the numbers used to describe the peat structures have been taken from "The Radforth Muskeg Classification System".

There are sixteen numbers used to describe the categories of peat structure. The lower the category number, the finer the texture of the peat. Spruce Falls Power & Paper Company's peats are generally of a medium to coarse type and are therefore described as being in categories 10 to 14.

The vegetal cover classes are described by letters or combinations of two or three (but never four) letters.

The classification does not refer to the species of plants but rather to qualities of vegetation such as stature, degree of woodiness, external texture, etc.

For the Company's purpose, the following symbols are explained:-

AEI - ADI	- Spruce Flat	(SF)
ADE	- Spruce Swamp	(Swp)
BEI	- Stag	()
DFI	- Alders	()
FI	- Marsh or grass	()

****

Discussion

Mr. Brawner (B. C. Department of Highways) in commenting on the slides illustrating the talk, wondered if it was advantageous to have ditches so close to the road. Mr. Davis replied that the slides represent former practices; ditches are now placed 50 ft. from the centerline, at the tree line, rather than close to the road. Prof. Anderson (University of Alberta) commented on the case of the road failure with 3 ft. of fill above grade. He wondered if there was any other information on height of fills for rapid failure conditions. Mr. Davis said that this was the only failure they had every experienced. The true depth of the peat had not been correctly determined, and the mat laid down was inadequate. They never put down more than 3 ft. of fill on the mat. Three ft. of bulldozer fill compacts to 2-2.5 ft. The failure described happened five years ago; since that time this area has presented no further problems.

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II. 3 THE ORGANIC TERRAIN FACTOR AND ITS INTERPRETATION

L. Keeling

The exploration and development of Canada's natural resources recognizes no topographical boundaries and the presence of such a large proportion of organic terrain over our northland is providing an interesting challenge to the nation's industries.

The intensity of competitive exploration is now demanding a complete exploitation of muskeg from accessibility considerations. Consequently, at present there is a very high interest in any studies related to this subject.

Few projects to be undertaken in Northern Canada in the future will be completed without encountering muskeg, and the successful completion of any of these projects will not be achieved by concentration of any one aspect of trafficability over muskeg. In order to obtain maximum exploitation, it is necessary to co-ordinate developments in vehicle design with the precision now possible in muskeg identification and interpretation, and also with modern organizing abilities.

The design of tracked vehicles is now sufficiently advanced to meet most requirements, although they may well be in insufficient numbers to satisfy all projects.

This paper concentrates on the exploitation of the muskeg medium itself, with particular emphasis on the use of air-photo interpretation as an aid in the selection of routes in organic terrain.

Integration of Effort

To ensure the maximum effort so necessary for a completely successful operation, the initial planning groups, in proposing the projects must be well aware, in a general way, of the extent to which muskeg is involved. Senior planners should be familiar with what can and cannot be done with reasonable economy in organic terrain.

Few companies have projects in which only one department of their organization is involved; most have at least two or three. The senior personnel of each of these operating departments should be kept informed on the latest developments in the subject of muskeg and its exploitation.

The conditions prevailing in muskeg areas are as varied as the weather itself and this is an apt comparison, for the meteorological state has a very great effect on the full utilization of muskeg.

Off-the-road track vehicles have been built to certain design specifications and will perform satisfactorily only as long as the demand made upon them does not at any time exceed these specifications. The design of track vehicles has resulted in a very versatile array of machines but their limitations should be well studied.

It is the recognition of the prevailing conditions in the terrain, and the application of the design characteristics of the vehicles, which are the keys to the problem.

Every summer there is a larger contingent of track vehicles in the bush, with each group building up a valuable store of experience. The aim should be the application of this new experience to the problem of extending the "organic terrain season" to a year-round period, so that, in the future, work in muskeg becomes commonplace.

To be eagerly awaiting the first frosts of approaching winter, or relying on the early summer periods when winter ice is still present in the peat, is certainly not exploiting muskeg to the full.

The demand for track units of all kinds will increase as time goes on, with consequently greater need for their economic use. The majority of the vehicles in use today are owned by operating companies, but off-the-road hauling is already becoming the specialty of transportation companies offering long or short-term rental contracts or even hourly rates.

What the Project Involves

A carefully laid plan of operation in muskeg will consider the following important points:-

(a) When is the project to be carried out? Is it a one-time short duration move of a few miles; is it a series of moves at predetermined intervals throughout the year; or is it a season-long project such as a continuously profiling seismograph operation?

(b) What is the nature of the job? Does it require large load carriers; can it be done with light mobile units; or does it demand a combination of different types of vehicles?

(c) It is important to realize that a plan drawn up to meet the demands of one project cannot necessarily be re-used, since the identical circumstances may never recur.

Planning the Route

The actual examination of the land forms should follow a logical line so that maximum use can be made of up-to-date methods of muskeg interpretation.

In addition to the accumulation of details on the previously listed considerations, those making preliminary investigations should bear in mind the following factors:-

(a) the surface and subsurface conditions of the different types of terrain;

(b) the limitations of the terrain.

The Role of the Photogrammetrist

Preliminary studies of the land in the area of the assignment may be started in the office. Vertical air photographs will be used both for the detailed study of stereographic pairs and for the mosaic work sheet and prompt availability of these will speed the job.

The photogrammetrist working these projects must possess an intimate knowledge of muskegs. He must also have at his disposal a method of identifying, classifying and recording graphically the various terrain types that he recognizes on the photographs, or which he can indicate to be present from the physiographic growth patterns apparent on the photographs. Identification and interpretation of muskeg must be well developed to produce the wealth of information that is discernible.

Classification System

It is of no consequence which classification system is used provided that it is a true one, and provided that the characteristics of the types can be predicted under a wide range of conditions. It must, of course, be a simple one for there will be a large number of people engaged. This is not to imply that everyone working on a project should be a muskeg expert, but the greater the band of people who possess a ready referencing system when talking about organic terrain, the sooner the problems in muskeg will be reduced and conquered.

The group with which the author is associated uses the Radforth Classification for referencing the muskeg types and topographic features. It has been found to be both accurate and easy to apply with a minimum of confusion.

Interpretation

The realization has grown that, for all its complexities, muskeg is an organized mass. There are definite relationships between the physiographic character, environmental conditions and the cover classes so easily identified and classified. This relationship also exists between the surface appearance and the subsurface conditions. Thus, through examination of the physiographic aspects represented on the air photographs, the cover types and the surface conditions can be determined, even though many of the types and conditions are not directly discernible on the photographs. Useful interpretations of this form are now being made with a fairly high rate of success.

The most satisfactory base on which to work is a mosaic of the area, constructed from the best possible set of photographs, at a scale of about two inches to one mile. On this the pertinent land forms are outlined - streams to be crossed, hills, existing trails or roads, and any large areas of mineral terrain - as they occur between the terminal points of the project. Fig. 1 illustrates well the terrain restrictions that must be considered.

Adding to this same mosaic, and avoiding the severe terrain, an experienced interpreter will then lay out one or more preliminary routes. This will not take very long and will form the initial framework on which a more detailed examination of stereo pairs of photographs can be made. It was felt that if full use is to be made of the air-photo coverage in existence over the large areas involved, we should concentrate first on the refining of interpretation from the high altitude photographs.

A thorough study of all the published literature on this subject, and a close association with the five high altitude air-form patterns of Marbloid, Stipploid, Terrazzoid, Reticuloid and Dermatoid, has shown that the physiographic pattern approach will next lead to the identification of the majority of the actual cover classes and topographical features in the muskeg direct from the photos. Since there has not been much opportunity to study photographs taken from the 5000 ft. altitude, the many lower air-form patterns have not entered into common use. For those who have access to them they can form a step in interpretation and no doubt would afford additional differentiation.

It is felt that it is better, however, to get a sympathetic feeling for the environment of the area under analysis, the physiographic atmosphere so to speak, and to develop a familiarity with the natural relationships rather than slavishly relying on the photographs to reveal all the details.

Condition Forecasting

A good interpreter will know that in the various muskeg forms or cover class formulae there are varying degrees of the following conditions and situations, and that the severity will vary with the physiographical growth patterns:-

undisturbed mat cohesion to be expected;
probable vehicle speed in M. P. H.;
vehicle subsidence in summer;
depth of the receding seasonal ice on, say, June 15th;
vehicle pitching to be expected;
relative traction expected;
deterioration rate or number of passes;
route deviation expected due to vehicle impedence;
relative severity of subsurface ice contours;
likelihood of a mineral sub-layer;
availability of timber for corduroying;
relative drainage.

Marbloid

Refer to Fig. 2 which depicts well the surface characteristics of the Marbloid air-form pattern. The peat plateaus (i) with the cover classes HE, BHE, or HEB, and the network of well-developed drainages filled with I or IE, would indicate a number of interesting conditions.

The predominantly H cover classes on top of the plateaus serve as an insulation to check the seasonal ice recession in the approaching summer months. There may well be no ice left in the drainages on June 30th, but frost would surely be found 6" below the surface of the HE plateaus at that time.

The plateaus will provide a good firm footing with relatively little need for route deviation, but there may be severe pitching, particularly when vehicles are crossing the I and IE drainages. The traversing of these small mossy streams and the climbing up onto the height of each frozen plateau will prove tricky, particularly for short vehicles, for traction is considered to be low in HE, and especially so when the ice is near the surface.

Deterioration of the mat will be moderate, but vehicle impedence will be very low if detouring becomes necessary, and it will be possible to maintain a relatively good rate of progress of, say, 3-4 miles per hour.

A good example of the topographic feature, free polygon (o), so common in Marbloid, is also seen on Fig. 2. Another higher view of this air-form pattern is shown on Fig. 3.

The Use of Existing Cutlines

A very common practice in summer operations is the use by tractor crews of existing bulldozed cutlines. A bulldozer in clearing a road for winter travel will, without exception, force off the trail every fibre of organic material that is not frozen to the solid earth. In doing so the important upper mat of entwined twigs, mosses, grasses, trees and shrubs are lost to those who choose to travel that way in the summer.

In an attempt to provide a smooth winter trail this same bulldozer will pack snow and organic debris into the small drainages that traverse muskegs. This, coupled with the practice of windrowing two packed ridges of frozen snow and organic debris along each side of the trail, results in an upsetting of the natural drainage, and therefore creates added impedence to the vehicles passing that way in the summer.

In early summer the presence of frost on these trails may be more of a hindrance than an asset. Since all the melting takes place from the top down, the melting water lies trapped until sufficient vertical thawing takes place to provide internal drainage through seepage. The wave motion set up by the moving vehicle tends to lift the friable peaty material and wash it aside. After the vehicle has passed by, the water rushes back into the depression dislodging still more material. At this stage the action of the moving water has the greater deteriorating effect on the peat layer. Shortly afterwards the floating mass of peat emulsion becomes more viscous, the impedence to the vehicle increases resulting in greater stress on the tracks, which then causes rapid and final destruction of the remaining mat.

Deterioration of this type can be expected to be most rapid in AFI and AEI. The practice of passing through the higher cover classes such as AFI by way of old bulldozed lines should be seriously reconsidered. Very real danger of machine damage is often to be found in these situations.

Reticuloid

In the high altitude air-form pattern Reticuloid (see Fig. 4) there is a combination of cover classes FI, DFI, or EFI, with ridges, mounds, and open or closed ponds with abrupt lake margins as the typical topographic features. These need only a brief explanation. Ice recession in the FI ponding will be rapid and deterioration due to entrapped water will follow. The ponding is ringed by ridges a foot or so high where the ice recession is slower. Machines meeting these features will experience severe pitching and, unless designed with good track approach angles, may well have difficulty in accomplishing a crossing.

Travelling this way would be rugged, and one can only cringe at the thought of crossing over this terrain in wintertime with wheeled vehicles. Yet there is evidence that this is sometimes done.

A muskeg phenomenon which is very similar from the point of view of vehicle trafficability is the ice wedge polygon condition illustrated on Fig. 5. Here the cover classes are HE, EH, or EI, with clefts forming the featured drainage pattern. The clefts would be quickly thawed out in early summer and longer track units would be best suited to routes chosen across them. This oblique view of ice wedge polygons is representative of the features as they occur near Fort Providence in the N. W. T. and may be influenced by permafrost.

Severe vehicle pitching producing considerable strain on loaded vehicles and a real rugged trip for the drivers would result from crossing the irregular peat plateau (j) shown on Fig. 6. Ice knolling conditions would be most prevalent beneath the surface of the plateau in the April to July period, with no great amount of relief expected for the remainder of the summer. With such an abundance of more negotiable types around, this feature should be avoided. Ice knolling would be almost non-existent in the DFI, and along the common FI corridor little or no vehicle impedence can be forecast.

Practical Interpretation Problems

Practically all the data shown on the illustrations can be interpreted directly from the high altitude air photos. That which cannot be observed under stereo examination can be inferred by knowing the environmental condition relationships.

Before going out into the field on any job, it is a good idea to first spend time making office interpretations that can be ground checked. In this way confidence and background experience is built up. The self-training is not a time-consuming job and is well recommended as a logical approach. Some interpretations of this type result in a very satisfactory high degree of success.

It is always a sound plan to take time to fly low over the proposed route before the project is begun, to check and add to the interpretation already completed.

An interpretational problem does occur occasionally when air photos taken many years previously are used. For example, forest fires can change the value of an interpretation made from an old photo or can prevent the correct interpretation of a recent one. See Fig. 7.

Trail Selection

Fig. 8 will serve to illustrate a very simple trail selection problem.

The presence of FI in large amounts is very evident on the high altitude photographs of this feature and the drainage lines across it are obvious.

The descending order of cover classes from AFI through BFI, DFI to EFI, will also be evident.

If the lesser units are not actually discernible under stereo examinations of the air photographs, the physiographic "atmosphere" of this type of feature will indicate a descending stature order to the open water in the drainage.

Certainly the stream in the narrow FI section can be seen, and corduroying the crossing with material from the AFI cover class less than 100 yards away would be straightforward. The confused jumble of fallen fire-killed trees in the AFI amply illustrates the reference to high vehicle impedence to be forecast for this muskeg.

On Fig. 9 is shown the type of brief notations that are added to the preliminary mosaic, the first route choices, and an indication of the general air-form patterns and cover classes that have influenced the interpreter in selecting where best to study the stereo photos.

Following the stereo examination and notation with coloured pencils on the air photos the information can be transferred to the final or intermediate mosaic by an office aid. This can be commenced as soon as the interpreter has finished three photos. Sufficient information can be interpreted in this way to give practically all that is needed at the rate of about three miles of proposed trail per hour.

A sample of this type of mosaic is shown in Fig. 10 and it can be seen that there is quite a lot of information scattered along the 13 miles of this proposed route. It is a 100% muskeg route.

For the discussion, in a little more detail, of what might form a representative portion of a route analysis mosaic constructed in this manner, refer to Fig. 11.

On the L.H. side is a section of the proposed route marked out in 1/4 mile increments. The predominant interpretations from that 1/4 mile interval are tabulated on the R.H. side. Space does not permit the inclusion of all the interpreter's remarks but the following might be indicative of what is inferred.

It will be noted that the topographic feature symbols are not necessarily the same for each section with the same cover class formula.

BEI is the most predominant cover class formula, and ice knolling in this type is the most prominent surface condition. This would be most pronounced during April-July, and would induce pitching motion in vehicles. Longer tracked vehicles would not find pitching so severe throughout the length of this trail. Certain types of the ponding observed would need to be detoured; the frequency depending upon the type of vehicles, but good cover types well suited for travel surround the ponds. The forward motion restriction due to ice knolling, vehicle subsidence, and cover class stature, will be low. The mat deterioration rate will also be low.

DFI, which on this photo appears to be patterned on the general drainage of the area, affords many favourable conditions for traffic. Hummock symbols (a) would indicate the near presence of water, and in themselves would not be as restrictive to vehicle as, for instance, the mounds (b) which are indicated as present in the BEI cover. Deterioration rate would be higher than BEI, but the subject of deterioration of trails is, of course, dependent upon the nature of the project. For instance, a tracked recovery vehicle making a round trip into muskeg to extricate a downed aircraft would not be too concerned about the deterioration rating of the muskegs. In any case, the vehicle impedence imposed by detouring in DFI is low.

The AFI which is shown linked to the BFI cover class is present adjacent to a drainage feature. Although the proposed trail avoids entering this heavier class formulae, its presence is indicated since it will supply suitable material to corduroy across the drainage.

DBF, for which local deviations and pronounced pitching would be forecast for some vehicles, has been found to be a very common "living" cover class springing up after fire in former BEI. If this is the case, entangled deadfall is likely to be present, but the mat deterioration with vehicles passage will be low. However, the DBF indicated here with k, l, and n ponding, is not associated with any evidence of former fire.

Incidentally, the high altitude air-form pattern over most of this short strength of trail is Terrazzoid.

Conclusion

Working on a prepared mosaic along the lines proposed here, a skilled interpreter should be able to sketch out a preliminary route, to carry out the necessary stereo examination of the air photographs, and to dictate a multi-page analysis of a 12-15 mile proposed route, all in the space of one working day. With a confirming flight over the area on the following day and revision of his report, if necessary, on the third day, an office interpreter can supply an appraisal of the access possibilities for any job in a short time.

The cost involved in using air-photo interpretation as an aid in the selection of routes in organic terrain is low in comparison with the value of the information that can be made available.

If these air-photo observations and condition forecastings are being fully effective, then they should result in a more expeditiously run operation.

Transportation companies or traffic foremen should be able to better employ the vehicles at their disposal and to do so with the minimum of maintenance. Drivers and field workers will be forewarned of obstacles to be expected and time-saving preparations can be employed to ensure speedy accomplishment of their tasks.

The majority of exploration work now being undertaken in our northern areas is of a seasonal nature and this imbalance of operations is becoming more and more costly. Planners are striving for ways and means to increase the operational period in the north and at the same time prepare themselves for the "panic" situations which often portend new interests and rapidly compound the exploration and development. When organic terrain is involved, situations such as these can become major problems for all concerned.

There is still a lot to be understood about this complex subject but the time has now come for everyone to direct their thoughts more to what can be done and not to what cannot be done with regard to muskeg.

Acknowledgment

The author would like to thank the Subcommittee on Muskeg for inviting the presentation of this paper and to express his appreciation to Mr. George Schlosser of Imperial Oil for assisting in its preparation.

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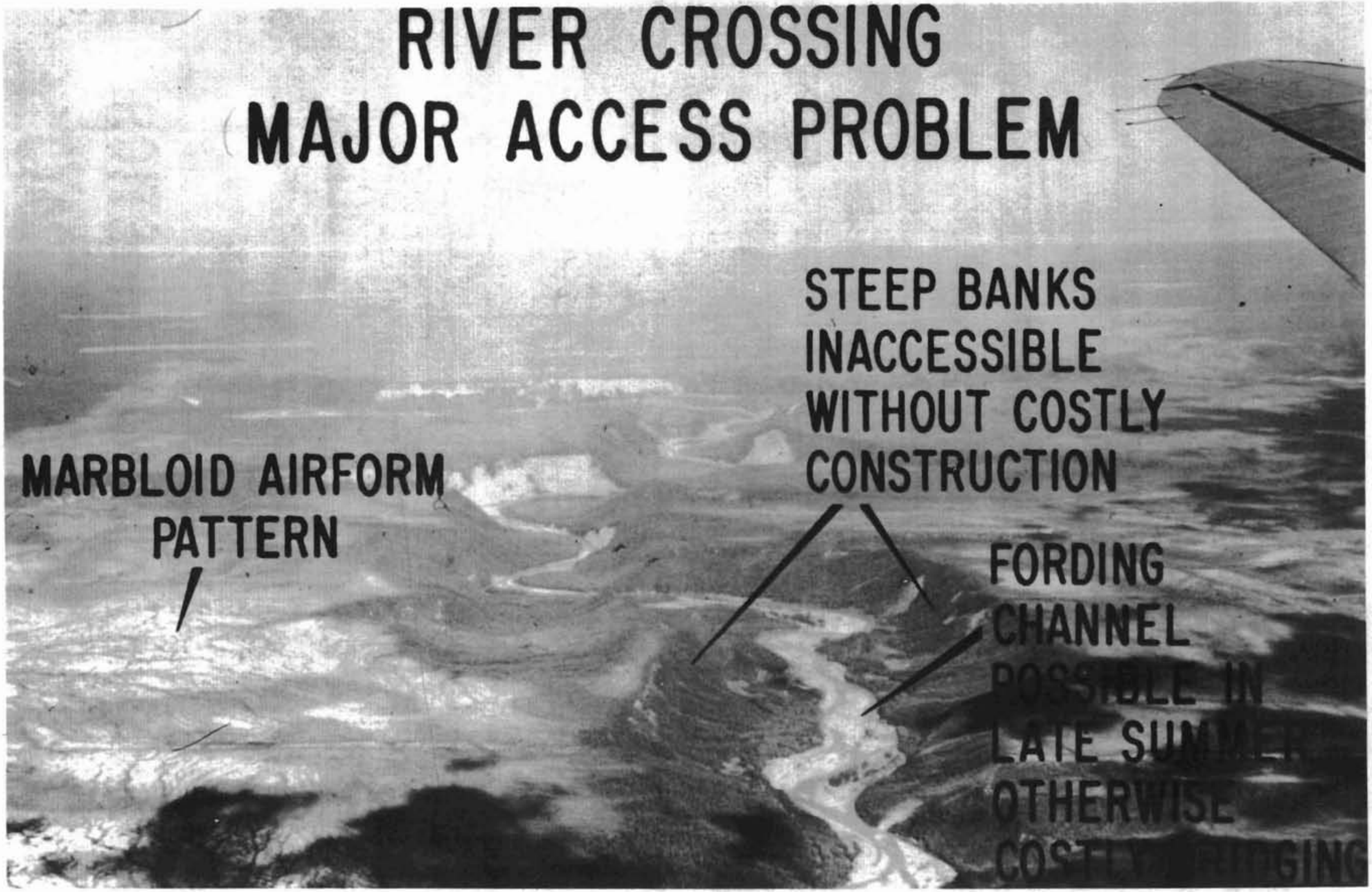

RIVER CROSSING
MAJOR ACCESS PROBLEM
STEEP BANKS
INACCESSIBLE
WITHOUT COSTLY
CONSTRUCTION
MARBLOID AIRFORM
PATTERN
FORDING
CHANNEL
POSSIBLE IN
OTHERWISE

Fig. 1.

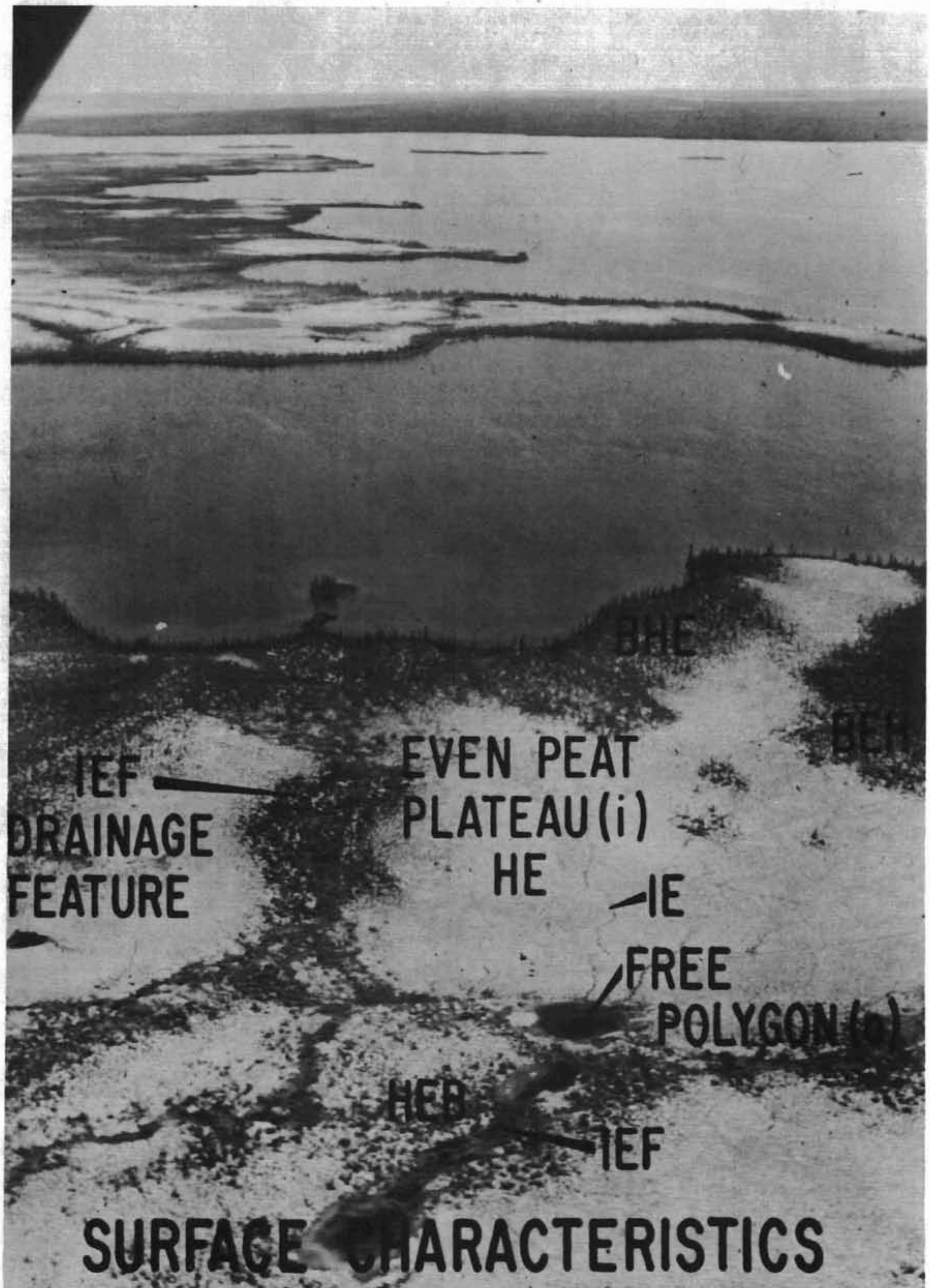

BHE
BEH
IEF
DRAINAGE
FEATURE
EVEN PEAT
PLATEAU (i)
HE
IE
FREE
POLYGON (o)
HEB
IEF
SURFACE CHARACTERISTICS
IN MARBLOID AIRFORM PATTERN

Fig. 2.

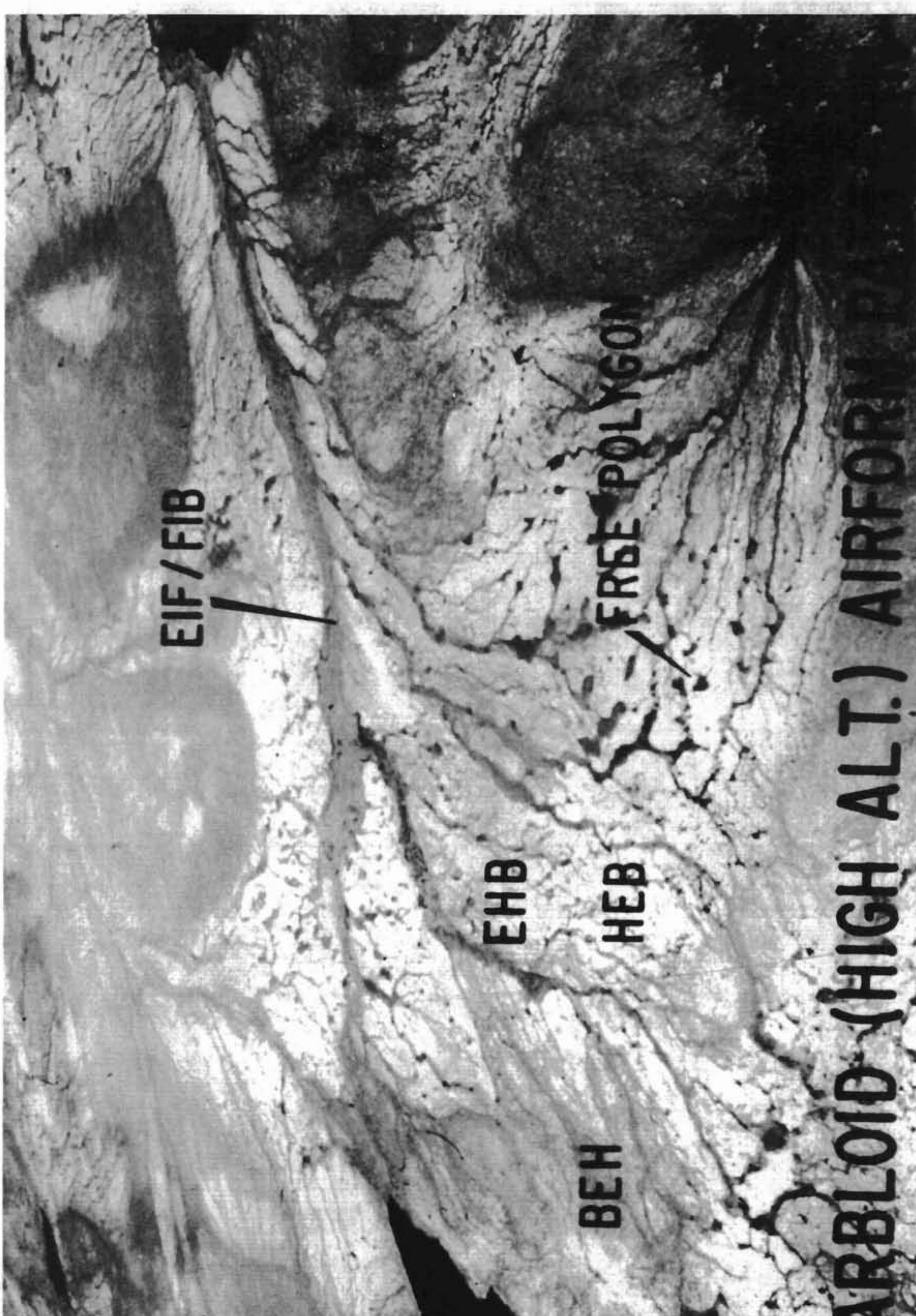


Fig. 3.

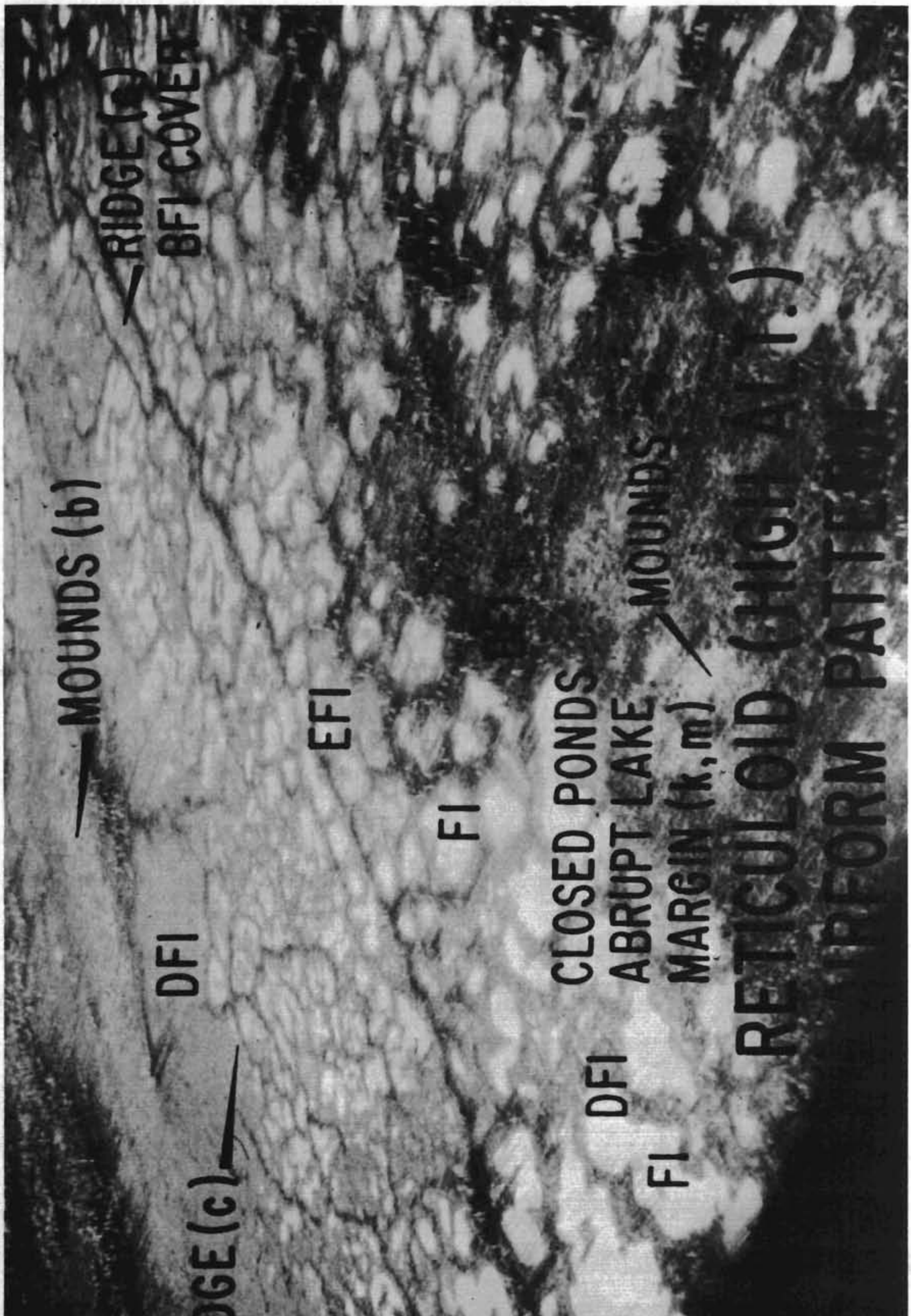

RIDGE
BFI COVER
MOUNDS (b)
DFI
RIDGE (c)
EFI
FI
CLOSED PONDS
ABRUPT LAKE
MARGIN (k,m)
MOUNDS
DFI
FI
RETICULOID (HIGH ALT.)

Fig. 4.

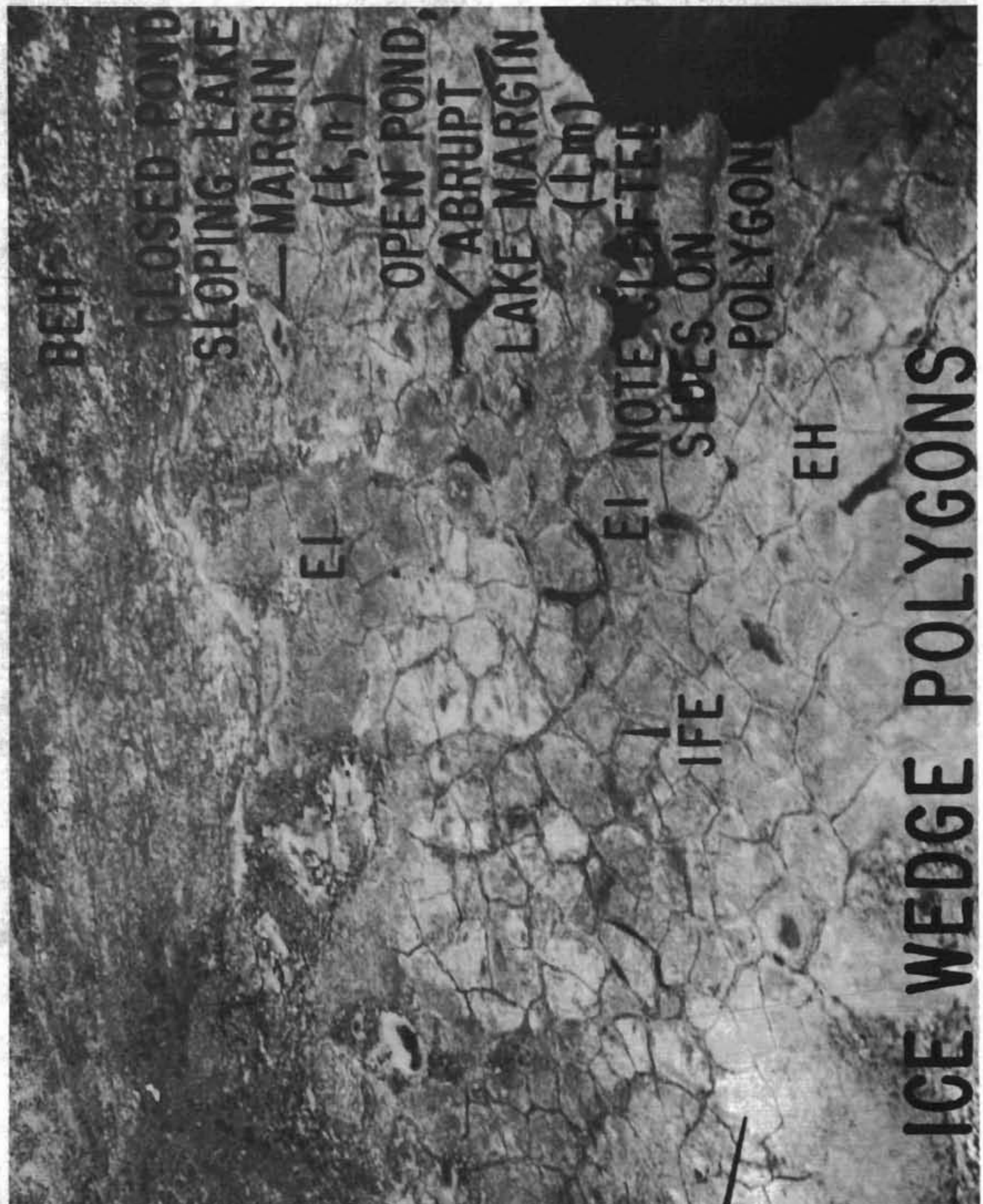

BEH
CLOSED POND
SLOPING LAKE MARGIN
(k,n)
OPEN POND
ABRUPT LAKE MARGIN
EI
EI
EH
IFE
HE
ICE WEDGE POLYGONS

Fig. 5.

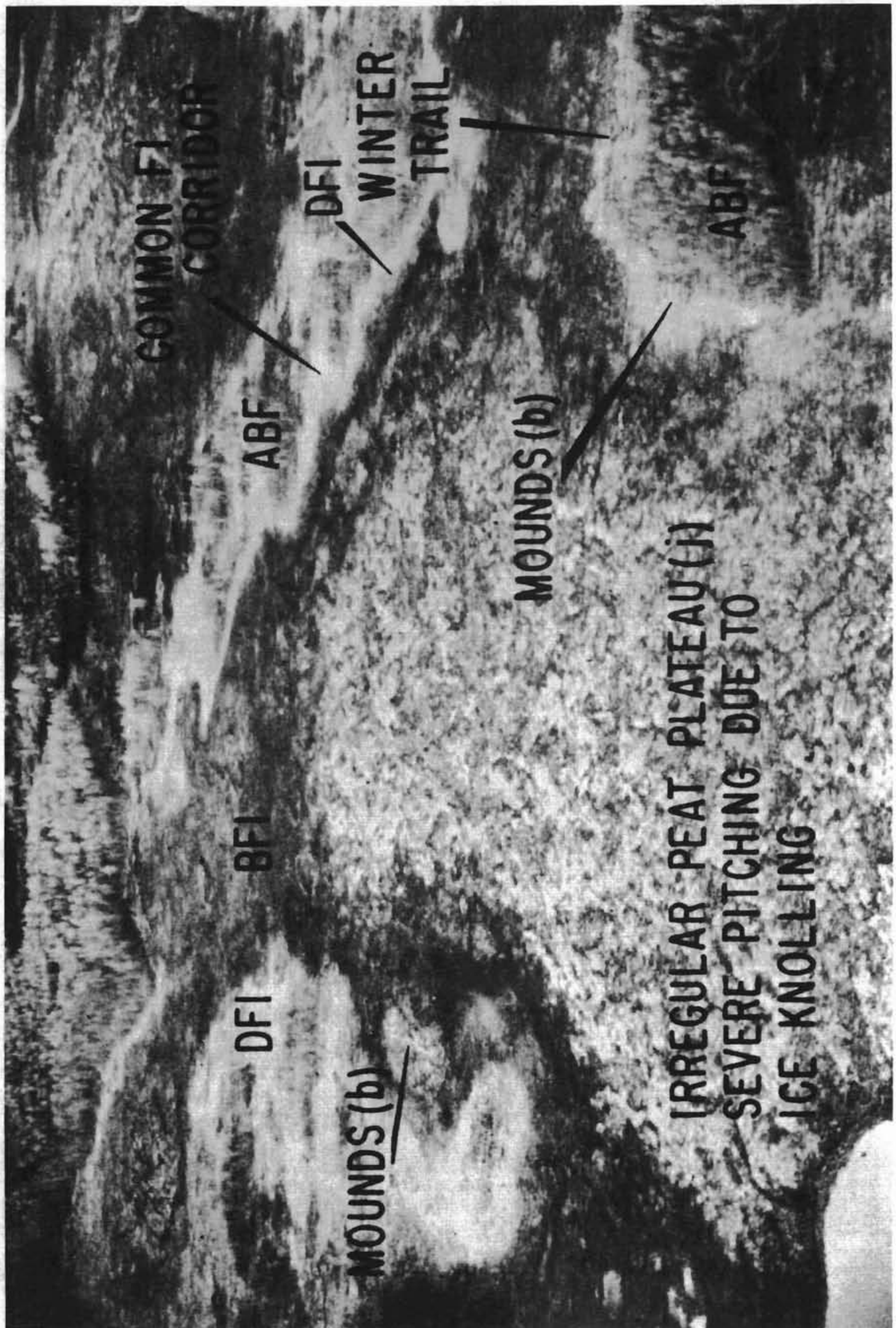

COMMON FI CORRIDOR
DFI
WINTER TRAIL
ABF
ABF
MOUNDS (b)
BFI
DFI
MOUNDS (b)
IRREGULAR PEAT PLATEAU (i)
SEVERE PITCHING DUE TO
ICE KNOLLING

Fig. 6.

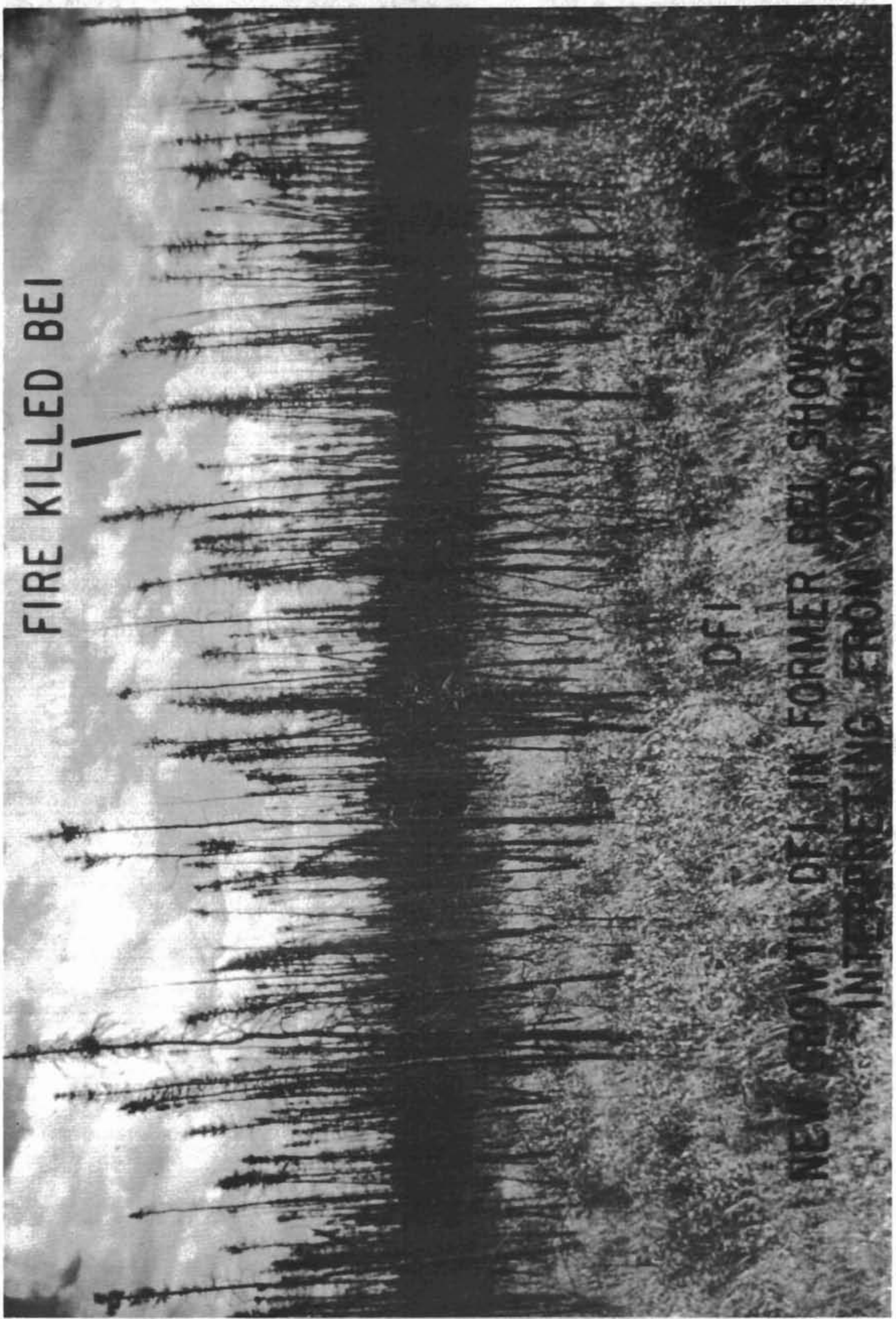

FIRE KILLED BEI
DF I
NEW GROWTH DF I IN FORMER BEI SHOWS PROBLEM
INTERPRETING FROM OLD PHOTOS

Fig. 7.

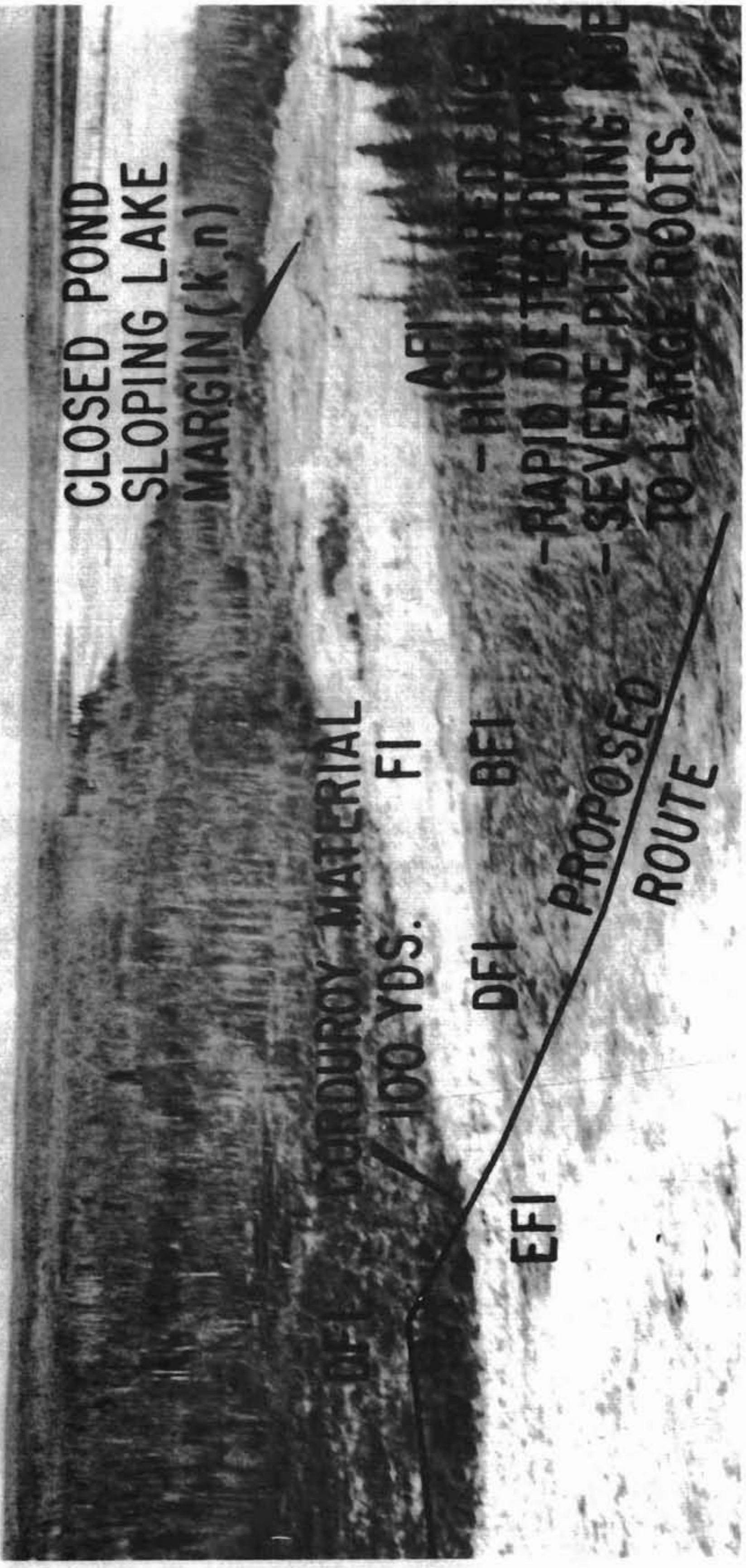

CLOSED POND
SLOPING LAKE
MARGIN (k,n)
AFI
- SEVERE PITCHING
TO LARGE ROOTS.
CORDUROY MATERIAL
100 YDS.
FI
BFI
DFI
PROPOSED
ROUTE
EFI

Fig. 8.

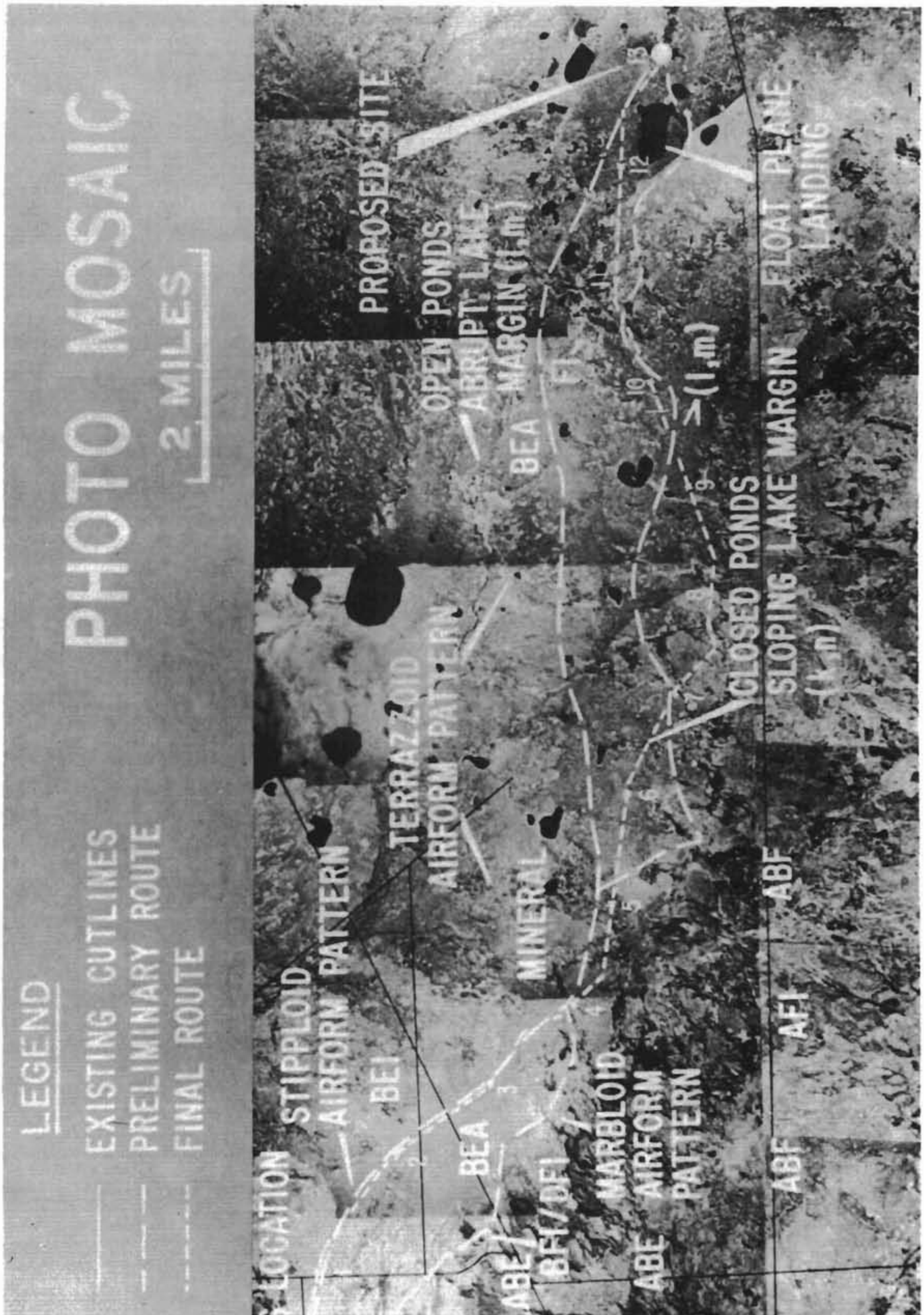

LEGEND
EXISTING CUTLINES
PRELIMINARY ROUTE
FINAL ROUTE
PHOTO MOSAIC
2 MILES
RIG LOCATION
STIPPLOID AIRFORM PATTERN
BEI
BEA
TERRAZZOID AIRFORM PATTERN
MINERAL
MARBLOID AIRFORM PATTERN
ABE
BFI/DFI
ABE
ABF
AFI
ABF
CLOSED PONDS
SLOPING LAKE MARGIN
OPEN PONDS
ABRUPT LAKE MARGIN
BEA
(l,m)
PROPOSED SITE
FLOAT PLANE LANDING

Fig. 9.

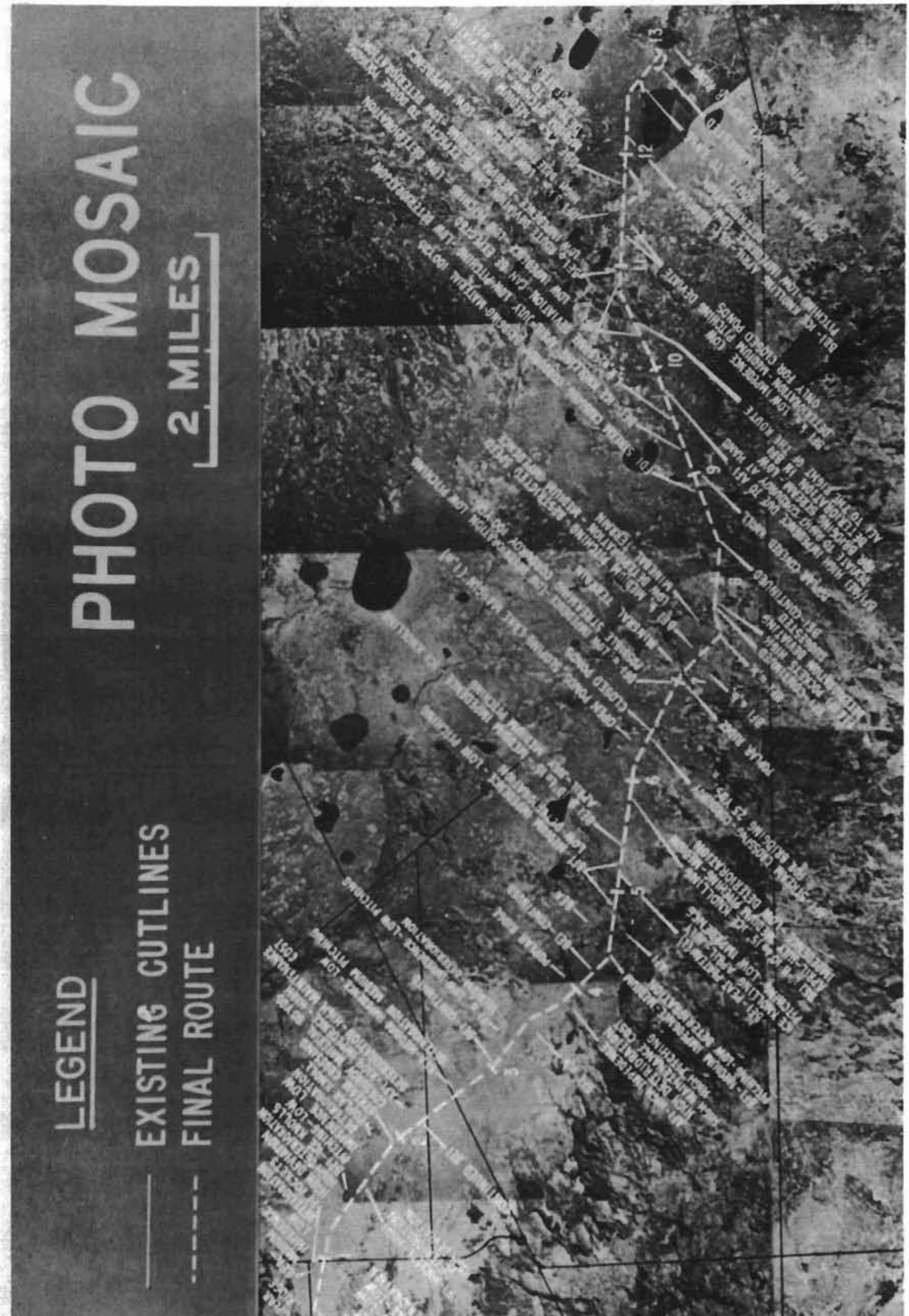
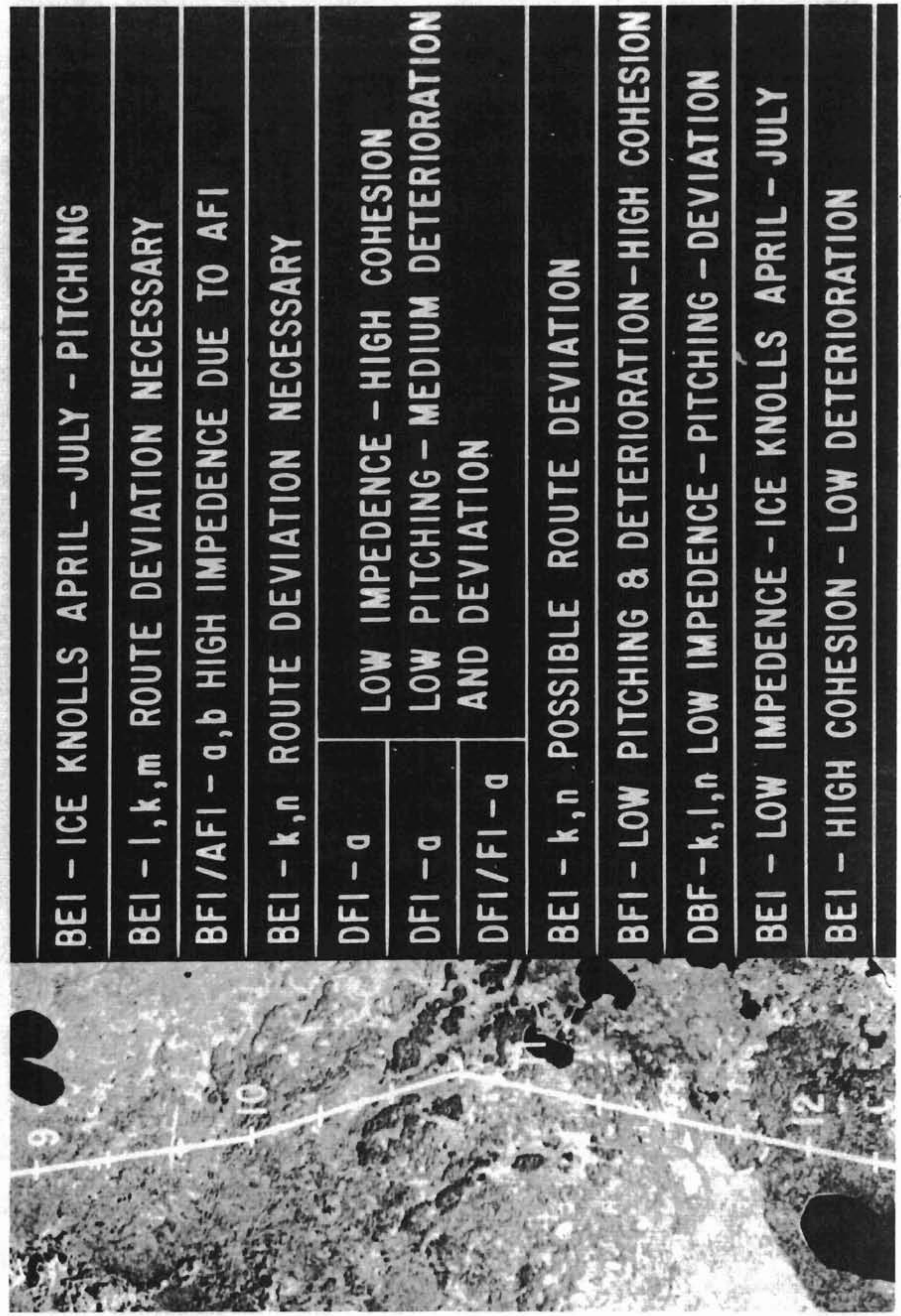


Fig. 10.



BEI - ICE KNOLLS APRIL - JULY - PITCHING
BEI - l,k,m ROUTE DEVIATION NECESSARY
BFI /AFI - a,b HIGH IMPEDENCE DUE TO AFI
BEI - k,n ROUTE DEVIATION NECESSARY
DFI - a
LOW IMPEDENCE - HIGH COHESION
DFI - a
LOW PITCHING - MEDIUM DETERIORATION AND DEVIATION
DFI /FI - a
BEI - k,n POSSIBLE ROUTE DEVIATION
BFI - LOW PITCHING & DETERIORATION - HIGH COHESION
DBF - k,l,n LOW IMPEDENCE - PITCHING - DEVIATION
BEI - LOW IMPEDENCE - ICE KNOLLS APRIL - JULY
BEI - HIGH COHESION - LOW DETERIORATION
9
10
11
12

Fig. 11.

Discussion

Mr. Peckover (Canadian National Railways) wondered how much saving is represented by this sort of interpretation. Mr. Keeling replied that work thus far has been only on interpretation of muskeg; the cost of interpretation is small compared to overall costs. The exact saving is not easy to assess; however, it is considerable. In answer to a question regarding the length of the deviations from route, Mr. Keeling said that they are not very far, often just an arc to get around a particularly difficult obstacle.

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II.4 THE ORGANIC TERRAIN FACTOR IN NORTHERN RAILWAY CONSTRUCTION

J. L. Charles

It would appear that the basic economic justification for extension of railway systems in Canada is their ability to haul heavy bulk commodities long distances with the minimum of motive and manpower. If further railway construction is required, this is to be expected in connection with the development of the northern territories. As there are vast areas of organic terrain, generally termed muskeg, in the North, study of the factors which affect railway location, construction and maintenance in such areas is important, in relation to the weight of a train.

Railways, however, are not a rigid structure; their design facilitates distribution of the heavy loads through wheels to rails, bridged on ties, ballast and subgrade, to the natural formation, so that flexibility is not as serious as it would be to a paved highway.

One train manned by a crew of four and powered with two diesel-electric locomotives, together generating 3,000 h.p., can haul 7,000 tons gross load over normal ruling gradients. Two hundred or more heavy-duty trucks, each with one or two drivers, would be necessary to move an equivalent load.

The principal factor to be considered is location. Railways, to fulfill their function of transporting heavy loads, should be located with low rates of ruling gradients and minimum rise and fall. In this respect, railway location requires more care than do highways. It may be more of a problem for railways to avoid unfavourable terrain.

Dr. Radforth, through his extensive research, has provided location engineers with a comprehensive classification system for organic terrain. Together with aerial photography and photogrammetry, this is very helpful to indicate the various types of muskegs which should be avoided, if practicable to do so, in relation to gradients and distances between terminals.

What may appear to be expensive initial cost of construction may be justified by reduced annual expenditures for maintenance and operation. For example, to obtain overall economy, under some conditions it may be advisable to avoid muskeg to the extent of locating on solid rock and setting the grade line so that there will be sufficient solid rock excavated to construct embankments on organic materials. In such cases, the grade should be set so that the weight of rock required for embankments will compress, or displace, the organic materials upon which it is placed. The characteristics of the formation under the organic materials should be ascertained; in some circumstances they seriously aggravate the situation.

Where embankments are to be built with the grade line unavoidably low in relation to the ground line, the organic materials should be cast out before construction is commenced.

Drainage is a major factor. However, where embankments are wholly constructed - or even the base to above high water level - of shot rock, ditching is not as important as in areas where embankments are to be constructed of other materials, particularly cohesive soils. Drainage is then of the utmost importance and should be carried out in advance of grading. Good topographical maps are essential to design, and instrument men who are not averse to getting wet and who do not become excited about flies are the men to stake drainage schemes on the ground.

Where topography is so featureless that thorough drainage is impracticable, the toes of embankments, other than those of shot rock, should be protected by berms built to above high water level.

Construction across long continuous sections of muskeg presents a special problem. It can be overcome by laying skeleton track on the natural ground when frozen during winter and commence lifting by train fill methods as soon as conditions permit in the following Spring. The northerly 150-mile section of the Hudson Bay Railway was built in this manner in 1928-29.

Settlement should be expected and embankments should be constructed of additional width and height to provide for this when trail fill can be hauled comparatively long distances. As previously stated, a railway is a flexible structure and track can be lifted and resurfaced under traffic, between the passage of trains.

Conditions vary considerably, so location of culverts and preparation of foundations requires study at each site. A wide timber mat and good camber will help prevent settlement and distortion. Box culverts built of 12" x 12" cedar timbers drift bolted together will withstand considerable distortion and will function unless failure is total. Inverts of culverts should be set sufficiently low to permit free passage of water and prevent accumulation of water upstream, but not so low that the culvert might become submerged. The number of culverts and their capacity should permit free flow from the upper side of the railway to lower ground. In muskeg territory, the dimensions of culverts should be sufficient such that they are never expected to carry more than one-half their capacity. This is an added precaution against settlement and distortion. Where there is a possibility of progressive accumulation of ice during winter, supplementary culverts should be installed at elevations above extreme ice level in order to ensure a free waterway during the Spring run-off.

It is poor practice to endeavour to economize on culverts. Also, offtake ditches should be provided to divert water from the right-if-way as frequently as the topography will permit.

To carry streams too large for culverts but not large enough to require wide spans, timber pile trestles are usually more satisfactory than rigid type bridges. Flexible structures may be easily resurfaced in the event of either settlement or heaving.

Rail of not less than 100 lbs. per yard is advisable as there is less deflection than with lighter rail. Adequate anchors per panel are essential to prevent rail creep. Longer track ties than normal assist in distributing loads and also assist in reducing rail creep, which is aggravated by a comparatively soft roadbed.

The conditions under which organic terrain has to be contended with vary widely. In the North, where there are few differences in surface elevation, the areas of organic terrain are frequently very extensive; however, they may not be as deep as the smaller muskegs which occur in territory of rugged rock formation. Treacherous organic terrain may also be encountered at the summits of some mountain passes as well as on side-hill slopes in mountain country.

Where the organic materials, and others, are in a permanently frozen condition, care should be exercised to preserve this condition. Costs for construction and maintenance will then be comparatively light.

Sectional trench mats, such as those used on the Western Front during the First World War, are helpful for access through extensive muskeg areas of personnel on foot.

In organic terrain, the use of tripods is the most practical method for erection and maintenance of telegraph lines in connection with railways. The first extensive use of tripods was on the Hudson Bay Railway in 1927.

Initially, the principal terrain factors are ascertained: classes of organic terrain, their surface extent and depth and the underlying formation. In some locations, this latter factor may present a more serious problem than the organic material above;

sloping rock, for instance, can cause much difficulty. Each situation requires individual study to reach a decision whether to:-

1. Build on the natural surface, with or without a corduroy mat and expect compression, settlement and perhaps displacement.

2. Employ progressive pre-consolidation.

3. Cast out the organic materials before commencing to construct an embankment.

4. Impose a super load above subgrade elevation and plow this off, thus widening the embankment, before track-laying.

5. Construct a pile trestle.

6. Drain with side ditches and offtakes.

7. Drain with offtakes only, without cutting the surface mat parallel to and near the embankment.

Also in railway construction, ruling gradients may necessitate cuttings through "summit" muskegs followed by excavation below sub-grade to a stable formation; backfilling is essential.

The respective solution should be based on practical economics - capital costs together with maintenance expenditures, in relation to classes of traffic and potential earnings.

Time available for construction may also be a factor to be considered. Frequently there is a deadline to be met relative to a date set for production from a mine or other industry.

The approach to initial construction of a branch line for pioneer development where known sources of traffic are light, should be quite different to building a Class "A" line to handle immediate proven sources of heavy traffic. However, the pioneer type of line should be located and designed so that it could be readily up-graded if and when development of the territory to be served would produce sufficient traffic to justify additional expenditure.

Two comparative examples of wide difference in classes of construction as well as maintenance of railways recently built across extensive areas of organic terrain are the Canadian National Railways' branch line to haul one train of concentrates every other day from Sherritt-Gordon Mines at Lynn Lake, Manitoba, in comparison with the Quebec Labrador and North Shore Railway to transport immediate shipments of iron ore up to thirteen million tons per season. Both of these lines function with respect to requirements.

The author's first experience with organic terrain was forty-eight years ago, during construction of the Hudson Bay Railway. Good labour was plentiful. The procedure was very simple, with a minimum of overhead for staff and investment.

When winter conditions facilitated it, supplies were transported to caches at ten-mile intervals along the line. Before Spring break-up, teamsters and horses were moved off. One man was left in charge of each cache, under a "walking boss", who would supervise up to fifty miles of line.

In the Spring, labourers walked with their packs from the railhead, distances up to one hundred miles, to the cache nearest their respective working sites. These men, termed "station-men", usually worked in partnership, about six men in a group. They were given small sub-contracts to be completed during one summer season. After being issued with supplies, axes, shovels, wheelbarrows, a tin stove and a few nails, they packed these few necessities to their job, built a log shack, hewed out poles to make trestles and planks, then went to work excavating offtake ditches and wheeling materials from side borrow to construct embankments. This work was on a firm unit price basis, as measured by the railways' engineers.

There was no pre-excavation of muskeg and much of the embankments were built of organic materials placed on the natural surface. As these embankments dried, fire became a hazard, which could be difficult to extinguish. Track was laid when the roadbed was frozen and was lifted and consolidated by trainfill the following summer.

With draglines and other mechanical equipment, the situation today is very different but many miles of railway were constructed over extensive areas of organic terrain by the earlier simple method such as between The Pas and Churchill, a distance of 510 miles. This track carries heavy loads - 21,000,000 bushels of grain annually and this will soon be augmented by nickel from "Inco's" new plant at Thompson.

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Summary of Address Given at Dinner Meeting by R. F. Legget

The guest speaker at the dinner meeting - attended by close to 100 people - was Mr. R. F. Legget, Director, Division of Building Research of the National Research Council. Mr. Legget described the commencement of muskeg research in Canada, pointing out that it was just 20 years ago that he and Dr. Radforth first collaborated on a problem involving muskeg. He referred to the trafficability studies carried out on soils, snow, and muskeg during and after the Second World War, and stated that in those early days it was very difficult to find out exactly what muskeg was as everyone had his own definition. Mr. Legget paid tribute to the pioneer work of Dr. Radforth and briefly traced his research from its beginning to the present time. In illustrating how the scope of the muskeg problem had changed in the past half century, Mr. Legget pointed out that it is presently estimated that muskeg covers almost 500,000 square miles of the land area of Canada. In 1904, the estimate was a mere 37,000 square miles!

Mr. Legget referred to a very early record (McTaggart, 1829) indicating an approach to the engineering problems arising from the presence of a local area of organic terrain, now known as Coot's Paradise. The current approach to similar problems of road construction, drainage, etc., is represented by the present very successful conference.

In conclusion, Mr. Legget raised some questions about future progress that may be anticipated. He cited physical problems such as evapotranspiration and heat balance at the surface of the muskeg; engineering problems such as access, road construction and drainage; and problems of utilization of peat, such as for fuel or as a building material.

Mr. Legget's talk was followed by a colour-sound film entitled "Building in the North", produced for the Division of Building Research, National Research Council, by the National Film Board of Canada.

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III. 1 INVESTIGATIONS OF THE PROBLEM OF CONSTRUCTING ROADS ON PEAT IN SCOTLAND

J. R. Lake

Conditions of high rainfall, high humidity and poor drainage have encouraged the development of large areas of peat in Scotland. Most of the deposits occur at the higher altitudes in the north and west of the country where blanket bogs (8) and raised bogs (8) exist with thicknesses up to 15 ft. In the Lowland areas, peats of the basin type (8) often with raised bog developments are common, and these may be as deep as 35 ft. Roads constructed on peat continue to settle for many years and this often produces serious differential settlement of the road surfaces with consequent deterioration of the surfacing and poor riding qualities. The magnitude of the problem may be judged by referring to a Conference held in Inverness in 1953 (9) when the delegates considered that the most pressing problem in Highland road construction was the difficulty of constructing satisfactory roads on peat.

Since the formation of the Scottish Branch of the Road Research Laboratory, a study of the properties of peat has formed a large part of the research effort and some progress has been made towards a better understanding of the behaviour of peat under load. This paper summarizes the work done so far by the Laboratory on this subject.

Review of Existing Methods of Constructing Roads over Peat

As the initial stage in a programme of research and investigation, a survey (8) was carried out of the world literature on methods of constructing roads over peat together with a more detailed examination, on actual sites, of methods employed in Great Britain and Ireland. The review showed how different were the methods used in various parts of the world. This, and the fact that no definite conclusions regarding the relative merits of the various methods could be drawn, emphasized the limited knowledge on this subject and the need for a careful laboratory study of peat in conjunction with carefully designed field experiments.

Existing Practice in Scotland

An appraisal of methods of constructing roads over peat in Scotland in past years does not reveal any definite policy. Engineers have realized that with the present state of knowledge the only satisfactory way of constructing a road which will not settle differentially is to excavate the peat from the line of the road and replace it with more stable fill material. However, this is expensive and, up to a few years ago, public funds available for road works in the United Kingdom were very limited and only rarely was peat of depth greater than 5 ft. excavated. It was usual to find greater depths left undisturbed and designs were sometimes developed to attempt to reduce differential settlement of the road after construction.

Considerable differences of opinion exist in Scotland on how best to construct roads over peat. For example, some engineers employ extensive drainage measures while others are opposed to this idea and employ only shallow ditches to drain surface water, contending that deeper drains reduce the level of the water table, reduce the moisture content and cause shrinkage of the peat.

Methods of constructing roads over peat often included the use of fascine mattresses between the peat and the road pavement. Where available, glacial gravels were laid on the fascines and these often formed both the road base and the surfacing. Gradually, as more funds became available, the gravel roads were strengthened, often with crushed stone, and surfaced with bituminous materials. Most roads in the Highlands developed in this way.

One case is known in Scotland where concrete was used for a road over peat. This length of road in the Highlands built in the 1920's consisted of a heavily reinforced concrete raft (2) eight inches thick with longitudinal and transverse beams provided at the edges of the slabs to prevent differential settlement and displacements of the peat from under the slabs. Although very expensive by present-day standards, it is of interest to note that this length of road is in good shape and has carried light to medium traffic for nearly forty years with very little surface maintenance. Whether or not this particular design of concrete raft would have been so successful on a heavily trafficked road seems doubtful.

The road building programme in Scotland has increased during the past five years and major roads in the Lowlands and southern part of the country are being converted from single to dual carriageway. Where peat is encountered, the general trend nowadays is to excavate the soft material, and depths of up to 20 ft. have been removed.

Laboratory Studies

Consolidation tests were carried out in the Laboratory to investigate the effect of length of drainage path on the dissipation of pore water pressure in peat and the rate of compression of the peat.

Remoulded and undisturbed peats from the sites of full-scale and small-scale loading experiments were used. The samples were 3 ins. in diameter and with each peat, sample thicknesses of 1/2 in., 1 in., 2 ins., and 4 ins., were tested under single drainage of pressure, each 204 lbs/sq. in., were applied and pore water pressures were measured in the bases of the samples furthest away from the drainage faces. Settlement and pore water pressure measurements were taken under each increment of pressure until the amount of settlement in twenty-four hours was 1 per cent or less of the total recorded settlement.

Fig. 1 shows the rates of settlement of the four samples of remoulded peat from one source under five separate increments of pressure. Substantially the same results were obtained with both undisturbed and remoulded peat samples from the other sites. The relation between thickness and time for the dissipation of the excess pore water pressure conformed reasonably well with the law relating time and length of drainage path suggested by Terzaghi (7) for clays.

It may be deduced from Fig. 1 that except for the first increment of load, the rates of settlement of the samples after one minute from the time of loading were largely independent of the length of the drainage path. Table I shows the amount of settlement that occurred after the pore water pressure in the laboratory tests had returned to normal.

TABLE I

SETTLEMENTS AFTER PORE WATER PRESSURE RETURNED TO NORMAL

Pressure increment (lb/ft^2)	Settlements (expressed as percentage of total settlement)			
	Sample thickness			
	1/2 in.	1 in.	2 ins.	4 ins.
0-204	39	19	10	12
204-408	48	36	24	21
408-612	67	50	33	28
612-816	80	67	52	29
816-1020	81	72	60	64

Small-scale tests

Small-scale loading tests were carried out in 1957 and 1958 (5) on two peat bogs in Scotland to measure the effect of vertical sand drains on the pore water pressures developed in peat and on the rate of compression of peat. Loading was done by means of water-filled rigid and flexible tanks. As the loading tests at the two sites were the same in principle and substantially the same results were obtained, it is proposed to describe only the larger of the two tests.

The site of this experiment was near Dalmellington, Ayrshire, where the peat was of the basin type having a raised bog development with characteristic dome-shaped surface (8). The peat was 13 ft. 6 ins. deep and was underlain by a thin layer of silty clay resting on gravel-sand. The average moisture content of the peat was 1850 per cent and the water table was 6 ins. from the ground surface. Loading was done with two flexible rubber-fabric tanks, 12 ft. long x 8 ft. wide, filled with water to a depth of 3 ft. 2 ins.

In the area to be occupied by one of the two tanks, vertical sand drains 6 ins. in diameter and spaced at 3 ft. 6 in. centres, were installed by the driven mandrel technique (6), to a depth of 12 ft. 6 ins. The grading of the sand used was just within the lower limit of grading suggested by Stanton (6). Turf was removed from the areas to be occupied by the tanks and replaced with 12 ins. of sand of the same type as used in the drains.

The apparatus used to measure pore water pressures in the peat with and without sand drains was similar to the type developed at the Building Research Station in Great Britain (3). Fifteen filters were installed at different depths under each flexible tank. Each tank contained fifteen settlement gauges specially designed for the purposes of the test and settlements were measured with a geodetic level from the start of loading, relating the movements of the gauges to an ordnance benchmark on a bridge a few hundred yards away from the site.

The empty rubber-fabric tanks were in position for several months before loading, and filling with water increased the surface pressure by 200 lbs/sq.ft. The tanks were filled on consecutive days at a rate of 2 ins/min.

The maximum pore water pressures under the tanks with sand drains at Dalmellington were approximately 65 - 95 per cent of the values recorded under the tank without sand drains (Fig. 2) and the detailed measurements of pore water pressures showed that the maximum values occurred in the first day (Fig. 3). Differences in pore water pressures under the two tanks were noted for a period of up to 70 days from the start of loading, after which time the excess pore water pressures were virtually dissipated.

Although vertical sand drains affected the pore water pressures in the peat, the rates of settlements of the tanks were not affected (Fig. 4). The rates of settlement of the central points of the two tanks at Dalmellington were practically identical and similar results were obtained with other corresponding pairs of settlement gauges. These results confirm results obtained by Brawner (1) during experiments in British Columbia. The settlements, occurring at Dalmellington after the pore water pressures had apparently dissipated to the original values, amounted to 10 per cent of the total settlement recorded in 140 days.

Full-scale Experiment

Early in the 1950's, the County Council of Dunbartonshire and the Scottish Home Department prepared a road scheme to convert part of the Trunk Road A. 80 near Condorrat into a dual carriageway. The existing carriageway was 22 ft. wide, and this was to be widened to include two 24 ft. wide carriageways. Under an 875 ft. length of the road, peat of the basin type with a raised bog development (8) lay in two distinct pockets of 20 ft. and 33 ft. maximum depth respectively. A layer of soft compressible silty clay 5-10 ft. thick existed under the peat and this, in turn, was laying on a bed of sand. As the existing carriageway was to continue to carry heavy traffic during the reconstruction, it was considered inadvisable to attempt to excavate the peat adjacent to it during the construction of the new carriageway and the road authorities therefore decided to use vertical sand drains as a possible method of accelerating the settlement of the road embankment on the peat and increasing the stability particularly during the construction period.

At the suggestion of the Laboratory, vertical sand drains were omitted for experimental purposes from part of the peat under the road embankment and detailed measurements of pore water pressure and settlement were made on the sections with and without sand drains to determine the effectiveness of vertical sand drains in peat (4). Construction of the carriageway commenced in mid-1958 and was completed by the middle of 1960. Experimental observations were kept during this period and are continuing. Fig. 5 is a soil profile along the center line of the new carriageway.

In the construction of the new carriageway, a working platform 2 ft. 6 ins. thick of permeable gravel-sand was first laid on the area to be occupied by the sand drains. The drains, 12 ins. in diameter and spaced at 10 ft. centers in a triangular pattern, were then driven through the fill, peat and silty clay into the sand underneath using the driven mandrel technique (6). Fascines were laid on the surface of the ground where no sand drains were installed as a precaution against differential settlement and possible failures during construction. The road embankment was constructed in 9 in. to 12 in. layers allowing time for the pore water pressures to dissipate at least in part before the next layer was added.

A maximum height of fill of 27 ft. 9 ins. was required to raise the surface level of the fill to a height of 8-9 ft. above the original ground level. This 8-9 ft. included a 5 ft. surcharge which was added to increase the rate of settlement of the embankment independent of any effect achieved by the presence of the sand drains.

Very large settlements occurred during the construction of the road embankment and approximately two-thirds to three-quarters of the height of the embankment was lost due to settlement. Observations showed that the settlements were partly caused by consolidation and partly by displacement under constant volume. Fig. 6 shows the settlements of the two parts of the embankment where the ground conditions were similar before loading, and therefore comparable. The results show that the sand drains did not increase the rate of settlement, in fact the embankment on the sand drains appears to have settled less than the other part of the embankment despite the fact that the fill was placed more quickly on the sand drain length. This effect was also noted in the smaller of the two small-scale loading tests mentioned earlier.

Perhaps the most important practical result emerging from the settlement results obtained to date is the displacement of the peat which occurred during the construction phase despite the relatively low speed of construction (approximately 9 ins. in three weeks). With such soft peat, it appears that either the fill has to be added very slowly, too slowly for normal practical conditions, or alternatively, it must be accepted that the peat will be overstressed and some displacement will occur. The effect of displacement on any vertical sand drains is uncertain.

Very high pore water pressures were obtained during the construction of the embankment. Expressing the excess pore-water pressure obtained during construction as a percentage of the applied pressure of fill, ratios approximately 100 per cent were obtained during the initial stages of construction. A comparison between the pore water pressures at corresponding points in the peat with and without sand drains is given on Fig. 7.

Discussion

Laboratory small-scale and full-scale experiments have shown that reducing the length of drainage path either by using thin samples in the laboratory or by installing vertical sand drains in the field, did not affect significantly the rate of settlement of the peat under load. These measures did, however, appear to increase the rate of dissipation of excess pore water pressure. It is clear that the results are at variance with the classical theory of consolidation which would indicate an increase in the rate of settlement corresponding to a more rapid decrease in excess pore water pressures.

The experimental conditions appear to have been reasonably satisfactory and it seems probable, therefore, that the behaviour of peat under load is affected by properties of the peat itself, as yet not fully understood. A fuller understanding of these matters will only be derived as a result of more fundamental studies of the properties of peat.

Acknowledgment

The author acknowledges the assistance received from his colleagues, Messrs. C.K. Fraser, R. Richards, and G. C. Woodford who assisted in making the observations during the tests described in this paper. The work described was carried out as part of the programme of the Road Research Board of the Department of Scientific and Industrial Research. The paper is published by permission of the Director of Road Research.

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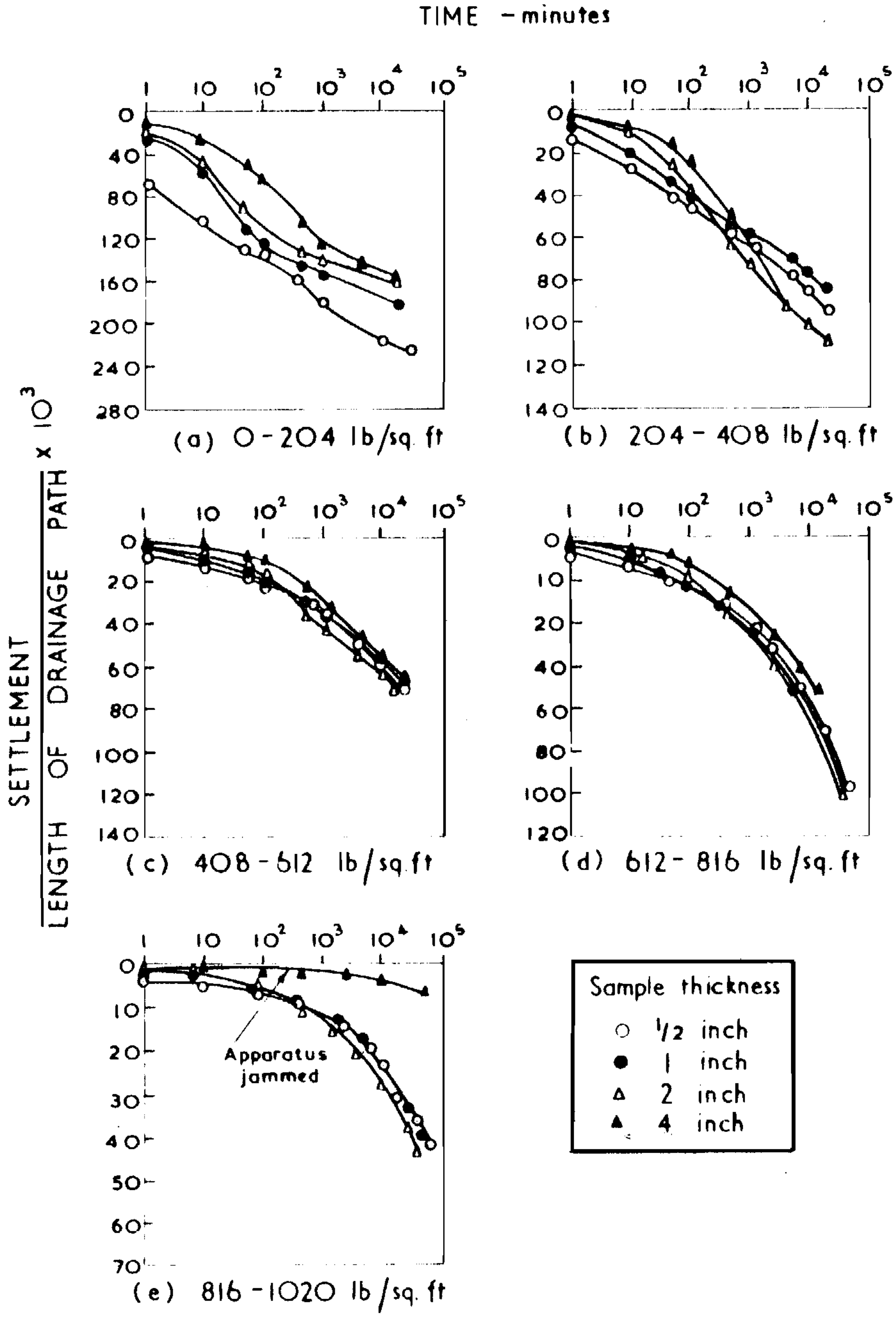

Fig I. RATES OF SETTLEMENT OF FOUR SAMPLES OF REMOULDED PEAT HAVING DIFFERENT LENGTHS OF DRAINAGE PATH

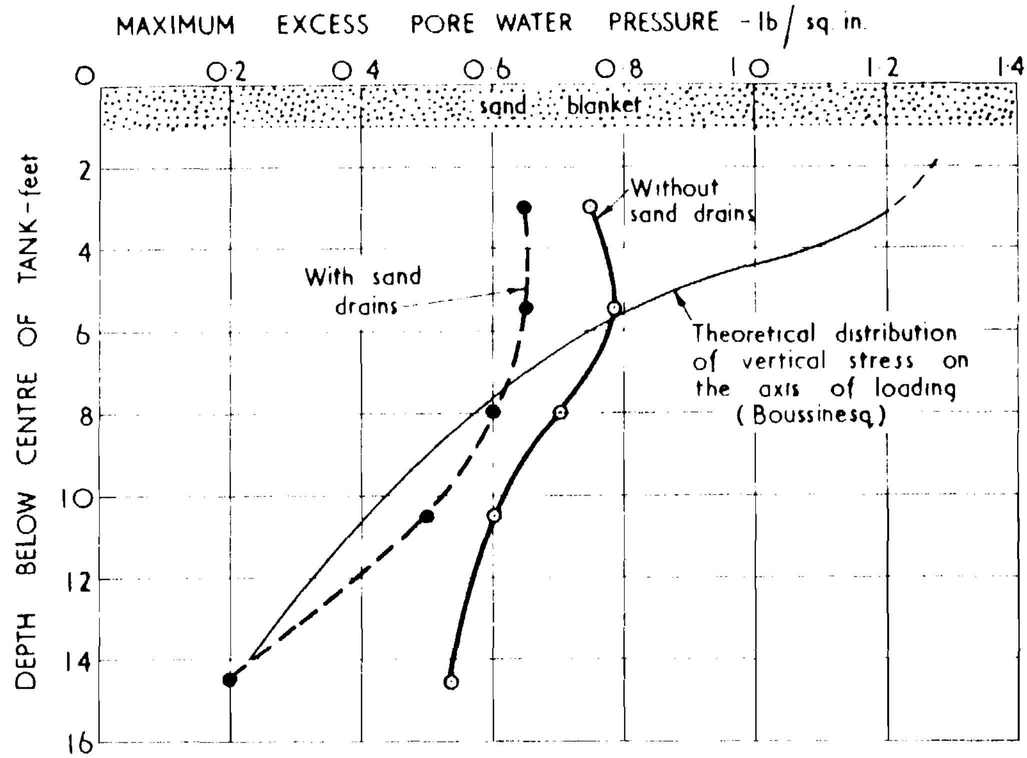

Fig. 2 DISTRIBUTION OF THE MAXIMUM EXCESS PORE WATER PRESSURES IN THE PEAT UNDER WATER - FILLED TANKS AT DALMELLINGTON

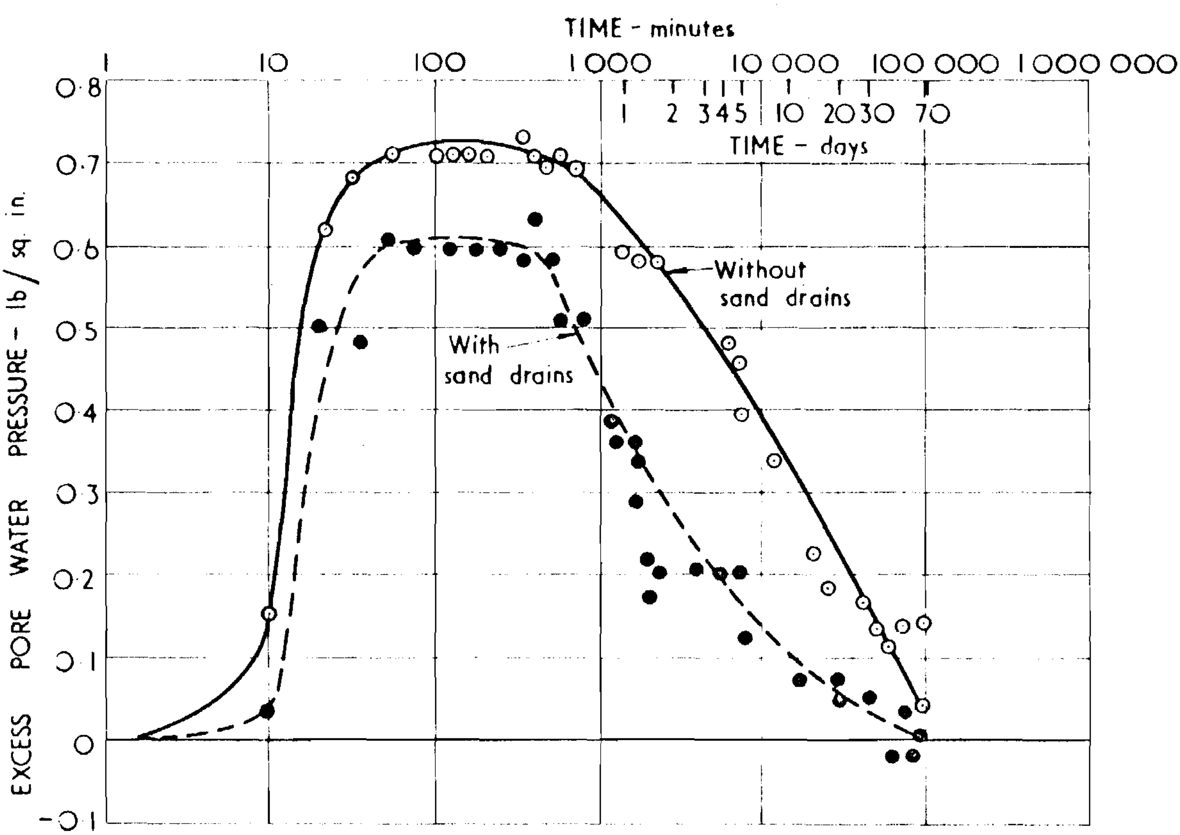

Fig. 3. RECORDS OF THE EXCESS PORE WATER PRESSURES IN THE PEAT 8 ft. UNDER THE CENTRES OF WATER - FILLED TANKS AT DALMELLINGTON

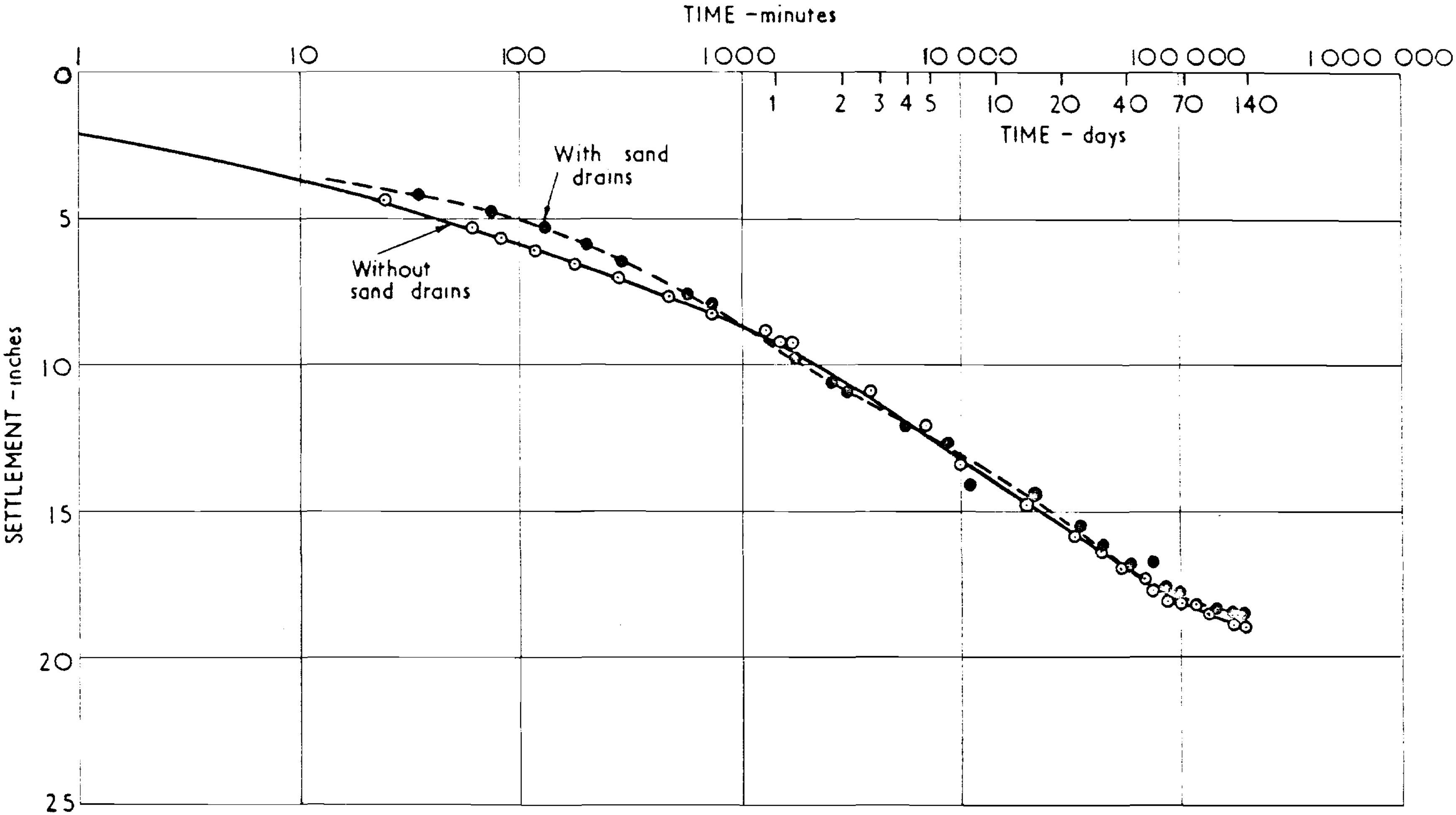

Fig. 4. SETTLEMENT / TIME CURVES OF THE CENTRES OF THE TWO FLEXIBLE TANKS AT DALMELLINGTON

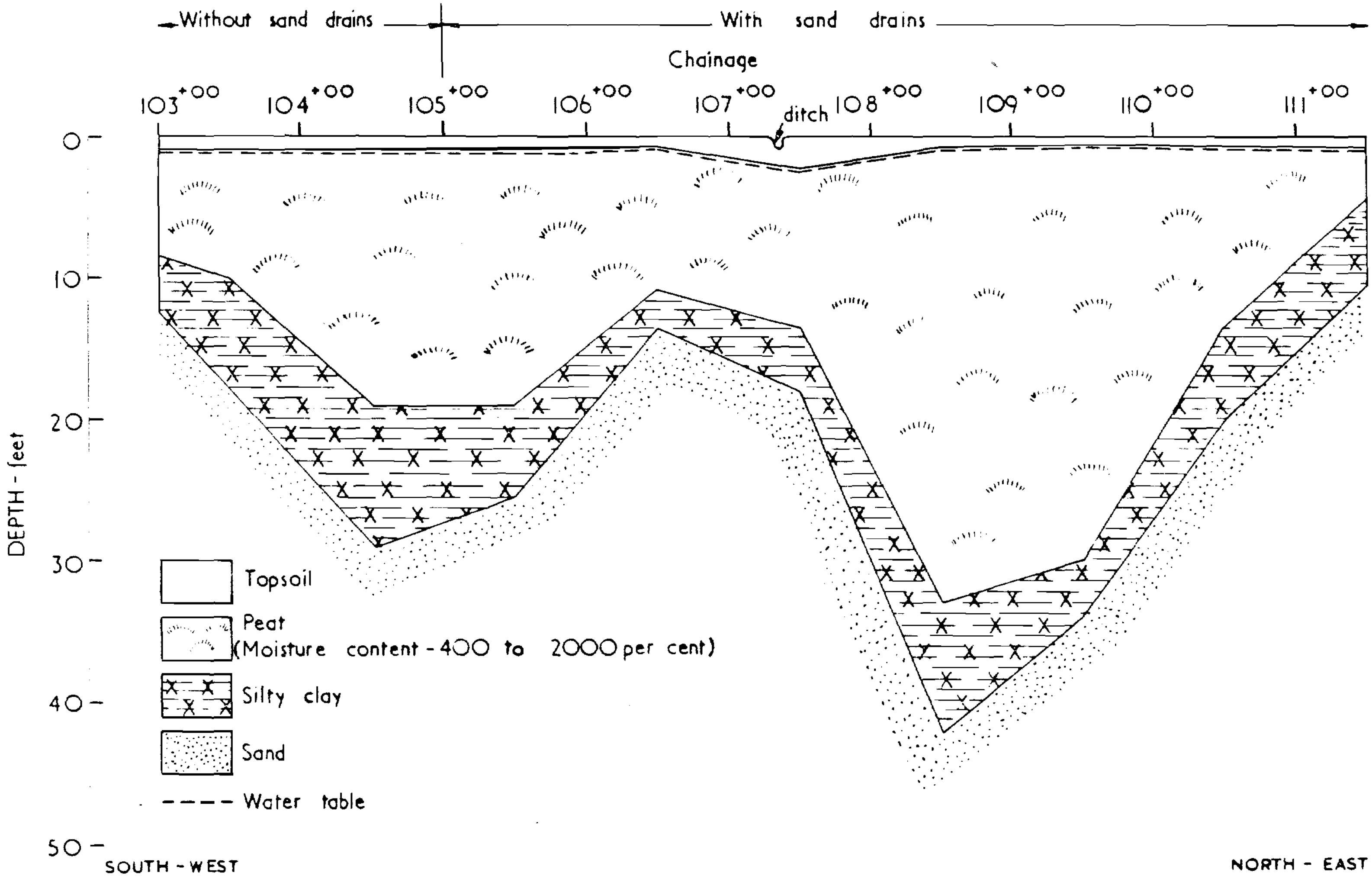

Fig. 5. LONGITUDINAL PROFILE ALONG CENTRE LINE OF NEW CARRIAGEWAY AT CONDORRAT, DUNBARTONSHIRE

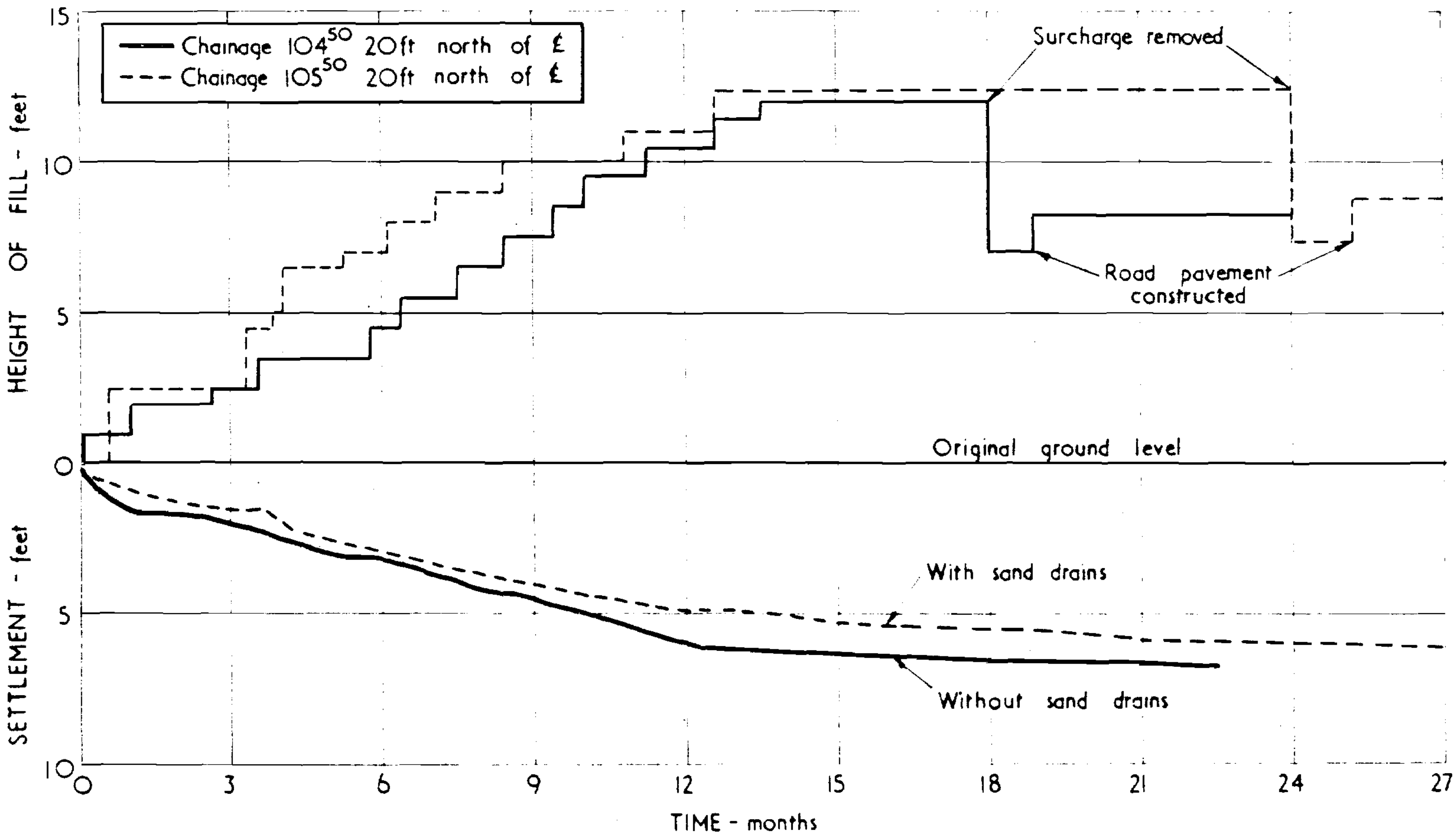

Fig. 6. SETTLEMENTS OF EMBANKMENT WITH AND WITHOUT SAND DRAINS AT CONDORRAT, DUNBARTONSHIRE (ORIGINAL DEPTH OF PEAT - 18ft.)

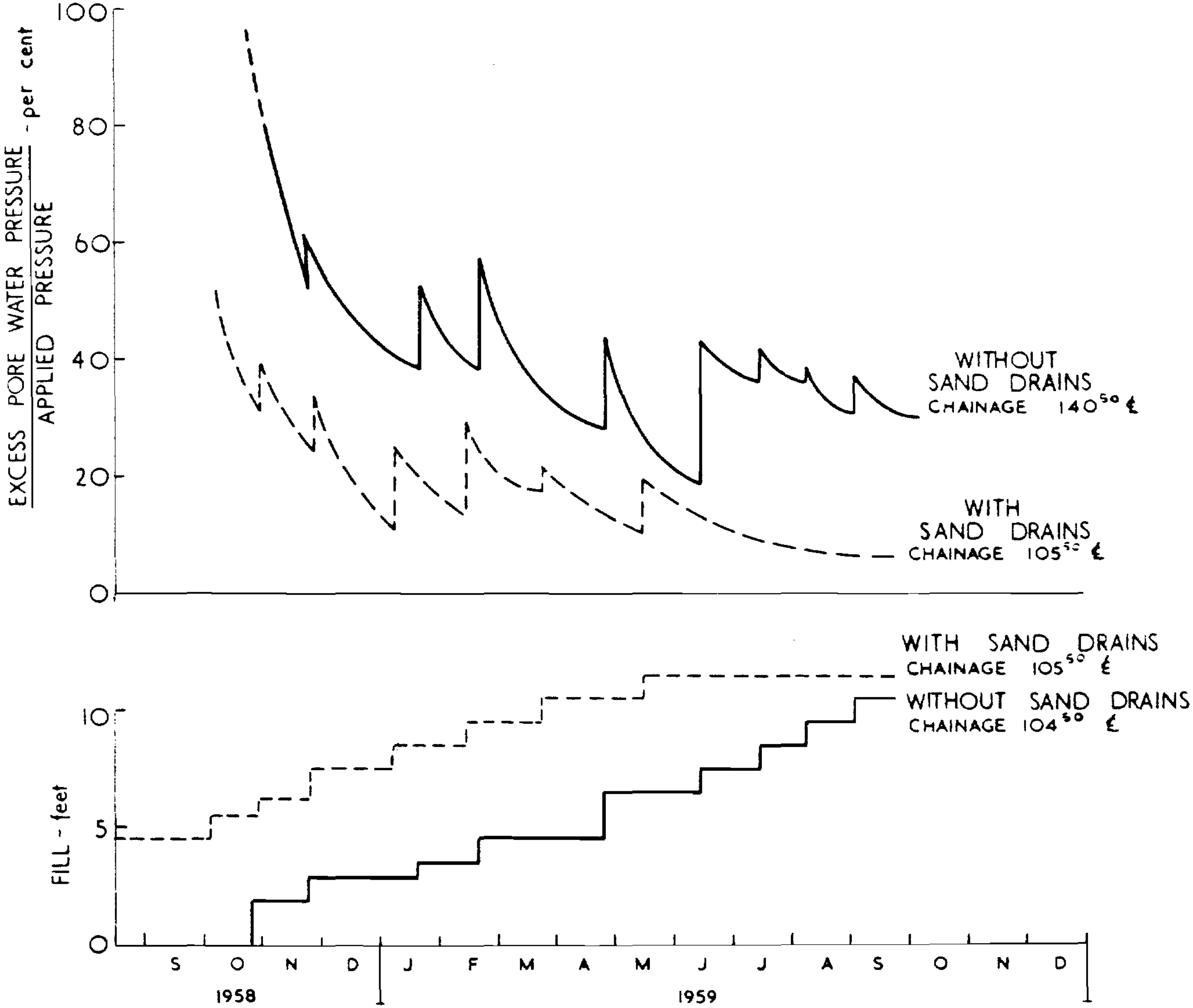

Fig. 7. EXCESS PORE-WATER PRESSURES EXPRESSED AS A PERCENTAGE OF APPLIED FILL

III. 2 ASPECTS OF RESEARCH AND DEVELOPMENT OF ROADS OVER ORGANIC TERRAIN IN JAPAN

Report prepared by I. C. MacFarlane

Japan, like Canada, has a muskeg problem. This problem is most acute in the northern island of Hokkaido which has organic terrain over some 775 square miles, or 23% of the total area. The muskeg occurs chiefly in the low-lying plains of the river valleys. The problems of road construction in these areas are aggravated in many instances by the presence of deep deposits of soft clay underlying the organic material, by heavy snowfalls, by cold weather and the accompanying frost action problem, by lack of suitable granular material for backfilling, and also by danger of flood damage due to inadequate protection from the rivers.

In addition to road building, many other difficulties have been encountered in the course of development of organic terrain in Japan. Land reclamation by dyking to prevent flooding is hampered by unavailability of appropriate fill material and by existing dykes being built of peat on a peat foundation with the resulting possibility of seepage. Agricultural projects such as construction and maintenance of drainage ditches present their own problems. Heavy structures, such as bridges and pumping stations, constructed on muskeg have required very deep foundations, the underlying soft clay stratum adding to this difficulty. Drainage of these areas, or pumping water from them, has resulted in subsidences which have had rather disastrous results. Trafficability in general and the operation of construction equipment over these areas is a matter also under investigation.

Research into the engineering characteristics of peat has been underway in Hokkaido for several years, initially being carried out at Hokkaido University, but since 1953 it has been conducted at the Hokkaido Development Bureau under the direction of Dr. Isamu Miyakawa. Dr. Miyakawa and his colleagues have published a number of reports in Japanese. His most recent report, however, was in English and is entitled "Some Aspects of Road Construction over Peaty and Marshy Areas in Hokkaido, with Particular Reference to Filling Methods" (1). This report has created considerable interest in Canada due to the similarity of the problem in Japan to that in Canada and also due to the fact that the theoretical approach being developed by Dr. Miyakawa can doubtless be usefully applied to the design and construction of roads over muskeg in Canada. Consequently, a resumé of Dr. Miyakawa's paper has been prepared for information purposes.

In Hokkaido, peat is defined as "a soil which has been produced over many years by natural deposition of more or less decomposed remains of plants under conditions of low temperature and high humidity, and the organic matter content of which is 50% or more". Peat is classified more or less after the German fashion, into three categories: low peat, intermediate peat and high peat, which in turn are subdivided according to the plant species which make up the peat (based largely on surface cover). This classification system is admitted to be inadequate in that it bears no relationship whatever to the engineering properties of peat, which are related to the degree of decomposition, the organic matter content, density, etc.

The substratum is usually clay, often with an intermediate layer of fluid organic "mud". For instance, in the Ishikari District (west part of island) most of the peat bogs are 5 to 6 metres (16 to 19 ft.) deep, with some as deep as 10 metres (32 ft.); the peat is highly decomposed and is underlain by clay. On the other hand, in the Kushiro District (east part of island), the peat bogs are generally 2 to 3 metres (6 to 10 ft.) deep, with a low degree of decomposition, underlain by sand, and with considerable "mounding" at the surface. The water level generally is within 10 to 70 cms. (4 to 28 ins.) of the surface.

The engineering properties of peat are discussed in some detail. In summary, the vane shear strength of peat is within the range of 0.05 to 0.25 kg/sq. cm. (102-510 p.s.f.) and in most cases is around 0.1 kg/cm (205 p.s.f.). The density is extremely low, about 1 gm/cm.^{3} and - due to the high moisture content, ranging from 3 to 10 times the dry weight - the dry density is within the range of 0.08 to 0.25 gm./cm.^{3} (5.0-15.5 p.c.f.), and generally is around 0.1 to 0.2 gm/cm.^{3} (6.3-12.5 p.c.f.). Organic matter content varies from 50 to 95%.

A good correlation has been found between the penetration index (P) of a cone penetrometer and the shear strength (S) determined by a vane tester. This relationship is: -

$$P = 10\,S$$

Peat is noticeably anisotropic. The shear resistance (measured by the vane) in the vertical plane is approximately twice that in the horizontal plane.

Permeability varies widely depending upon the amount of mineral matter in the soil, the degree of consolidation, and other factors. In the Kushiro District, the permeability coefficients of peat are $(2\text{-}13) \times 10^{-3}$ cm./sec. in the horizontal direction and $(2\text{-}7) \times 10^{-3}$ cm./sec. in the vertical direction, and an anisotropy ratio of approximately 2. In the Ishikari District, the permeability coefficient of peat is $(6\text{-}50) \times 10^{-5}$ cm./sec. in the horizontal direction and $(2\text{-}7) \times 10^{-5}$ cm./sec. in the vertical direction and with an anisotropy ratio of 3 to 6. The comparative impermeability of a peat layer consisting of more than 80% water by volume is a paradox, but possibly can be attributed to the effect of the colloidal properties of the decomposed organic material.

In the consolidation testing of peats, in addition to the complex phenomena found in clays, there is the additional factor of colloid action in a highly compressible organic framework containing large quantities of water. Consequently, the time-settlement phenomena will be somewhat different from the Terzaghi theory and settlement will continue over a long period of time. While the one-dimensional consolidation test on peat has some limitations, such tests were carried out on Ishikari peats. A straight-line relationship is shown to exist between compression index (C_c) and the moisture content as well as with the void ratio.

In view of the origin and development of peat, it may be expected that a correlation will exist between various characteristics of peat such as density, moisture content, ignition loss, shear strength and consolidation characteristics, particularly below the water table. An analysis of physical properties has resulted in the relationship:-

$$f = \frac{w}{n} \quad \frac{\text{(water content)}}{\text{(Ignition Loss)}}$$

The ratio "f" varies with the history of consolidation of the peat and it is suggested that if "f" is equal to or greater than 10, then the peat is normally consolidated; if it is less than 10, it is over-consolidated. In addition, vane shear and cone penetrometer index show good correlation with "f". For values of "f" less than 10, there is generally a decrease in the resistance as "f" increases. When $f = 10^{+}_{-}$, there is no variation in the resistance index and this is presumed to be a layer of virgin peat.

It is suggested that since there is a straight-line relationship between compression index and water content and void ratio, and if the pre-consolidation pressure can be estimated from the moisture content: ignition loss ratio (in a manner described by the author), then it is possible to infer the amount of settlement of a peat layer due to consolidation simply by measuring the moisture content and the ignition loss.

The present roads in Hokkaido have evolved from primitive trails built many decades ago. A comprehensive development programme of the whole area will result in an improvement of not only the roads but in the drainage channels and the river protection system as well. In 1955, an engineering analysis was made of some of the roads in the vicinity of the Ishikari river system. The purpose was to develop a standard for future planning, design and construction based on information gathered from studying the characteristics of existing roads over peat bogs and contrasting them with natural ground, taking into account the traffic loads on existing roads.

The structure of, and materials comprising, the road cross-sections were examined. Shear and penetration tests were carried out beneath the roadway and in the natural ground. Both were considerably higher beneath the fill, for a considerable depth. Plate loading tests were carried out on the road surface and at each layer comprising the fill. A circular loading plate was used, 30-ins. diameter for the gravel layer and 50-ins. for the peat layer. Results showed that the coefficient of bearing capacity of the gravel layers was 7-10 times that of the peat layers.

A review is given of the various methods of road construction over organic terrain. It is pointed out that the construction method will depend a great deal on local conditions and consequently a careful survey must be carried out before an intelligent choice of method can be made. The pros and cons of side ditches for roads floated on muskeg are discussed.

Existing roads in Hokkaido are generally of the floating type. New roads are aligned along the existing road when possible to take advantage of the increased stability of the peat under the existing road. Excavation and replacement is very rarely used except occasionally where the mineral substratum is sand or a firm clay. The fill is usually built to 1.0 to 1.5 metres (3 to 5 ft.) above grade which requires a total of 1.5 to 3.0 metres (5-10 ft.) of fill to compensate for the settlement which will occur. A granular fill is essential to eliminate the danger of frost heave. For minor roads, raft foundations are sometimes used, utilizing corduroy, fascines or wire mesh. However, the usual method at present for low fills is stage construction (so that as much as possible of the settlement will occur during construction) or the use of counter-balancing side berms. When the foundation is exceptionally soft, the construction period is limited, or a high fill is required, vertical sand drains are used. The author is doubtful, however, about the ability of sand drains in peat to accelerate settlement. He suggests that the use of the preloading technique for accelerating settlement will be more prevalent in the future.

The Hokkaido Development Bureau has been studying the problem of consolidation of peat since 1953 and has issued a number of reports. As has already been pointed out, tests indicate that from a fairly early stage settlement takes a straight line relationship with log-time and lasts for an extremely long period of time. Since long-term testing was impractical, routine 24-hour consolidation tests were used to calculate settlement. In several cases the laboratory values were seen to check closely with the observed field values.

A coefficient of secondary compression "C_s" is developed equal to: -

$\frac{b}{H}$ where,

b = rate of settlement of secondary phase for one cycle of log t.

H = thickness of compressible layer under fill.

C_s is shown to increase in a straight line relationship with the consolidation pressure up to the pre-consolidation pressure, after which it begins to level off. It is concluded that this is proof that the surcharge (preload) method of construction is effective for decreasing the effect of secondary consolidation.

In view of its current wide use in Japan, progressive stage construction is discussed in some detail and a detailed quantitative analysis is presented of the increase in shear strength of the peat and soft clay upon the completion of a stage, with a method of estimating when the next stage should commence and the height of fill to be used. Peat is shown to have a much greater increase in strength following consolidation than does clay. A detailed mathematical analysis is also given of the function of counter-balancing side berms. It is shown that side berms not only lessen the danger of lateral shear failure, but also tend to accelerate settlement of the fill.

The report concludes with a review of the many problems encountered in the construction of a road over organic terrain and emphasizes that the final solution to many of these problems has not yet been found and that much fundamental research remains to be done.

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REFERENCES

1. Miyakawa, Isamu. Some Aspects of Road Construction over Peaty or Marshy Areas in Hokkaido, with Particular Reference to Filling Methods. Published by Civil Engineering Research Institute, Hokkaido Development Bureau, Sapporo, Japan. June 1960, 54 pp. 29 ref.

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III.3 ROAD CONSTRUCTION FOR TRANSITIONAL PERMAFROST ZONES AS TYPIFIED BY THE HAY RIVER AREA OF THE NORTHWEST TERRITORIES

J. J. Wallace

The purpose of this paper is to relate some conditions, observations and techniques affecting road construction in transitional permafrost country. The Department of Public Works has, in the past five years, constructed development roads around Great Slave Lake as part of its programme in the Northwest Territories. These roads are indicated in Fig. 1.

At present, the Department of Public Works is engaged in rebuilding the Mackenzie Highway from the Alberta-Northwest Territories border to Hay River. Most of this paper will concern the Northwest Territories section of the Mackenzie Highway. This section stretches from the Alberta-Northwest Territories boundary to Hay River on the south shore of Great Slave Lake and is approximately 80 miles in length. Geologically, about one-quarter of the area is recent lake sediment underlain by Hay River shale and a large part of the remainder is an area of drift underlain by Hay River limestone.

The area lies in a zone which is transitional from permafrost to non-permafrost. In different parts of this area permafrost may either be quite predominant, may exist occasionally or may not exist at all. Consequently, permafrost is a problem in this area as it may advance or recede. Because this frost appears and disappears, there is some doubt as to whether it is truly permafrost or just some form of seasonal frost. Dr. Radforth has called this type of frost "climafrost". However, the more general term "permafrost" will be used for ease of reference, even though in some cases the terminology may be incorrect.

In the building of development roads in the Hay River area, conditions are encountered which do not exist in southern areas or even in far northern areas, because it is necessary to contend with continual changes in permafrost conditions. The mere existence of permafrost is not in itself a major road building problem; the problem is maintaining stable conditions.

The Department's work in the Northwest Territories consists of building development roads and related structures. The purpose of these roads is to provide access to areas which warrant development. The roads will be improved or rebuilt from time to time as the areas grow. The standards of development roads are not rigidly detailed but are tailor-made for each individual project, taking into consideration economy, topography, and the type of development. For example, roadway widths vary from 15 ft. for the Yellowknife East construction to 32 ft. for reconstruction of the Mackenzie Highway.

Generally speaking, these development roads must be constructed to carry a light density of very heavy loads. These roads are built where soils which will underlie embankments are poor and not well-drained, and good borrow material may be difficult to find, or at times even non-existent. In many other parts of the country, methods such as stripping of organic soil, underfill blasting, or chemical stabilization are sometimes used. However, in the Hay River area where permafrost is transitional, much consideration must be given before using these methods. It must first be known what conditions exist. These conditions are determined by aerial photographs, by ground observation of vegetative cover, by observing topography and moisture content of the soil, and always followed up by digging test holes. It has been found that in the Hay River area permafrost may be encountered from depths of immediately below the surface to possibly 30 ft. However, since the stability of the permafrost is the problem and not just its existence, concern is only with permafrost at working depths. When permafrost is anticipated at working depths, special treatments are required.

For this special treatment a clause is written into the contract, called "Special Clearing". Special clearing consists of cutting trees, brush, etc., on the right-of-way, within 8 inches of the ground surface and placing them in a flattened uniform layer on the embankment areas. The original intention was that this special clearing would be performed on permafrost areas only, to insulate the permanently frozen soil against thawing and thereby maintain it in its stable state. However, the special clearing mat serves other purposes. It gets rid of clearing material which would normally have to be disposed of by burning or burying, it distributes the pressures of the embankment over the underlying soils, and it also provides a working pad for movement of construction equipment. Therefore, special clearing is now being done on low-lying, wet, muskeg areas as well as on permafrost areas. This special clearing could probably be compared to so-called corduroy. However, with corduroy the logs are placed side by side in a flat, uniform layer, whereas with special clearing trees, roots, stumps, brush, etc., are thrown into the flattened, uniform layer. Therefore, an entangled mat is obtained which is not as rigid as the corduroy but does provide better insulation and is not disturbed nearly as much by differential settlement.

This special clearing is paid for on the basis of number of acres cleared and it has been found that the prices for special clearing differ very little from the prices for ordinary clearing. Special clearing decreases the amount of embankment material required since it lessens the danger of thawing of the permafrost and decreases the amount of differential settlement. Therefore, the use of special clearing decreases the cost involved in constructing embankments capable of carrying the loads.

Right-of-way drainage in the Great Slave Lake area is sometimes a problem and must be given careful consideration. In the first place, water lying on a permafrost area will cause the permafrost level to recede. This belief is substantiated by the fact that, generally speaking, permafrost levels in riverbeds are lower than they are in the banks adjacent to the rivers in both permafrost and transitional permafrost areas. Drainage in the Hay River area is a very slow, seeping procedure as drainage basins have very small gradients and the drainage is further retarded by hummocks, small depressions and terraces. Ditching in these areas should be done immediately after clearing and before any other earth work is carried out. This pilot ditching should be done so that the soil under the embankment will be as low in moisture content as possible while the embankment is being placed. Placing of the embankment will squeeze additional water out of the underlying soil, including the organic cover; but if the pilot drainage system has previously been constructed it will carry away the additional water. Therefore, with some fluctuations there is achieved a continual reduction in moisture content of the soil underlying the embankment in advance of and during construction. Consequently, more of the settlement is obtained during construction and less after construction. This gives a more stable grade immediately after construction in both permafrost and non-permafrost areas. Also, by keeping the moisture content to a minimum the possibility is decreased of water, which might thaw the permafrost, lying adjacent to the grade.

The excavation of roadway side ditches themselves must be carefully considered. It is interesting to consider what might happen if the standard turnpike ditch were dug in a permafrost area.

Digging this ditch would not only remove the natural insulation cover but would permit water to flow over the soil and possibly directly on the permafrost. This would cause the permafrost to thaw and therefore a portion of the fill would be resting on the supersaturated soil having little or no bearing capacity (see Fig. 2). Part of the embankment would probably slough into the ditch and additional material would have to be moved as a repair measure. Even if permafrost did not exist, digging ditches such as this would remove the lateral support which the material in the area of the ditch would develop against the soil under the embankment. Removing the ditch material might not cause serious failure but there would be a tendency for the embankment to move into the ditch. Therefore, more embankment material would have to be provided to allow for this sloughing effect.

In low-lying, wet muskeg areas, as well as in permafrost areas, a different type of side ditch should be constructed.

Since contractors usually find that this ditch is most easily constructed with a dragline, it is called a "dragline ditch". The ditch is dug with its closest edge about 20 ft. from the toe of the embankment. After the ditch is dug, the permafrost level in the vicinity of the ditch will usually recede, as shown in Fig. 3, due to the thawing of the permafrost. This, however, is of no immediate concern since drainage has been achieved and the soft, wet zone caused by the thawing permafrost is not in the area of the embankment. In low-lying, wet areas where permafrost does not exist, this type of ditch is still very effective. The lateral support of the soil under the toe of the embankment is not lost as it is with the turnpike-type ditch.

The material excavated from this ditch is sometimes cast outside the ditch in a flattened, uniform windrow or it is placed in a windrow inside the ditch and after the embankment is placed the waste material is trimmed up against the embankment as a berm (see Fig. 4).

If the natural ground underlying the embankment has a gentle slope, the change in the permafrost level may obstruct subsurface drainage and cause water to lie in the area of the embankment. However, placing of a berm adjacent to the embankment would run the water off into the ditch. The berm will also help to raise the permafrost level and make subsurface drainage conditions better. Placing this berm, however, has some drawbacks inasmuch as it is difficult to place without disturbing the natural insulation. An impervious berm should not be placed on the downhill side of a pervious fill for it will just keep water in the embankment. In fact, an impervious berm adjacent to any pervious embankment may just tend to trap water in the embankment and this is undesirable. Therefore, it is evident that some consideration must be given to whether the material excavated from this dragline ditch should be used as a berm or just wasted.

Culverts are a very important part of any drainage system and installing culverts in muskeg and permafrost country presents many special problems. To install a culvert on the existing ground surface without disturbing the underlying soil might seem like the answer to avoid disturbing the stability of the permafrost. This method would present some problems; for instance, ditch grades and culvert grades are not easy to adjust in this flat country and the necessary culvert grades would seldom match the existing ground. The bearing capacity of the bed under these conditions would not give proper support to the culvert and the pipe would probably fail structurally during settlement. Also, water flowing through the pipe might thaw the permafrost. The best solution then is to sub-excavate to some depth, and backfill as quickly as possible with compacted granular. In muskeg the required increase in bearing capacity is obtained and capping the ends of the bed and slope of the fill with impervious material helps to keep the seepage through the bed to a minimum. Where permafrost exists, the granular material must be placed before any thawing starts. The granular material will usually provide sufficient insulation to keep thawing to a minimum. In fact, if the granular material extends well below the permafrost level, it is believed that the permafrost level may even rise into it after the pipe and the embankment have been placed. Further north than the Hay River area and in the zone where permafrost is more predominant, the bottom of some culvert excavations have been lined with moss and peat before placing the granular bed. This is to try to restore the natural insulation; so far it seems to have been successful.

The slow seeping drainage from muskegs sometimes presents some very interesting problems. For example, the ice on a creek may freeze on the surface long before the creek freezes to the bottom, which creeks do in this area. A creek may freeze to the bottom in the area of a structure first, due to the fact that the velocity is usually slowed immediately upstream from the structure. However, the water will still be moving from the muskeg drainage basin long after this creek is frozen to the bottom, due to the insulation of the heavy snow cover over the muskeg. The water which is moving out of the muskeg will come up through the ice in the form of springs of slush and will freeze there. The final elevation of the top of this slush ice is generally fairly consistent and it is believed that the elevation is governed by the hydrostatic head of the water back in the muskeg. This winter, some experiments are being carried out by building tar paper fences to try to move this ice formation back upstream on one creek in the Hay River area. It is felt that if a constriction can be developed in the stream upstream from the structure, possibly the ice can be caused to form in an area where it will do no harm. However, the results of this experiment are not yet available. On the particular creek where the experiments are being carried out, one culvert will be installed at the invert elevation of the ice level and two smaller culverts at lower elevations to carry the normal run-off. It is believed that these three culverts will carry the run-off just as well as the bridge type structure, at one-quarter of the cost.

It may be wondered if in this area where a large amount of the road is embankment built from borrow, trouble with permafrost is experienced in borrow pits. In the Hay River area, permafrost may occasionally be encountered in the borrow pits but, where possible, this material is abandoned. To use permafrost material from a borrow pit, it is necessary to let a thin layer thaw. Drainage and evaporation will reduce the moisture content. The drier material is then excavated and another layer is allowed to thaw. The material must be spread in the proposed embankment in thin layers and allowed to dry to approximately optimum moisture content before compaction and placing of another layer. If at all possible, the use of permafrost material over special clearing should be avoided as permafrost material will not provide the desirable amount of bearing capacity required to avoid disturbing the special clearing mat. Normally, over special clearing, a fairly heavy lift of good material is placed to carry construction equipment without disturbing the special clearing. However, placing the wet permafrost material in this heavy a layer would not be desirable.

In conclusion, the problem is primarily to build roads which carry a low density of heavy loads over muskeg in an area where permafrost is transitional. Economy does not allow excessive stripping or large embankments on these development-type roads. Following are some general ideas which should be considered when building roads in this or any comparable area:-

1. Thoroughly investigate conditions and tailor-make construction to suit the conditions.

2. Disturb the natural conditions as little as possible.

3. Achieve distribution of pressures over the natural soils.

4. Construct efficient drainage.

5. Determine what materials and equipment are available locally and incorporate as much as possible of these in the construction procedures.

These are just some of the many broad statements that can be made. The refinements of these statements are slowly being evolved. As more knowledge is gained, it is hoped that some day the information will be compiled in textbooks and available to all who are interested in construction in this or any similar area.

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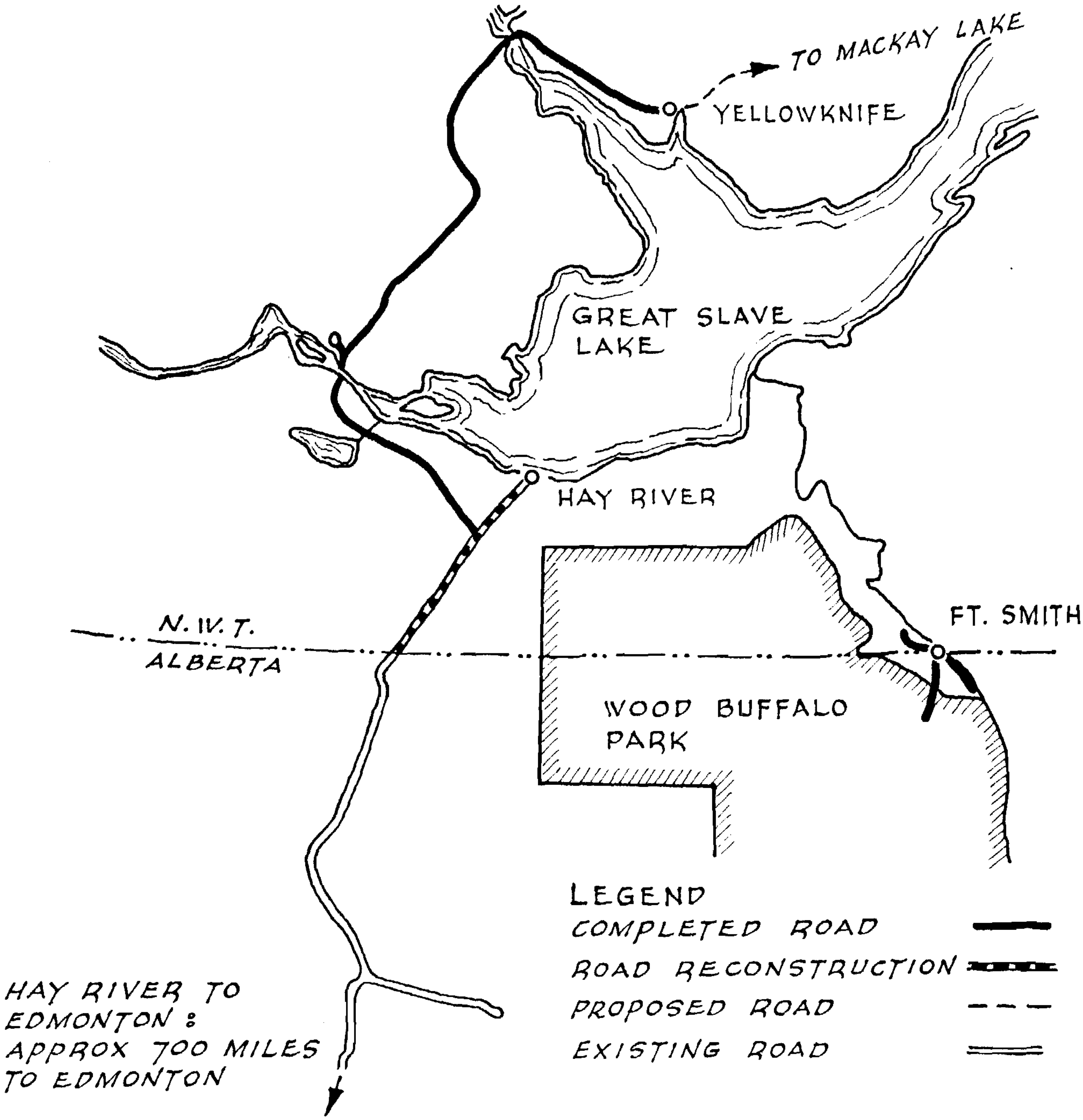
TO MACKAY LAKE
YELLOWKNIFE
GREAT SLAVE LAKE
HAY RIVER
FT. SMITH
N.W.T.
ALBERTA
WOOD BUFFALO PARK
LEGEND
COMPLETED ROAD
ROAD RECONSTRUCTION
PROPOSED ROAD
EXISTING ROAD
HAY RIVER TO EDMONTON: APPROX 700 MILES TO EDMONTON

FIGURE 1

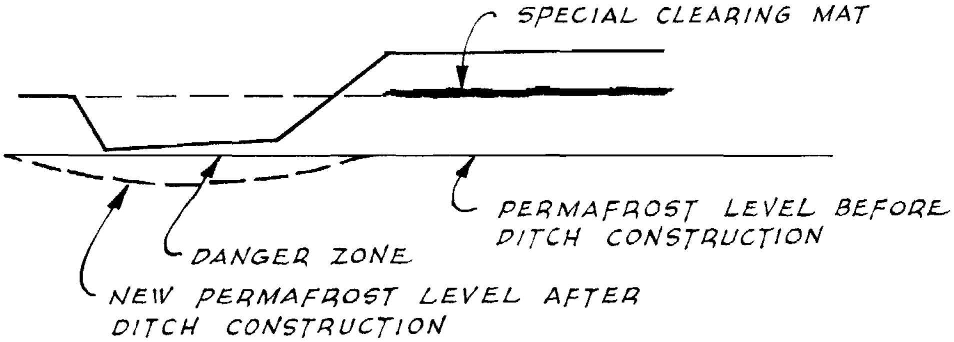

FIGURE 2 TURNPIKE TYPE DITCH

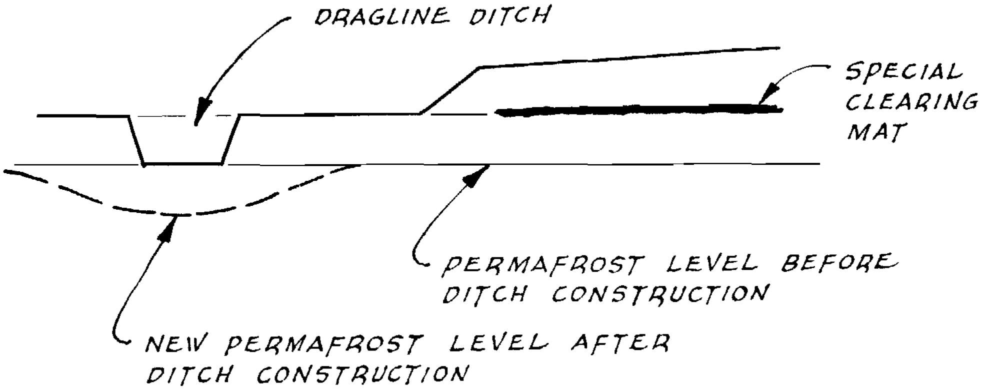

FIGURE 3 DRAGLINE TYPE DITCH

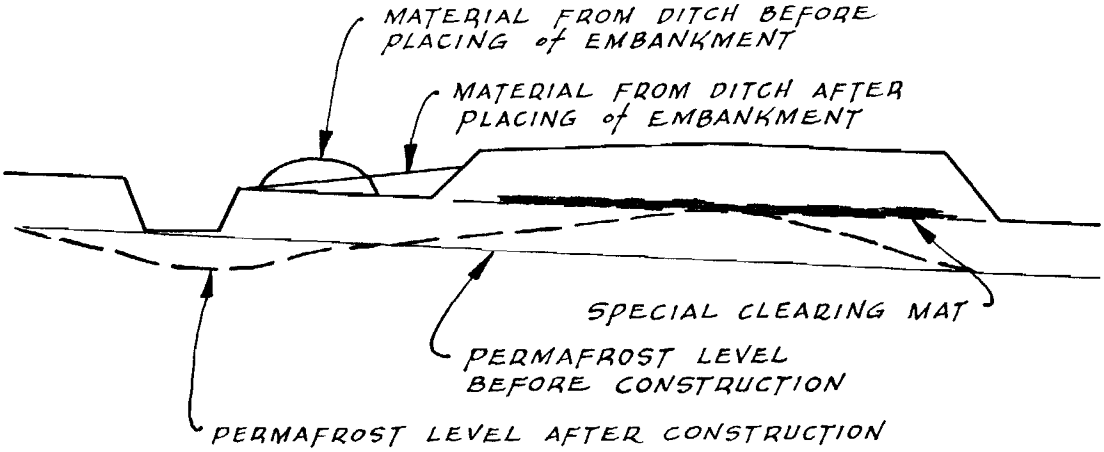

FIGURE 4 BERM CONSTRUCTION FROM MATERIAL EXCAVATED FROM DRAGLINE DITCH

Discussion

Mr. Pihlainen (Arctic Consultant) enquired about the cost of road building in the Hay River district. Mr. Wallace replied that there is such a wide variation in the roads that it was a difficult question to answer. A 15 ft. wide road in Precambrian Shield country costs about $15,000 per mile; in the Mackenzie River area it is $20,000 to $30,000 per mile. Mr. Davis (Spruce Falls Power & Paper Company) commented that they have built 12 ft. roads for as low as $3,700 per mile. For roads over muskeg, the top cost has been $20,000 per mile.

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III. 4 EMBANKMENT CONSTRUCTION IN MUSKEG AT PRINCE ALBERT

B. W. Mickleborough

As a result of increasing traffic volumes on the Provincial Highway system, the design and construction of high class, modern facilities has been required, especially in the approaches to major urban centers. In the northern part of the agricultural area of Saskatchewan, the construction of highways frequently involves the problems of building a grade through muskeg.

The component difficulties of the general problem are all too familiar. In brief, it is often difficult to obtain satisfactory highway performance for a minimum capital and maintenance cost to the road user.

The experience in the Department of Highways is not extensive where construction in muskeg is involved. This is particularly true for the case of primary highways.

In the past, when roads carried lighter loads and lower traffic volumes, the embankment height above muskeg surface was kept to a minimum. No special precautions were taken for drainage, prolonged settlement and excessive pavement deflections. Shear failure was usually unimportant because the fill heights were low. Displacement or part excavation and displacement methods were used. In few cases was the expensive complete excavation method employed.

The selection of the best embankment construction method for any particular muskeg problem was a difficult decision. Experience gained through better understanding of the principles of muskeg performance should make such decisions less difficult in the future. Design and construction work in Saskatchewan muskegs during 1960 has contributed to the experience of our Department. Of major importance is the information gathered from a primary highway grading project which crossed two muskegs near Prince Albert, Saskatchewan.

Muskeg Location

Highway traffic volumes at Prince Albert had increased to the point where a new major river crossing was necessary. This structure required modern facilities to handle the traffic in the approach areas. The new bridge features dual-lane traffic in each direction with barrier median separation. The north approach required the joining of traffic from two primary highways and the crossing of a single track line of the Canadian National Railways. The railway crossing is an overpass. A trumpet interchange handles traffic at the new junction of Provincial Highways 2 and 3. North of this interchange, Provincial Highway No. 2 is reduced to a two-lane facility.

The design and construction of this project presented a major undertaking for the Department of Highways. Two muskegs along the new location added a major foundation problem which affected the overpass structure and over one mile of embankment. The deeper muskeg occurred between the interchange of Provincial Highways 2 and 3 on the north, and the new traffic bridge on the south. The problem was made more serious by the high fill height required near the overpass. The second muskeg, about two miles north of the traffic bridge, was not as deep. Only the standard grade line height was required at that location. (See Fig. 1).

Muskeg Investigation

The Prince Albert area is underlain by the Alberta and Lea Park Shales of the Upper Cretaceous Period. There are no Tertiary deposits. The area is covered with glacial drift. These Pleistocene materials are silty and sandy glacial lacustrine or alluvial deposits. The soil is uniformly graded. No glacial till was found between the Upper Cretaceous shale and the surface deposit.

The two muskegs have formed on this basin of silt and sand. They are well developed as evidenced by the relatively thick tree growth. The swamp spruce grew up to 6 ins. in diameter. There was little open water or areas with only short grass and other low vegetation.

By the Radforth Classification System, the Coverage Class would be BEI and BFI. The main topographic features of the deep muskeg are the two boundaries. The north boundary is a high sand ridge at the interchange of Provincial Highways Nos. 2 and 3. The south boundary is the North Saskatchewan River. It is a braided stream, confined to the channel during flood periods. This muskeg is elevated above the river. There are no hummocks, mounds, boulders or other ridges. The trees were generally overlapping and sometimes impassable. The north muskeg had formed on a poorly drained slope.

The peat category would be woody fine-fibrous peat held in a woody coarse-fibrous framework. Peat classified as amorphous-granular was also observed.

Test borings were made by hand augers. One hole was used to obtain visual classifications, consolidation test samples and to determine the depths of the various materials. The consolidation samples were taken in 3 in. diameter thinwall spoons. The spoons were driven with a 30 lb. drop weight. Observed recovery ratios indicated the total disturbance was not excessive except in the very coarse peat. However, it was obvious that larger spoons were necessary - probably in the 6 or 8 in. diameter ranges. Also, an improved technique for advancing the spoon would reduce disturbance. The usual precautions were observed in waxing and transporting the spoon samples from the field to the testing laboratory.

The consolidation samples were trimmed to a specimen size 2 1/2 ins. in diameter and 1 in. thick. In some tests the new load increments were applied every 24 hrs. In others, longer loading intervals were used. In all cases, the initial load was 1/8 Kg/Cm^2. Each new load increment was double the previous load. The maximum pressure was 4 Kg/Cm^2.

The shear strength of the peat and the clay was determined by in situ vane shear tests. Difficult access conditions in the summer prevented the use of a refined type of test utilizing casing and guide bearings on the vane rod. The vane size used was either 3 5/8 ins. O.D. x 6 ins. long or 2 1/2 ins. O.D. by 4 1/2 ins. long. A torque wrench was used to rotate the vane. The larger vane was used unless the shear strength exceeded the limit of the torque wrench.

The borehole was cleaned out as well as possible with the auger to a depth one foot above the desired vane test depth. The vane was pushed into the material and rotated. The vane was then removed, the hole was cleaned out to the next depth and so on.

Test holes showed considerable variation in the soil profile, particularly on the south muskeg. This made it difficult to select representative cross-sections or critical cross-sections to use for stability and settlement analyses.

The south muskeg was about 1/2 mile long. The high fill for the four-lane highway was to be built over peat that varied from 0 to 8 ft. in depth. Under this, there was usually light silty material from 2 to 6 ft. thick. Some test holes indicated soft clay under the silt, presumably the soft upper layer of the marine shale. The soil profile became more complex when other holes showed fine sand to an undetermined depth below the silt.

The north muskeg presented a more favourable situation because the maximum peat depth was about 6 ft. The peat was underlain by sand to an unknown depth. This deposit was 2,100 ft. long.

The water table in both muskegs was usually within a foot of the peat surface.

Test Results

In the laboratory, the peat samples were tested for specific gravity and initial moisture content. Void ratios and degrees of saturation were determined from the consolidation samples. Both muskegs were mainly normally consolidated deposits because of the water table elevation. The one exception was that portion of the south muskeg between the Canadian National Railways' overpass and the traffic bridge. The water table in this section was at least 5 ft. below the surface at the time of construction.

The pressure-void ratio curves for peat samples showed negligible preloading. This would indicate that compression during sampling was not serious. During the consolidation tests, trouble was encountered with tilting of the loading cap. This was caused by differential settlement of the sample.

Typical test results are shown in Table I for both muskegs. Table II shows the average results of vane shear tests in the south muskeg. The specific gravity of the peat ranged from 1.6 to 1.8. Field densities varied from 60 to 80 p.c.f. and dry densities varied from 2.5 to 21 p.c.f. Natural moisture contents did not exceed 480%. The degree of saturation was in the 80% to 100% range. The values varied widely. The north muskeg had higher shear strength values than the south muskeg.

Design

A. South Muskeg -

The design problem in the south muskeg was approached from the viewpoints of stability and settlement. Minimum gradeline heights were controlled in the order of 27 ft. at the overpass, 17 ft. near the traffic bridge and 14 ft. at the northern muskeg boundary. To accommodate the four-lane pavement structure, the required subgrade width was about 80 ft. The fill was to be built with sand from the ridge in the vicinity of the trumpet interchange. This borrow was of good quality. It classified as A-3 and A-2-4 by the Highway Research Board system or SW, SP, SM by the Unified System.

The design possibilities at this stage were complete excavation, part excavation and displacement, normal consolidation or preloading.

From the test results, two typical cross-sections were selected for the stability and settlement analyses (see Figs. 3 and 4). Due to the variability of the muskeg profile, it was difficult to assess the most critical section. The section used for stability was as follows:-

Embankment	- 28 ft. high, 2:1 sideslopes, 80 ft. top width, density = 105 p.c.f., cohesion = 0, internal friction angle = 30°, Sand.
Foundation	- Water table at surface. Peat - 0 ft. to 3 ft., c = 300 p.s.f., density = 70 p.c.f. Silty clay - 3 ft. to 9 ft., c = 600 p.s.f. density = 70 p.c.f. Soft clay - 9 ft. to 13 ft., c = 800 p.s.f., density = 100 p.c.f. Hard shale - below 13 ft.

The foregoing values gave a conservative safety factor. All possible methods of failure were examined. The minimum safety factor was 1.2, based on the full fill height with no increase in shear strength from consolidation. This satisfied the critical condition for stability, provided that pore water pressure did not increase to the point where shear strength was inadequate.

The possibility of preloading the fill was also examined. The result was that a two-stage operation might have been necessary to avoid shear failure caused by high neutral pressures. Because the design fill was high, the required surcharge would also be high. The available time for completion of the surcharged fill was a significant factor because the surfacing contract was scheduled for the next summer. This left little time for the preloading and removal of excess surcharge. Another factor was the disposal of the removed surcharge. The design sideslopes were 2:1. It was not safe to build the initial sideslopes steeper than this for the surcharged fill. Therefore, the excess surcharge could not be used to widen the embankment slopes to the design slope.

Complete excavation of the peat was a very expensive venture. It was undesirable because if the peat were removed, the underlying silt could have presented a more serious stability problem. Disposal of the large volume of excavated material would have been costly. The method of part excavation and displacement was abandoned because of similar reasons and greater risk involved if the end result was not as planned.

The decision was made to proceed with embankment construction by normal consolidation. The maximum settlement predicted was in the order of 6 ft. north of the overpass. It was estimated that the primary consolidation would be complete by the summer of 1961 if the fill construction was finished in the summer of 1960. The estimated settlements decreased toward the north and south boundaries to values of about 3 ft. and 2 1/2 ft. respectively, under smaller design fills. It was accepted that secondary consolidation would take place. Therefore, the surfacing structure placed in 1961 would be of a temporary nature. This will probably be a base course and a seal coat. Maintenance work would be done when secondary settlements became detrimental to the temporary pavement profile and cross-section. It was considered that this was the most economical procedure in view of the higher first costs, the risks and the delays involved in the other construction methods.

B. North Muskeg -

This deposit had a maximum depth of 6 ft. with a designed fill height of 4 to 9 ft. The peat was underlain by sand except for two clay pockets. Stability was not a problem. The fill material was sand. The peat shear strengths were higher than those in the south muskeg. With the smaller fill, and a controlled rate of fill placement, it was unlikely that shear failure would occur if preloading methods were used.

A normal consolidation construction procedure was undesirable. Since grade width was 44 ft. through this muskeg, the quantities involved in complete excavation were not as formidable as in the other deposit. Disposal of removed material was not a serious problem. In two locations, the fill height was less than 4 ft. Here, complete peat excavation, 44 ft. wide, with sand backfill was the only practical procedure.

In two locations where the fill height was 4 ft. or more above the peat, preloading was used. The surcharge thickness of 2 ft. was determined from consolidation tests. The initial fill height, equal to the design fill height plus surcharge, would cause a settlement greater than that of the design fill. This settlement would occur before the paving contract was let. The sideslopes of the initial fill were to be built at 1 1/2 to 1. When the preloading was complete, the excess surcharge was to be pushed on to the sideslopes to obtain the design slope of 4 to 1.

There was a possibility that differential settlement would occur at the transitions from the completely excavated sections to the preloaded sections. To minimize this movement, the transition from zero peat excavation to full depth excavation was made on a flat slope. In addition, the surcharge heights were doubled to 4 ft. at these grade points.

Construction

A. South Muskeg -

In order to be well informed on the conditions of settlement and stability during and after construction, it was decided to place piezometers, lateral movement posts and settlement plates in the muskeg. This was done in March 1960, after the right-of-way was cleared.

The settlement plates (see Fig. 5) were placed on the peat surface at centerline. The spacing interval was usually 200 ft. The plates were made by nailing together two layers of planks. A collar, bolted to the plank, held the lower end of a 5/8 in. diameter black iron pipe. The pipe led up through the sand fill. Extensions were added as the fill construction progressed. Levels on the top of the projecting pipe recorded the settlement of the foundation.

Wood posts, 2 ins. by 2 ins. by 10 ft. long served as alignment pins. These were driven 6 ft. into the muskeg every 100 ft. on either side of centerline, 20 ft. from the toe of the embankment.

These were tied in to a horizontal control system. Measurements of lateral movements served as an indicator of a possible shear failure.

Piezometers were placed in the peat and silty clay to indicate the magnitude of neutral pressures. This information, with the settlement plate readings, was very important in controlling the rate of fill placement.

The piezometer consisted of a porous stone tube connected to a stand pipe. The stone tube was enclosed by clean, fine sand in a wire and cloth cartridge. A bentonite seal was provided above the porous tube. The piezometers were placed in the muskeg by auger holes and by casing. The 5/8 in. diameter standpipe was extended through the fill (see Figs. 6 and 7).

Neutral pressure readings were obtained in the stand pipe by the use of an electrical measuring device. Piezometers were placed at the same locations as the settlement plates. Every 400 ft. there was a piezometer in the silt and in the peat. Halfway between these, there was a piezometer in the silt only.

When the pore water pressure in a piezometer reached 25% or 30% of the effective pressure, and the settlement plates showed relatively small movement, the earthmoving equipment was sent to another work area.

Construction commenced early in April 1960 while there was about 2 ft. of frost in the muskeg. In this way, the top of the peat layer was not disturbed during the critical operation of placing a small fill across the entire subgrade area. This working base was about 6 ft. high. With one exception, gradual settlement occurred and consequently neutral pressures did not exceed the allowable value.

The exception was at a location where the muskeg was deep. The fill base was wider than usual because of two interchange connections. Construction in the immediate vicinity was stopped when the piezometric pressure increased to double the allowable value. The settlement plate showed relatively little movement. These danger signs were confirmed when an adjacent piezometer showed a large neutral pressure increase. Approximately 250 18-inch diameter holes were drilled through the 6 ft. fill and the frozen peat to relieve the pressure. The holes were on 20 ft. centers. The piezometric pressures dropped and settlement took place.

Under careful control, the fill was brought up at the rate of about 2 ft. per week. The fill increases were always reflected in the piezometer and settlement plate readings. Settlement continued steadily. The neutral pressures did not exceed the allowable values for the remainder of the work. Lateral movements at the toe of the embankment were negligible.

At the overpass with the Canadian National Railways, the peat and silt were excavated to a depth of at least 10 ft. This was necessary because of the danger of shear failure at the spill-through abutment. The abutment cap was placed on piles driven through the end of the sand fill. If settlement of the foundation material created a drag on the piles, differential settlement of the structure could have occurred.

The excavation took the form of two trenches, parallel to the railway and on each side of it. Compressible material was removed until sand was exposed. The trenches were then backfilled with sand borrow. This provided a firm foundation for the piers, the abutment and the spill-through portion of the fill.

B. North Muskeg -

Construction began on the north muskeg in September 1960. No piezometers were used. Settlement plates and lateral movement posts were installed on sections that were to be preloaded.

The base of the fill was formed by bulldozing a 3 or 4 ft. sand blanket over the peat. An effort was made to place this layer as evenly as possible with minimum distortion or rupture of the peat surface. After the first layer was finished, the remainder of the surcharged fill was placed in equal lifts.

The sections calling for complete excavation were subcut to expose the underlying sand or clay and backfilled with sand borrow.

No special problems arose during construction in this muskeg.

Conclusions

A. South Muskeg -

Settlement has occurred steadily since the embankment was completed. At the south end of the muskeg, the settlement rate is decreasing after 1 1/2 ft. of movement. In the deepest area, settlements of 4 1/2 ft. have been measured. The rate is beginning to decrease. At the north end, secondary consolidation is taking place after 2 1/2 to 3 ft. of settlement.

Primary settlement magnitudes have generally been somewhat less than estimated from consolidation tests. The trend on the field time-settlement curves indicates that in the deep muskeg the most extensive maintenance work will be required in the first 3 or 4 years after the temporary pavement is placed. This was expected when the decision was made to use the normal consolidation method for construction.

There have been no indications of instability of the embankment since it was completed. Now that the critical stability period has ended, it is reasonable to assume that shear failure will not occur in the future.

B. North Muskeg -

Total settlements to date have been in the order of one foot or less in the shallow peat deposit. According to the settlement estimates, sufficient consolidation has taken place so that when the excess surcharge is removed, the design fill will rest on a preloaded foundation.

In the deeper area, settlements have reached the secondary consolidation stage. The observed settlement was about two-thirds of the estimated settlement. The excess surcharge will be removed this summer to complete the pre-consolidation procedure.

General

At the present time, the Department of Highways has found the foregoing procedures to be satisfactory for these two specific muskeg problems. The method of normal consolidation, usually avoided, was considered to be the most economical solution to the problem of constructing a satisfactory major embankment over a deep muskeg in a relatively short time. Future observations at this site will yield valuable information for the application of the pre-consolidation method at other locations. The project will serve to provide a rational assessment of the consolidation characteristics of peat.

The more desirable method of preloading has been used in a shallower muskeg. There, the results to date are also satisfactory.

It is impossible to draw any final conclusions from limited information now on hand. It is regretted that the complete case history cannot be reported. A proper evaluation must include performance characteristics and the performance record will not be available for some time.

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TABLE I

TYPICAL TEST VALUES

PEAT SAMPLES

Station	Depth ft.	Field Density p.c.f.	Dry Density p.c.f.	Water Content %	Init. Deg. of Saturation %	Initial Void Ratio e_o	Compressive Index c_c
11+00	1'	59.2	2.9	480	88	9.86	4.20
	2'						
	3'	66.7	2.5	404	98	7.48	3.90
20+00	1'	79.0	5.1	147	98	3.31	--
	3'	79.5	5.6	144	99	3.21	1.0
	5'	75.4	4.6	187	97	4.23	--
102+00	2'	58.7	17.0	246	80	5.40	2.90
116+50	4'	62.0	12.0	417	88	9.00	3.80
	5'	69.3	20.5	244	95	4.87	1.90
	2'	68.9	21.4	222	96	4.58	1.50
120+50	1'	65.6	15.2	332	96	5.58	2.40
	2'	59.5	12.8	364	85	7.26	2.80
	3'	61.0	11.6	426	89.0	8.17	

11+00 and 20+00 are in South Muskeg

TABLE II

SOIL PROFILE
VANE SHEAR TESTS

Depth	Sta. 5+00	Sta. 8+00	Sta. 11+00	Sta. 14+50	Sta. 17+00	Sta. 20+00	Sta. 23+00	Sta. 26+00	P. S. F.	Use
	p. s. i.	p. s. i.	p. s. i.	p. s. i.	p. s. i.	p. s. i.	p. s. i.	p. s. i.		
1	2.8				1.4	1.8	2.7		302	
2										Peat c = 300
3		9.0	7.6	2.0	1.9	2.1	2.7	4.5	645	
4				6.6			4.0			
5			5.4	6.6	2.1	2.5	4.0	3.4	625	
6							6.4			light clay
7			6.3		3.6	2.5	6.4	3.7	648	c = 600
8										
9						8.1		5.5	865	
										Soft clay to 13' c = 800

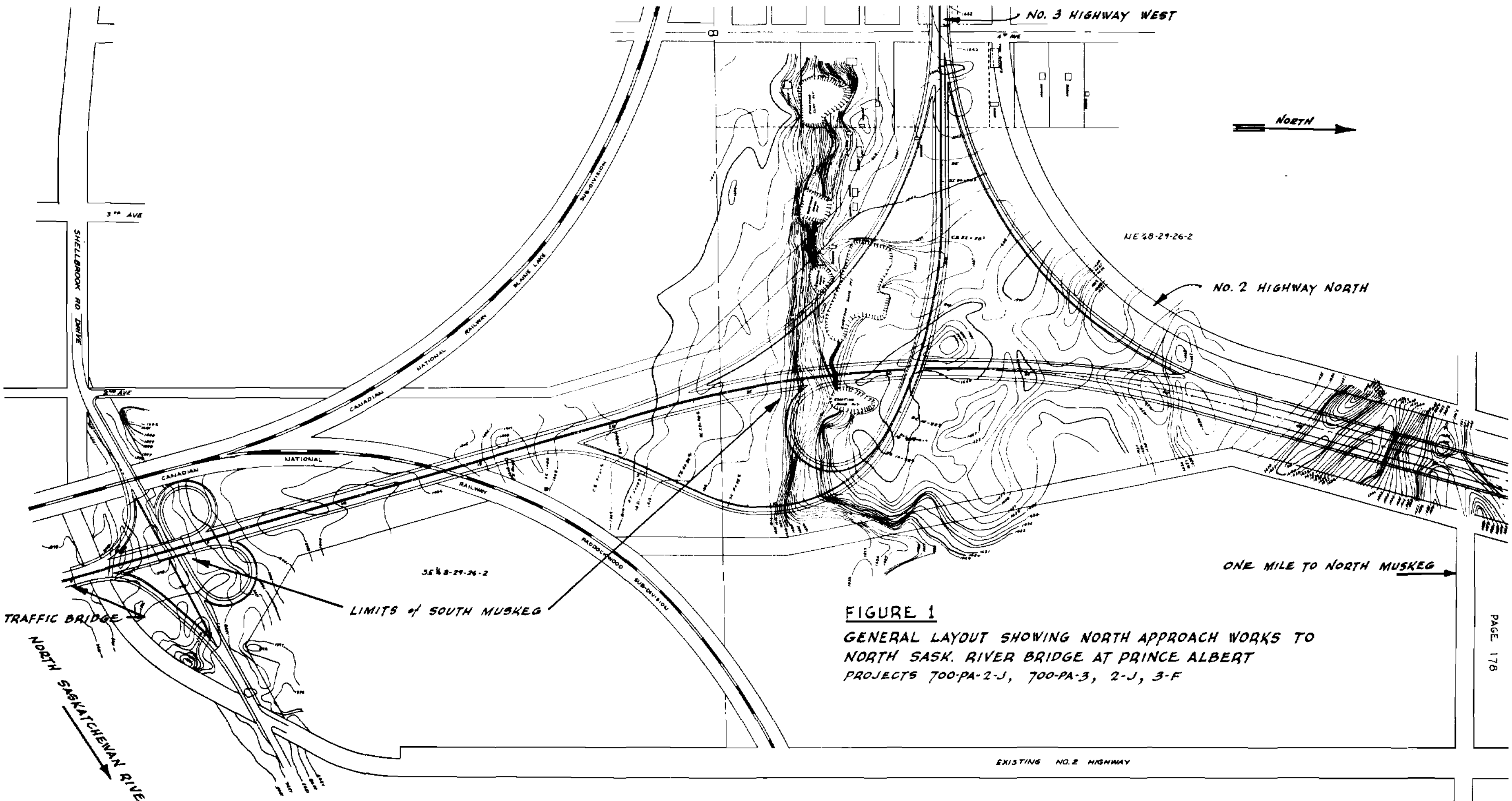


FIGURE 1

GENERAL LAYOUT SHOWING NORTH APPROACH WORKS TO NORTH SASK. RIVER BRIDGE AT PRINCE ALBERT

PROJECTS 700-PA-2-J, 700-PA-3, 2-J, 3-F

PART of A DRAWING BY PROVINCE of SASKATCHEWAN DEPT. of HIGHWAYS AND TRANSPORTATION

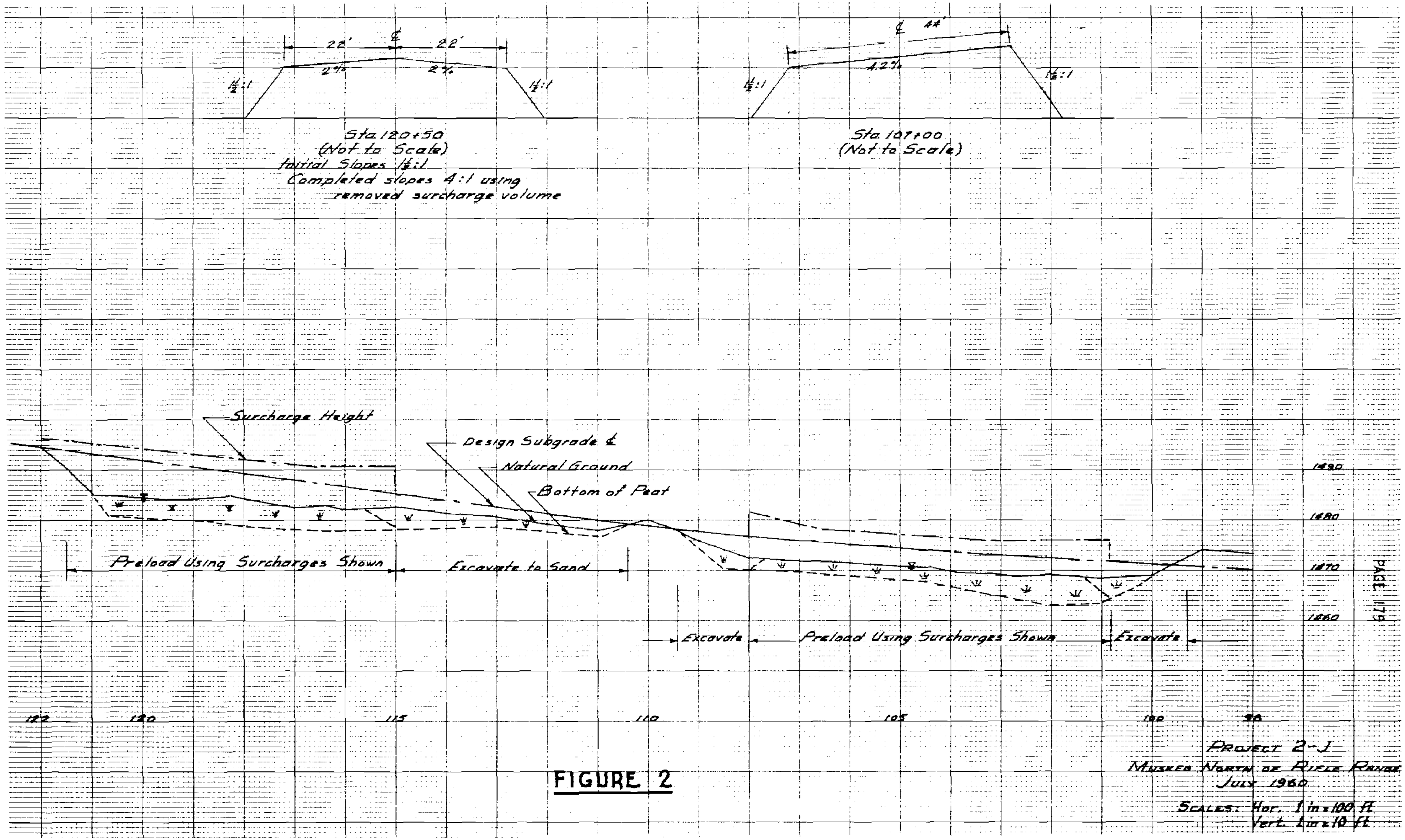

22'
℄
22'
2%
2%
1½:1
1½:1
Sta. 120+50
(Not to Scale)
Initial Slopes 1½:1
Completed slopes 4:1 using
removed surcharge volume
℄
44'
4.2%
1½:1
1½:1
Sta. 107+00
(Not to Scale)
Surcharge Height
Design Subgrade ℄
Natural Ground
Bottom of Peat
Preload Using Surcharges Shown
Excavate to Sand
Excavate
Preload Using Surcharges Shown
Excavate
1890
1880
1870
1860
122
120
115
110
105
100
98
Project 2-J
Muskeg North of Rifle Range
July 1960
Scales: Hor. 1 in = 100 ft.
Vert. 1 in = 10 ft.

FIGURE 2

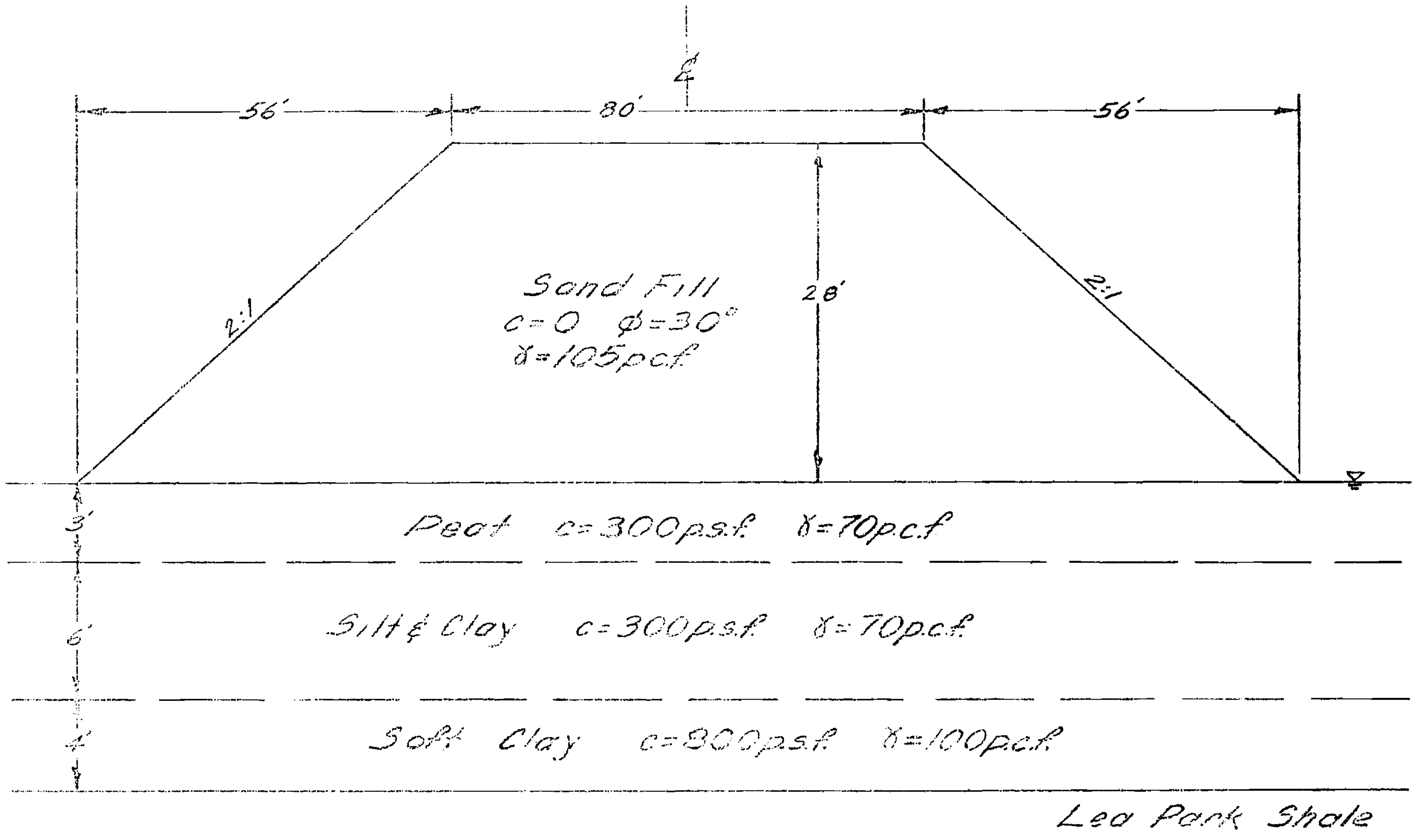


FIGURE 3

CROSS-SECTION FOR STABILITY ANALYSIS

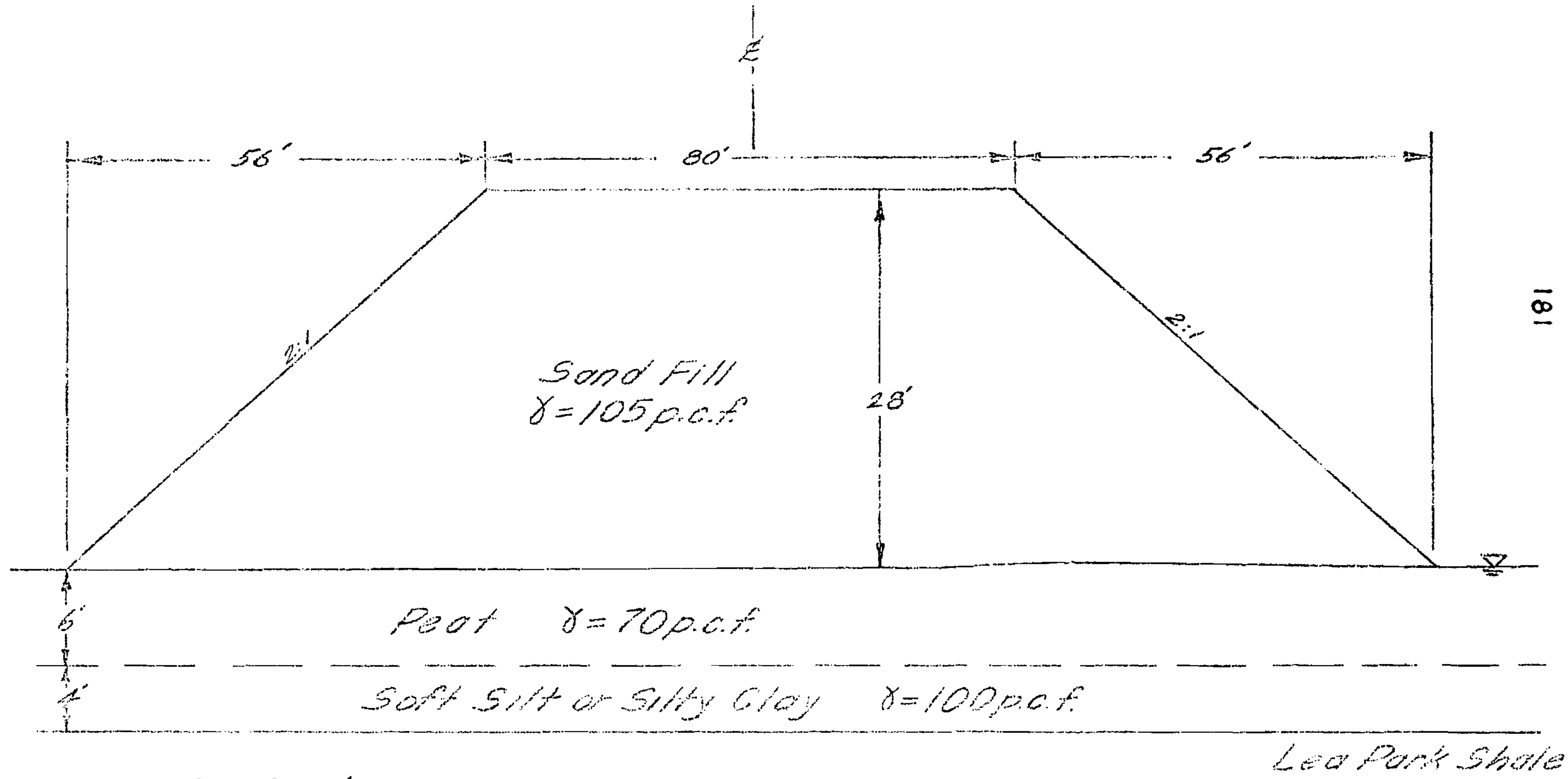


FIGURE 4
CROSS-SECTION FOR SETTLEMENT ANALYSIS

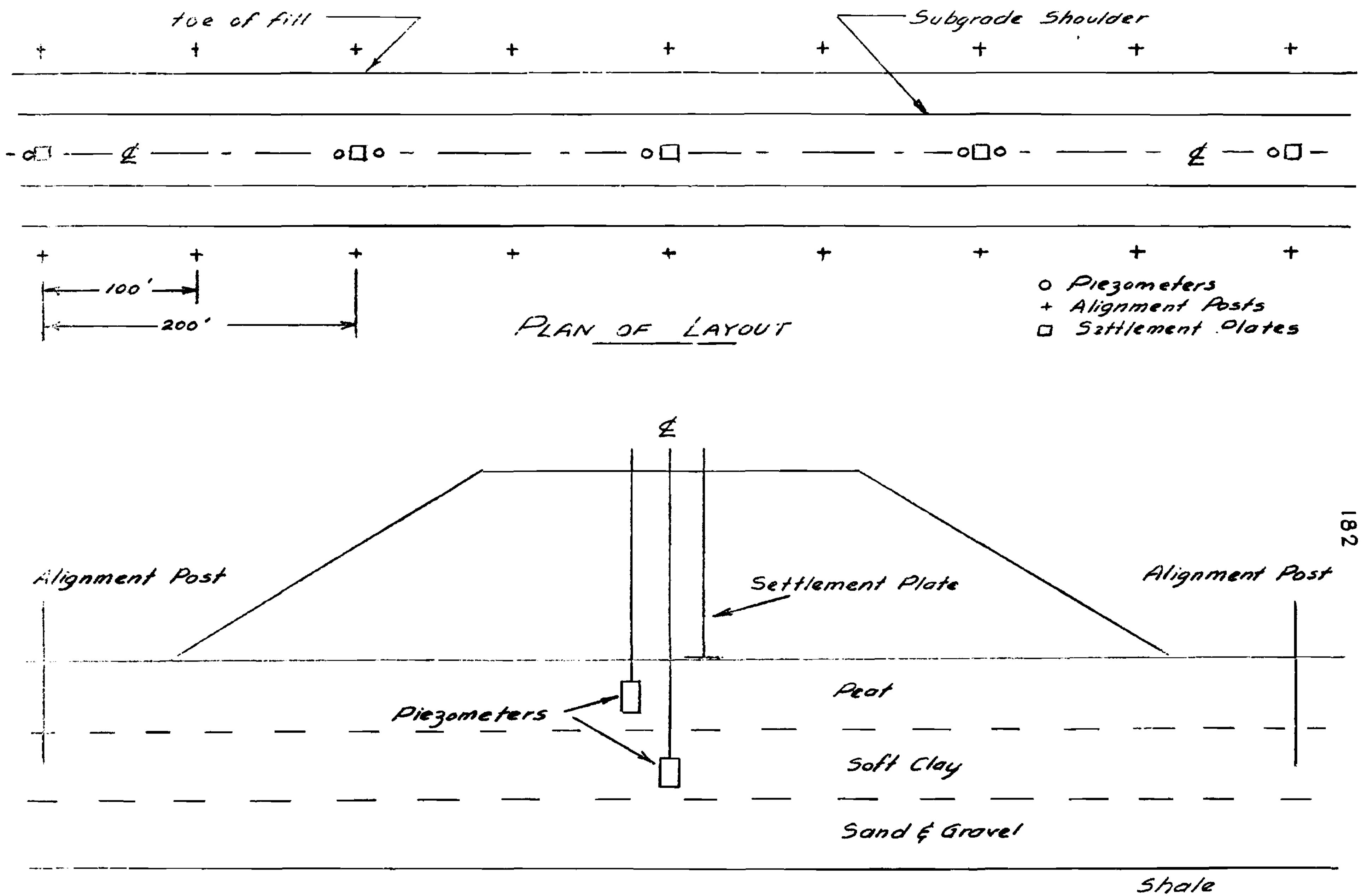


FIGURE 5

TYPICAL CROSS-SECTION

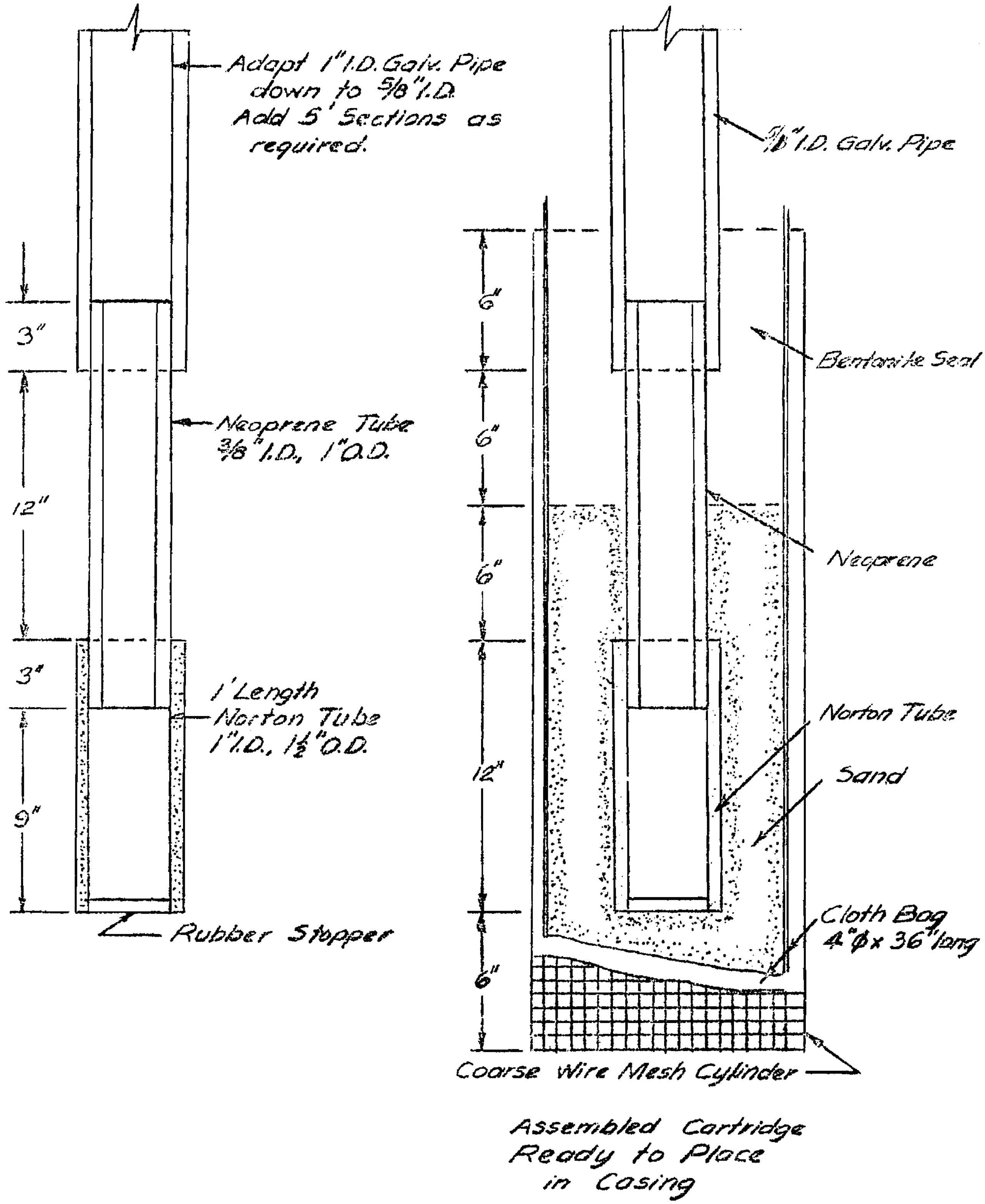
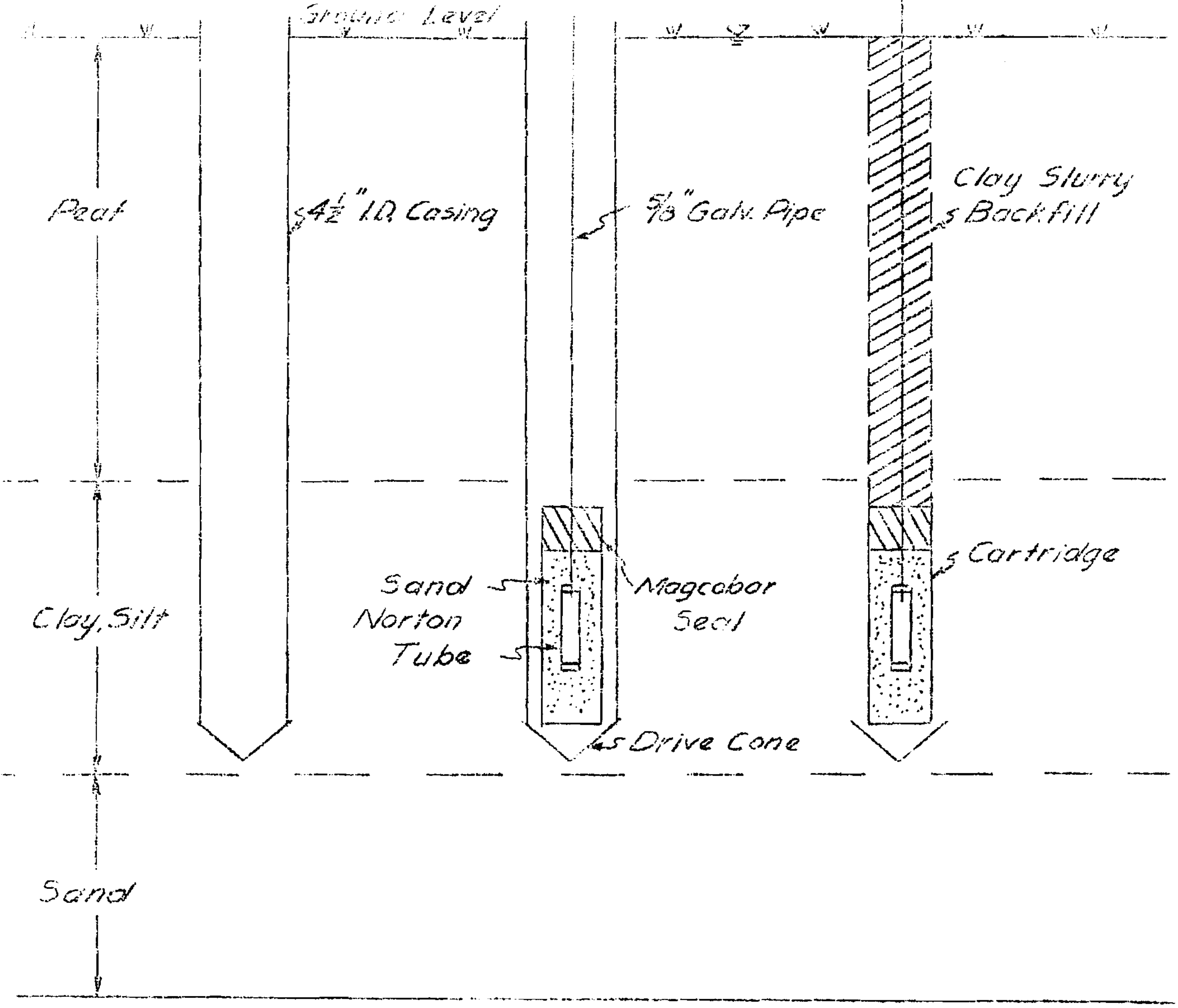


FIGURE 6

PIEZOMETER DETAILS



Casing driven with drive cone.
Casing filled with water.

Cartridge assembly saturated.
Magcobar seal placed in plastic state.
Cartridge lowered to bottom of casing.
Standpipe filled with water to ▽

Casing jacked out.
Drive cone and cartridge remain.
Backfill placed as casing withdrawn.

FIGURE 7

PIEZOMETER INSTALLATION

Discussion

Dr. Assur (Cold Regions & Research Laboratory) asked what the settlement plate (see Fig. 5) measured. Mr. Mickleborough said that the foundation settlement is being measured since the instrument sits on top of the muskeg. Mr. Keene (Connecticut State Highway Department) wondered how such a high fill and heavy construction equipment could be put on such soft peat. Mr. Mickleborough replied that it was done during the winter when the peat was frozen.

Mr. Brownridge (Ontario Department of Highways) wanted to know what criterion was used for determining whether to excavate or to surcharge the peat. Mr. Mickleborough explained that the criterion was rule of thumb together with experience; also the criterion suggested by Brawner was of assistance. In any case, there was a desire to try the method, which has resulted in a considerable saving of money. Mr. Brownridge wondered if an estimate had been made of the economics of excavation versus preloading. Mr. Mickleborough said that there had been an estimate made but could not remember the amount of money saved.

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III. 5. a. EMBANKMENT SETTLEMENT BEHAVIOUR ON DEEP PEAT

C. F. Ripley and C. E. Leonoff

Large areas in the vicinity of Greater Vancouver are mantled with peat ranging in thickness from 5 to 30 ft. During the past 15 years, with the rapid growth within the Metropolitan area, outlying regions surrounding the peat bogs have been developed for residential and industrial use. Because of their location, the peat bogs have now become increasingly desirable areas for development. Attempts are being made to develop the peat bogs for different uses by a variety of methods. The methods vary, depending upon the intended use of the property and the physical characteristics of the peat at the particular site. The two most common methods are: to completely remove and replace the peat with inorganic soil and to develop a fill above the peat using various procedures to cope with the inevitable settlements. Neither method is universally applicable nor superior under all conditions.

In the authors' experience, the period of observation of some of the floated fills which are being placed on peat has not yet been sufficiently long to establish their success. In spite of elaborate procedures of preloading, the ultimate settlements may yet be excessive within the normal design life of the structures. In several cases dealt with by the authors, the preloading method was neither practicable nor economic, particularly where the peat was deep, 20 to 30 ft., and the shear strength was low.

This paper presents the settlement history of a floated-fill embankment constructed on a 25 ft. deep peat bog. The fill was built to act as a storage dyke with a specific crest elevation, 12 ft. above the initial surface of the peat. The fill has not been a complete success because of excessive settlements. The cross-section of the embankment was designed on the basis of in situ vane shear tests to avoid shear failure of the foundation. Nevertheless, substantial shear movements have occurred even though the shear stresses have been significantly lower than the undisturbed shear strength obtained by the vane tests. At sections of the fill where the observed settlements have not been affected by shear deformations, the settlement records indicate that long-term settlement occurring in the period normally attributed to secondary compression will, within 25 years, equal the amount that has occurred in the period normally attributed to primary consolidation.

Description of Fill

The embankment site is in the Lower Fraser Valley, a few miles east of the City of Vancouver, at a point where a deep peat deposit exists beneath a marginal strip of land between the river and the rising ground at the side of the valley. The natural ground surface slopes across the site at a gentle gradient of 3% towards the river that is located 700 ft. to the north.

Fig. 1 shows a plan view and design cross-section of the embankment which is U-shaped, each arm being about 225 ft. long and the main stem 650 ft. long. At the main stem the cross-section consists of a 150 ft. wide raft of fill 6 ft. thick with a central dyke atop the raft having a crest width of 12 ft., rising to an elevation of 12 ft. above initial ground surface of the peat.

The soil profile beneath the main stem of the embankment is shown in Fig. 2. The peat occurs below the ground surface with a uniform thickness of 25 to 30 ft., resting on a layer of clean sand and gravel about 20 ft. thick. Below the gravel is a thick body of hard, heavily pre-consolidated clay. Throughout the year, the ground water level in the peat ranges between the ground surface and a foot below. By the Radforth system (5), surface vegetation would be classified as coverage formula "ADG". The peat category is "6", changing to "4" with depth; that is, predominantly amorphous-granular containing woody fine fibres, held in a woody, coarse-fibrous framework, changing to amorphous-granular peat containing woody fine fibres. By topographic features, the site conforms to contour type "n" - sloped river margin.

The natural water content shows abrupt variations both vertically and horizontally within the deposit (Fig. 3). The average value of samples from the test holes is about 1000%, with an extreme range from 100% to 2100%.

Fig. 2 shows typical results of vane shear tests made at several test holes. The shear strength of the peat prior to placing the fill ranged from 2 to 3 p.s.i. for the undisturbed condition, and from 0.75 to 1 p.s.i. for the remoulded condition. The vane, 5 ins. long and 2 1/2 ins. wide, was rotated at a rate of 0.4 degrees per second. Vane tests were made subsequently, adjacent to the initial test holes, after different stages of fill construction.

Design of Embankment

The purpose of the embankment was to form an impounding basin around a large oil-storage tank. In order to serve this function, as settlement occurred, the crest of the dyke had to be kept at a specific elevation, some 12 ft. above the initial ground surface. Being in an industrial area, the adjacent roads, railways, power lines, and underground service lines, existing along the boundaries of the site, could not be disrupted by the embankment construction.

Theoretical analyses of shear strength and settlement were made to determine whether it would be practicable to support a 12 ft. high embankment on the peat at this site. The analyses were based on the results of in situ vane shear tests and laboratory consolidation tests, using the conventional Boussinesq equations to compute the stresses induced in the peat by the fill loads. The studies indicated that an embankment of the proposed height could be floated on top of the peat using the special procedures discussed later. Other alternatives such as excavation and replacement of the peat, and displacement, were considered and rejected for different reasons.

The low shear strength of the peat was the major design consideration. The design studies indicated that a 150 ft. wide raft of fill was needed in the lower half of the embankment to reduce the stresses in the peat. Furthermore, the fill would have to be built in stages to mobilize an increase in strength of the peat so that it could support the full embankment load. The rate of stage loading was to be controlled in the field on the basis of in situ vane shear tests, piezometer and settlement observations, which would be made as the work proceeded.

The design studies indicated that large settlements of the peat would take place under the fill load. However, the settlements were not considered to be a major problem provided shear failure did not occur. Compensation for the settlements could be made by placing additional fill as the embankment construction proceeded. Computations were made of the probable magnitude and time-rate of settlement. These were considered to be little more than crude estimates because of the variations in compressibility of the peat as indicated by the differences in natural water content, the inevitable variations between the real and assumed distribution of fill-load stresses in the peat, and the difficulty in sampling and making reliable consolidation tests on peat. A programme of settlement observations was undertaken to determine the actual settlement behaviour, also to provide a more reliable basis for predicting the magnitude of long-term settlements for which allowance would have to be made when the final layer was placed. Experiences at other sites suggested that the primary consolidation of the peat would occur rather quickly, in a month or two, but that secondary settlement would continue for many years.

Construction of Embankment

The sequence of stage construction is shown in Fig. 4-A for a typical cross-section of the main stem of the embankment. The first stage, constructed during the last week of March 1957, consisted of a 4 ft. thick base layer of clean, medium sand placed directly on the peat. The second stage was placed after a period of two months, in late May 1957; it consisted of a layer of glacial till, which increased in thickness from 0.5 at the edges to 3 1/2 ft. at the center. The third stage was placed in the third week of September 1957, as a levelling layer, having a thickness of 1 ft. at the edges, increasing to about 3 ft. at the center. No additional fill was added until 1 May 1958. During the period 1 May 1958 to 1 November 1960, 16 ft. of fill were placed atop the raft in the central dyke section, in successive layers of about 1 ft. thickness at intervals of 1 to 3 months.

The total thickness of fill placed in slow increments over a 4-year period ranges from 26.5 ft. at the central dyke section to 6 ft. at the edges of the raft. The design crest elevation has not yet been achieved, the net increase in height to date being only 8.5 ft.

Settlement Behaviour

The cross-hatched segment of the fill diagram at each stage in Fig. 4-A, shows the position to which the fill in place had settled prior to placing fill for that stage. The bottom diagram of Fig. 4-A shows the position to which the fill had settled on 1 April 1961.

Fig. 4-B shows the observed time-settlement relation beneath the axis of the central dyke at settlement gauge 5. Time is plotted on the abscissa to logarithmic scale; settlement is plotted on the ordinate to arithmetic scale. A chart of the fill thickness placed at the gauge is shown at the top of the figure; a chart of effective vertical stress at the base of the fill is shown at the bottom of the figure. The latter chart reveals the effect of settlement of the base of the fill below the water table, which reduces the effective stresses applied to the peat by the fill. In the same manner Fig. 4-C shows the observed

time-settlement relations at a typical point midway between the central dyke fill and the edge of the raft, at settlement gauge 4. A summary of the settlement behaviour at the two gauges is presented in Table I.

The terms "primary consolidation", "secondary compression", and "shear deformation" are used in this paper in accordance with the definitions suggested by Rutledge and Johnson (6). "Primary consolidation" is decrease in volume of a soil through decrease in volume of its pore spaces, accompanied by a compression or squeezing out of pore fluid, whether gas or liquid, or both. "Secondary compression" is a volume change phenomenon which continues after completion of primary consolidation; for organic soils it is characterized by a straight-line relation between volume change and logarithm of time. "Shear deformation" is the phenomenon of change in shape under the action of shear stresses. This type of deformation can occur without volume change.

The data in Table I and Figs. 4-B and 4-C reveal a marked difference in settlement behaviour beneath the lateral shoulder fills and the central dyke fill. During the first 13 months (March 1957 to May 1958), 7.5 ft. of fill at gauge 4 produced 4 ft. of settlement for a net gain in elevation of 3.5 ft.; whereas 10.5 ft. of fill at gauge 5 produced 10.4 ft. of settlement for a net gain in elevation of 0.1 ft. The most reasonable explanation for the proportionally greater settlement at gauge 5 during this period is that not only consolidation but shear deformation occurred at that gauge; whereas at gauge 4 only consolidation occurred. The "straw which broke the back" appears to be the extra two feet of fill placed at gauge 5 in late May 1957: 3.5 ft. at gauge 5 as compared with 1.5 ft. at gauge 4. A comparison of the settlement readings suggests that, of the 10.4 ft. of settlement at gauge 5 during the above period, 4.5 ft. occurred as shear deformation. Similarly, part of the settlement of 7.5 ft. at gauge 5 during the period May 1958 to April 1961 appears to be the result of shear deformation. These rather large shear deformations in the peat are particularly significant since they occurred at stresses which were considerably lower than the in situ undisturbed shear strength measured by vane test. Furthermore, they were not accompanied by a definite break of rupture on the surface of the fill in the vicinity of the gauge. Lateral displacement of the ground surface was apparent beyond the lower (north) edge of the raft, but accurate measurements of lateral movement were not taken. The lateral movement was estimated to be about 2 ft., by visual examination at a parallel ditch where displacement was obvious.

Piezometers were installed beneath the fill to measure the rate of dissipation of pore pressure and the corresponding rate of increase in strength of the peat, as a guide in controlling the rate of fill construction. Difficulties, unfortunately, were encountered with upward drainage along the outside of the piezometer pipes, which invalidated the piezometer readings. Therefore decisions concerning the rate of filling were based primarily on the settlement readings.

Without pore pressure measurements, delineation of primary consolidation and secondary compression from the observed settlement curves is rather difficult. Nevertheless the settlements at gauge 4 have continued during the past two years on a straight-line relationship at a remarkably steep slope, on the settlement vs. log-time curve. By visual estimate, primary consolidation appears to have ended in the period 10 to 15 months (Fig. 4-C). The curve indicates that settlements due to secondary compression will amount to about one-half of the total settlement in a 25-year period.

Discussion of Settlement Behaviour

Behaviour similar to that observed at this site has been noted more recently in connection with other projects on deep peat bogs in the Greater Vancouver area. Investigations have been made at two different embankments where definite shear failures had occurred. They indicated that the average computed shear stresses at failure caused by the fill loads were much lower than the undisturbed shear strength measured by vane test. In each case, as at this site, the computed average shear stresses corresponded closely to the remoulded shear strength values measured by vane tests, being from one-third to one-half the undisturbed shear strength. Observations at another site with a lesser thickness of fill above an equally deep peat deposit have shown a high rate of settlement on a straight-line relationship, after one would have expected primary consolidation to be completed.

The authors have experienced, in several instances, significant differences between the observed field behaviour and the predicted behaviour computed using conventional theoretical analyses based on field and laboratory test results. Therefore a research programme has recently been initiated to investigate in more detail the laboratory properties of local peats, particularly with respect to primary consolidation and secondary compression. Consolidation tests have been made in the conventional consolidation test apparatus and in a triaxial compression cell. The tests have been made at low loads within the range of fill loads that can be safely supported on the peat during the first fill-load increment.

Fig. 5 summarizes the test results to date. At this preliminary stage the results are inconclusive. The mechanism by which the long-term settlements occur is not clear; that is whether they occur partly as primary consolidation and partly as secondary compression, or whether they occur solely as secondary compression. However, the test results suggest strongly, for the particular peat tested, that long-term settlements occurring after the normal primary consolidation period, may be as large as the short-term settlements occurring within the normal period of primary consolidation.

Fig. 5-A presents the settlement vs. log-time curves for long-term consolidation tests on two specimens cut from undisturbed block samples of Lulu Island peat. Test "A" was loaded in two increments; the first, 0 to 134 lb/sq/ft. was applied for about 2500 minutes, and the second, 134 to 304 lb/sq/ft. was applied for 100,000 minutes and is still continuing. Test "B" was loaded in a single increment, 0 to 340 lb/sq/ft., for a period of 25,000 minutes. During the first 100 minutes the curves in Fig. 5-A have the typical "S" shape for primary consolidation of an inorganic soil. After 100 minutes the gradient of the curves increases progressively resulting in a concave downward shape. While curves of this shape are somewhat unusual for inorganic soils, they appear to be common for peat soils at low loads. Similar curves have been reported by Thompson and Palmer (7), Lewis (4), Lake (3), Barber (2), and Adams (1). A clear explanation has not been set forth in the literature to date, for the shape of the curves or for the mechanics of the long-term consolidation of laboratory test specimens of peat. The laboratory research programme is being continued to study this further.

Fig. 5-B presents the results of a consolidation test carried out in a triaxial cell on a sample of peat from the same deposit

as the consolidation tests in Fig. 5-A. The specimen was subjected to a cell pressure of 400 lb/sq/ft. with drainage permitted at one end. Longitudinal filter strips were placed at the sides of the specimen for its full height inside the confining membrane. Periodically during the test the drainage was momentarily stopped, the pore pressure was allowed to equalize and a measurement of the pore pressure was taken. Burette measurements of the volume of water squeezed out of the specimen were taken periodically. Curves are given in Fig. 5-B of axial strain, volume change as measured in the burette, and the measured pore pressure, all plotted against logarithm of time. Fig. 5-C presents the results of a series of pore pressure vs. cell pressure measurements that were made on the specimen prior to the consolidation test in order to assess the degree to which the specimen was saturated.

The results obtained from the single test in the triaxial cell are interesting in a qualitative way but they do not justify an attempt at precise analysis of the settlement vs. time relationship because of the elementary nature of the test and the many unknowns concerning the drainage conditions within the specimen. Nevertheless, the almost full dissipation of pore pressure after a period of 1000 minutes suggests that primary consolidation was virtually complete by that time. The curves of axial strain and volume change, as measured in the burette, show a remarkably steep slope beyond the time of 1000 minutes. Thus this test also suggests that settlements of large magnitude may continue after the pore pressures have been dissipated and the primary consolidation has been completed. Additional triaxial tests of a more precise nature are being made to investigate more closely the pore pressure and volume change relationships with time.

Summary

1. Experience at this site indicates that stability computations of fills above deep peat deposits, based on the undisturbed shear strength measured by in situ vane tests, may not be reliable. Large shear deformations occurred in the deep peat deposit at this site even though the average shear stresses induced by the fill were about one-half the undisturbed shear strength of the peat measured by vane test. The average shear stresses at the

time of initial shear deformations corresponded closely with the remoulded shear strength determined by vane test. Analyses of shear failures at two other embankments above deep peat deposits in the Vancouver area have revealed similar relationships with respect to the shear strengths measured by in situ vane tests.

2. Experience at this site indicates that the preloading method of inducing settlements of a completed fill may not be practical for floated fills placed above deep peat bogs because of the extremely slow rate of fill placement necessary if rupture of the peat is to be avoided. On the other hand, if large shear deformations or rupture can be permitted, the fill may be placed more quickly and the preload method may be applicable. In such a case, however, the operation is a displacement fill rather than a truly floated fill.

3. The settlement behaviour at this site indicates that analysis of field settlement measurements of fills above deep peat bogs, on the basis of consolidation and secondary compression considerations only, may not be valid. Large shear deformations occurred at this site without evidence of a definite rupture surface. Their occurrence was noted only as increased settlement.

4. The data indicate that long-term settlements occurring during the period normally attributed to secondary compression may amount to 50% of the total settlement. The settlement measurements reveal a high rate of continuing long-term settlements. Settlements of the raft sections of the embankment where no fill has been placed during the past 3 1/2 years, continue on a straight line relationship at a relatively steep slope.

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TABLE I

Summary of Observed Settlements

Time Interval	No. of Months	Settlement Gauge 4			Settlement Gauge 5		
		Fill Added (ft)	Settlement (ft)	Net Increase in height (ft) per interval	Fill Added	Settlement (ft)	Net Increase in height (ft)
Mar 31 - May 24/57	2	4	1.5	+2.5	4	2.1	+1.9
May 24 - Sept 13/57	3 1/2	1.5	1	+0.5	3.5	5.5	-2.0
September 13, 1957 to May 1958	7 1/2	2.0	1.5	+0.5	3.0	2.8	+0.2
May 1958 to Apr/61	35	0	1.6	-1.6	16	7.5	+8.5
Mar 31, 1957 to May 1958	13	7.5	4	+3.5	10.5	10.4	+0.1
Mar 31, 1957 to Apr 1961	48	7.5	5.6	+1.9	26.5	17.9	+8.6

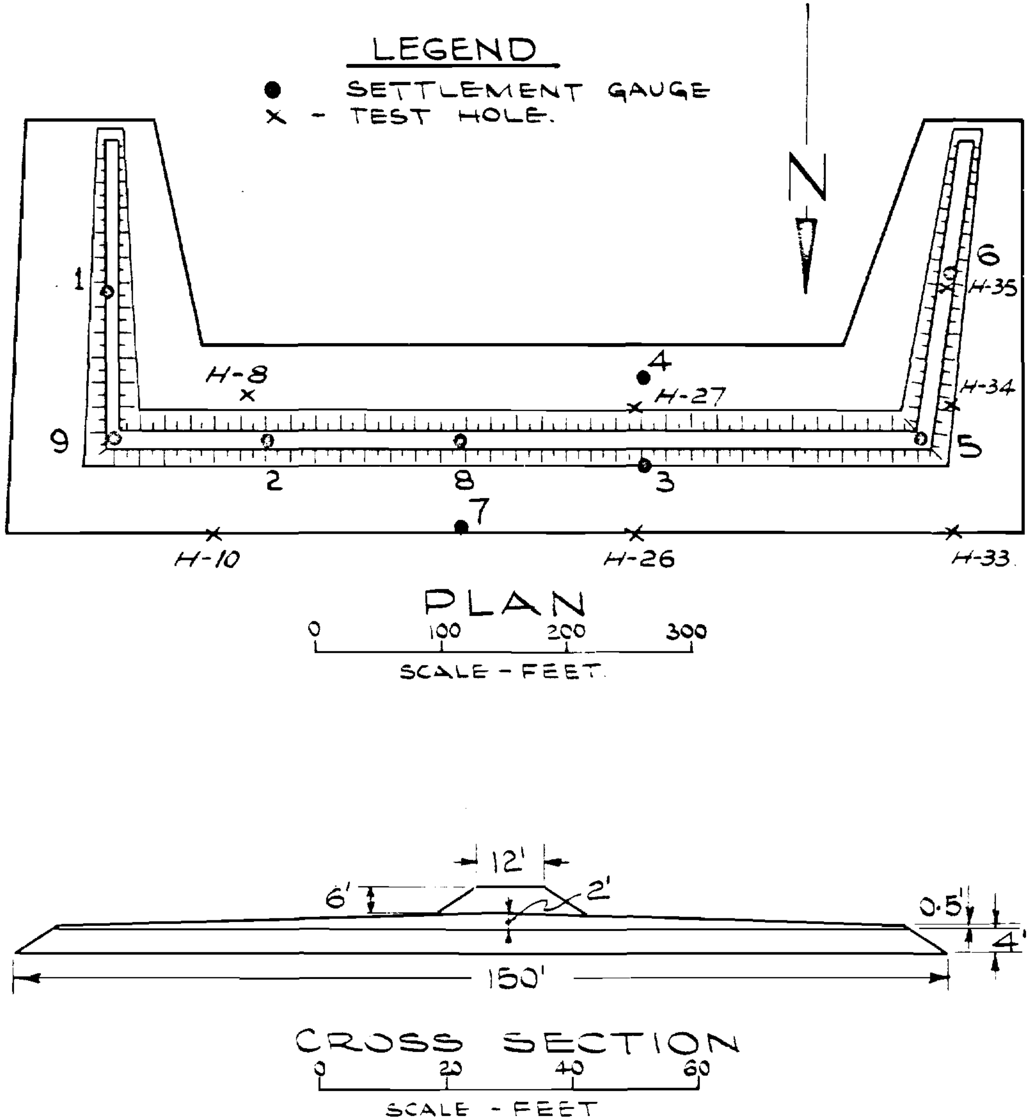


FIG. 1 - LAYOUT OF EMBANKMENT

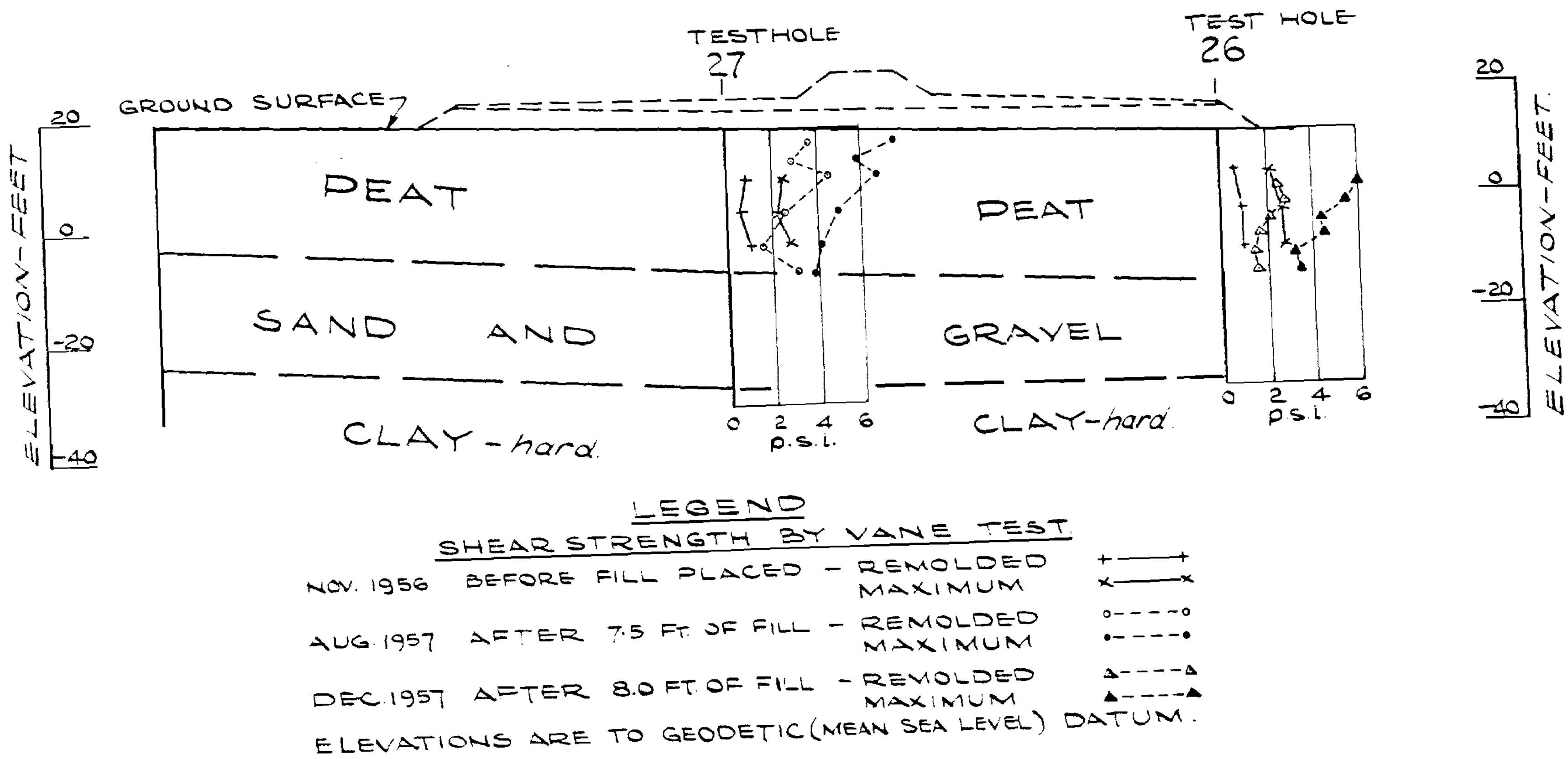


FIG. 2 - SOIL PROFILE.

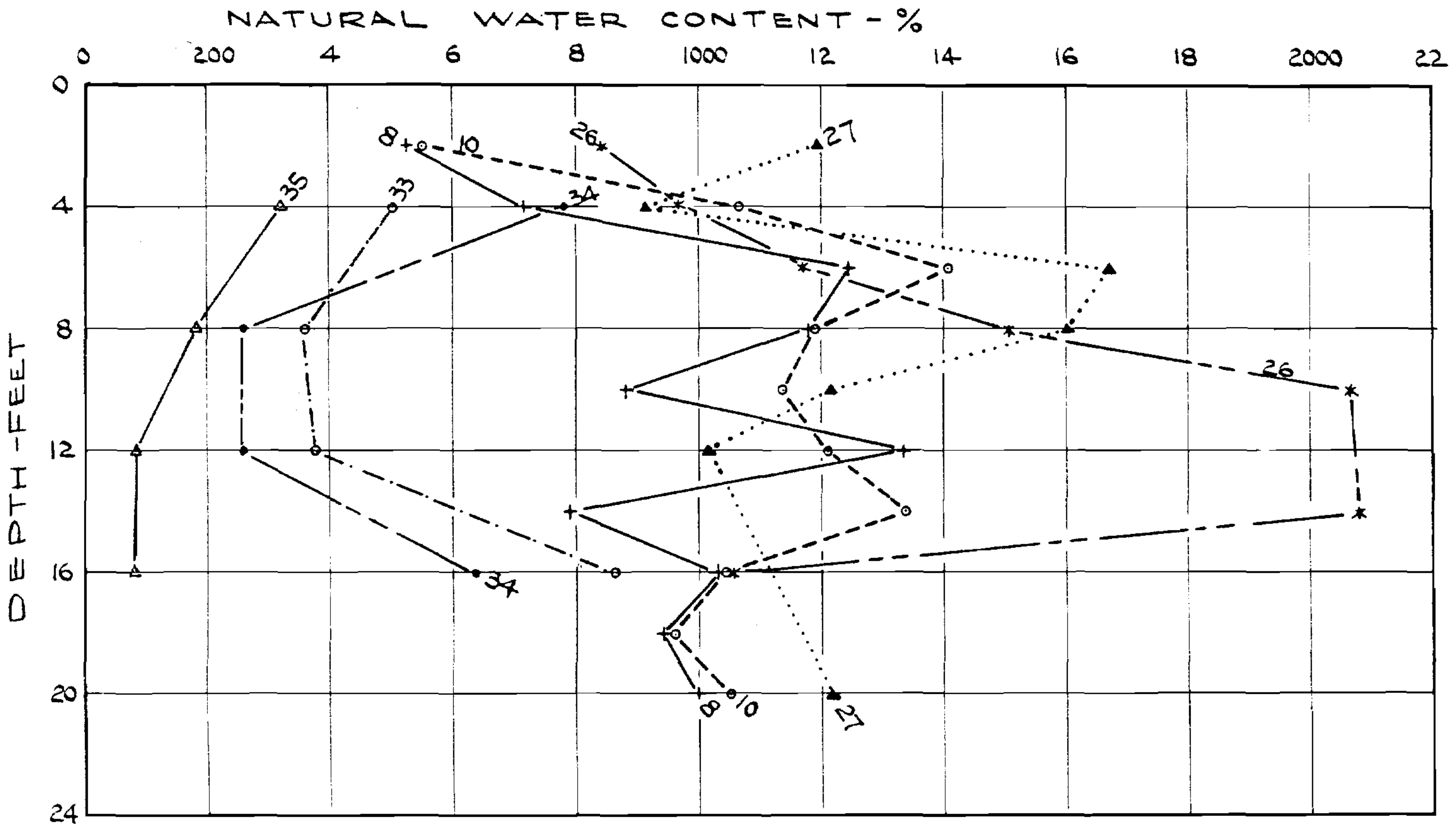


FIG. 3 - WATER CONTENT IN PEAT

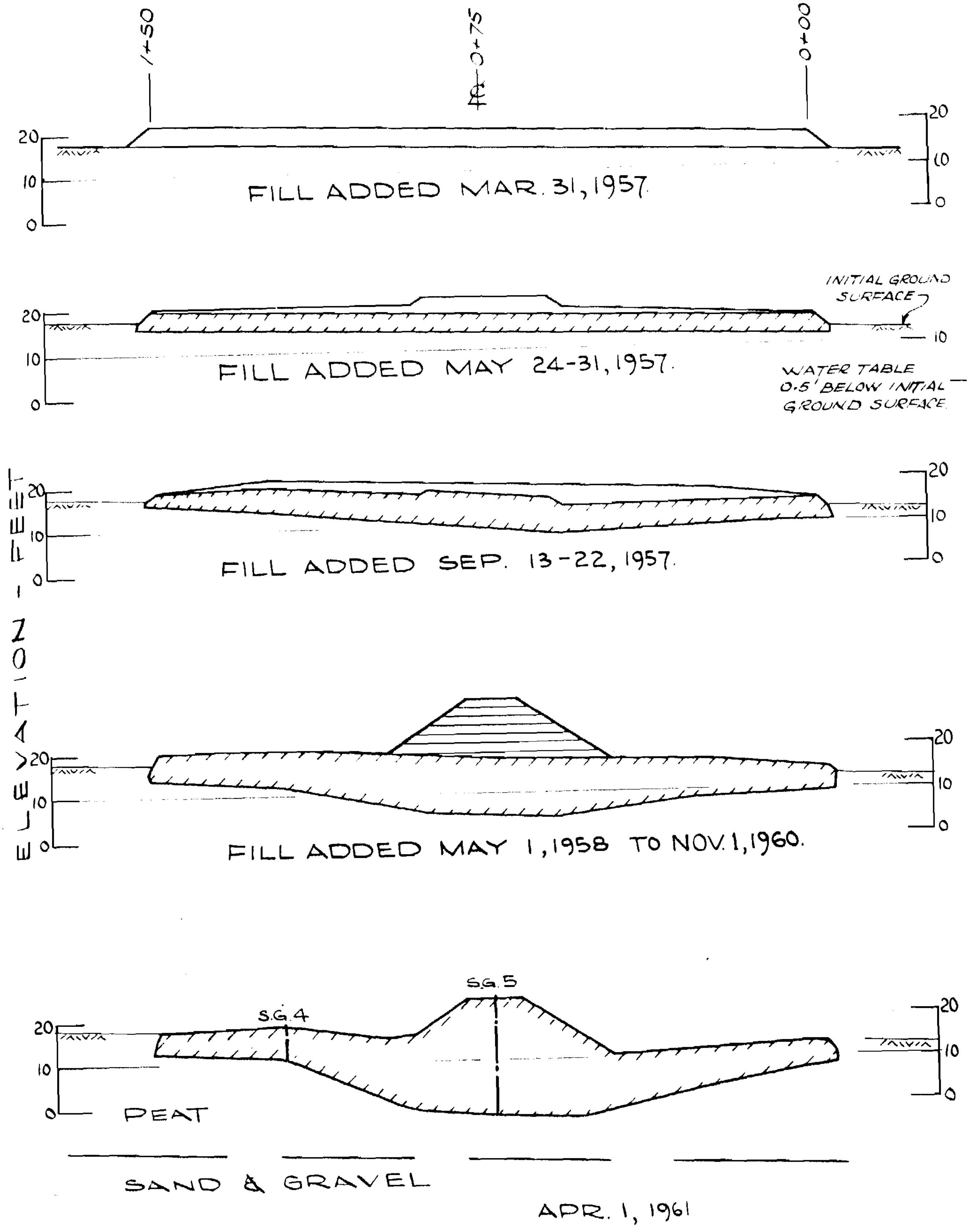
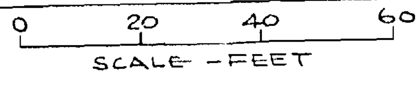


FIG. 4-A

FILL AND SETTLEMENT HISTORY

OF TYPICAL CROSS-SECTION



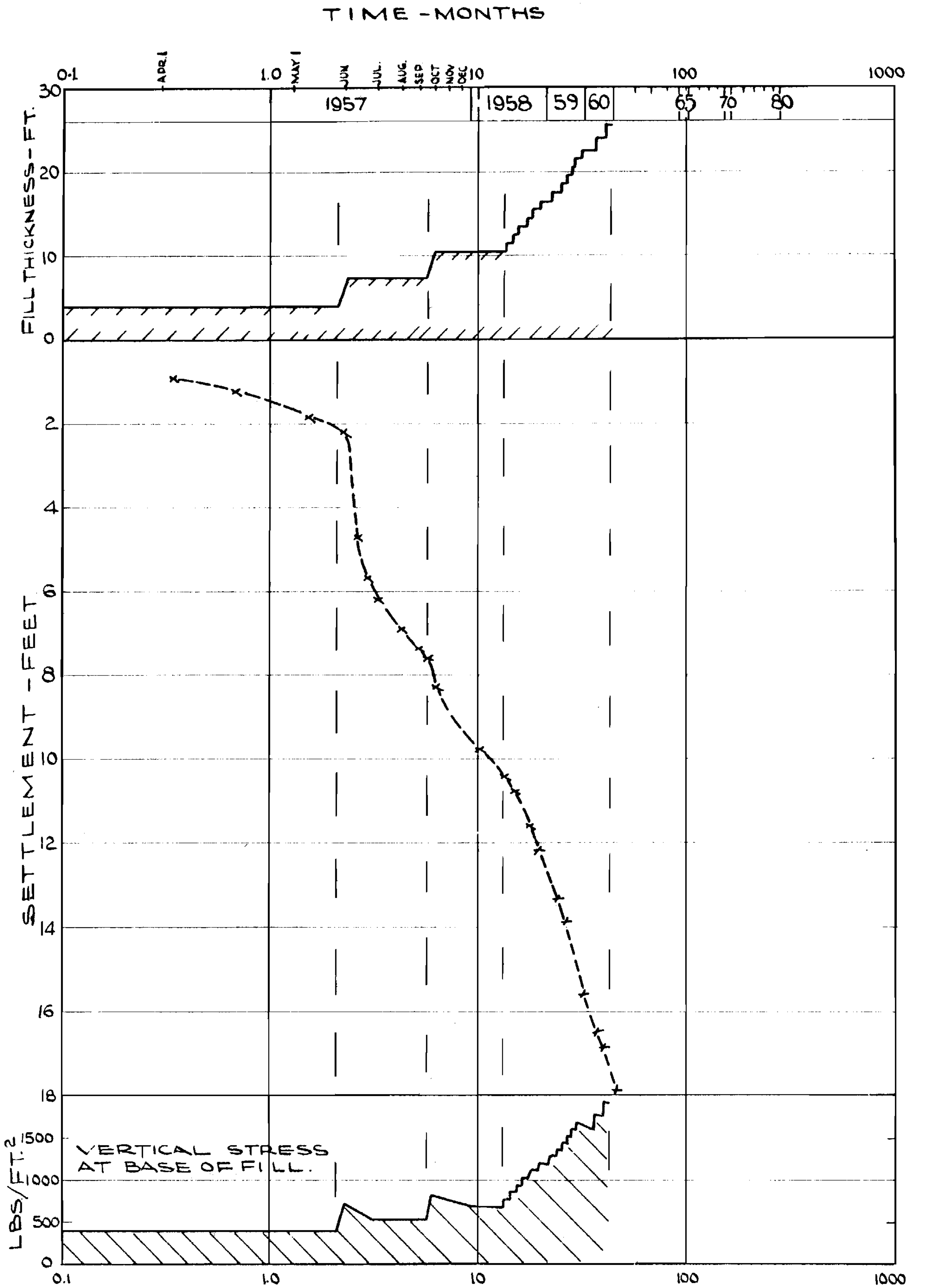


FIG. 4 - B

BEHAVIOUR AT SETTLEMENT GAUGE #5

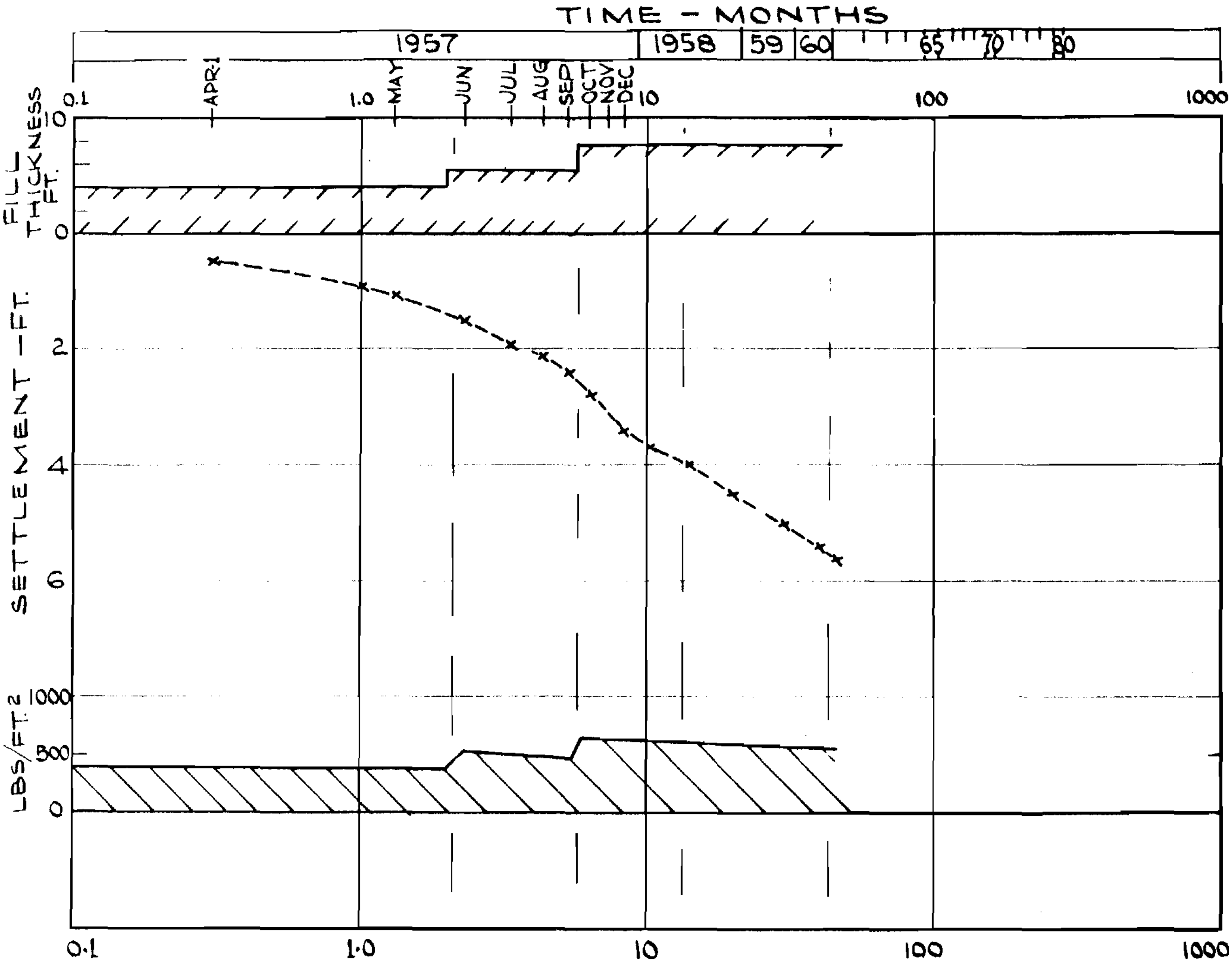
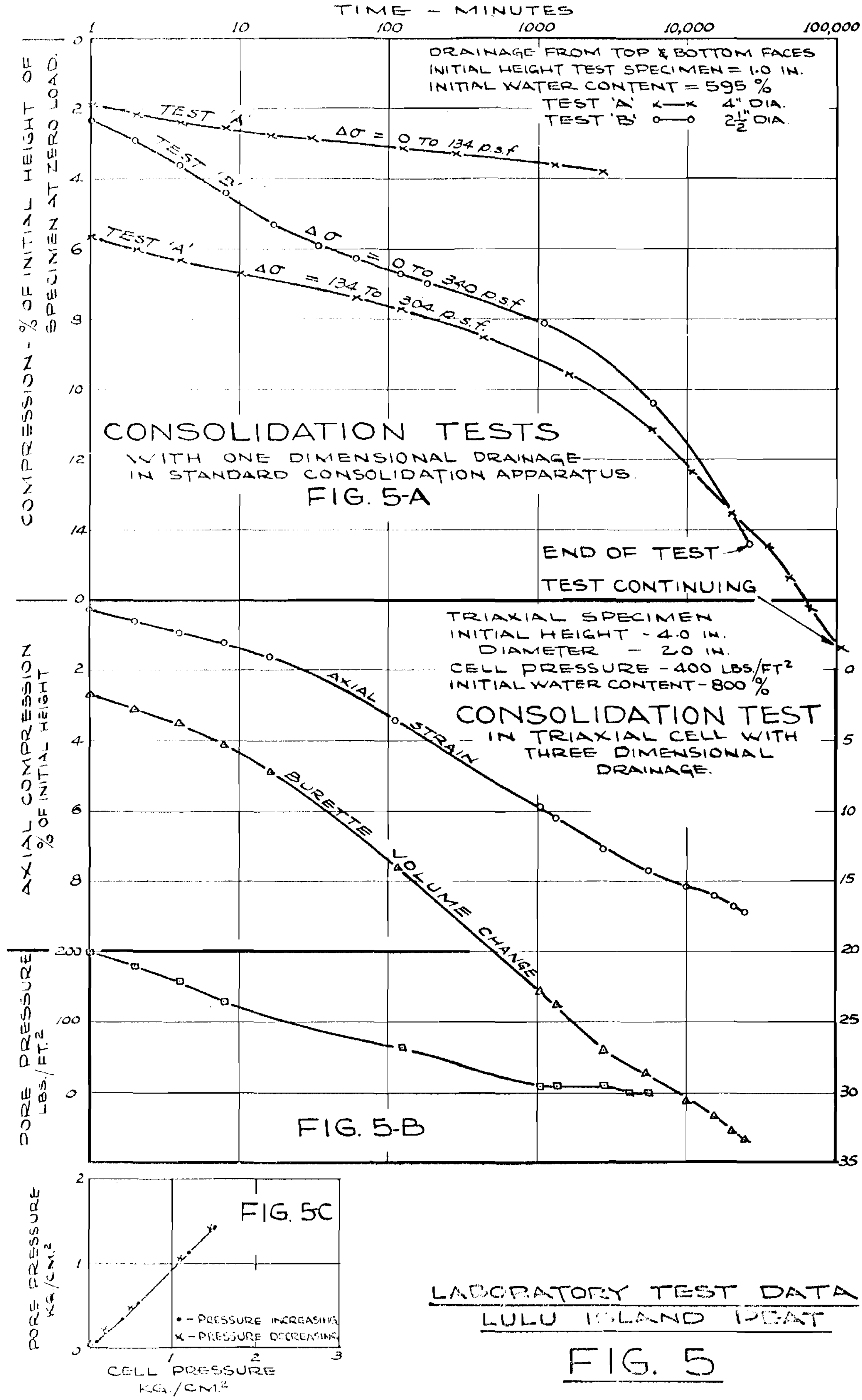


FIG. 4-C

BEHAVIOUR AT SETTLEMENT GAUGE #4



LABORATORY TEST DATA
LULU ISLAND PEAT

FIG. 5

Discussion

Dr. Chu (Tibbetts-Abbott-McCarthy-Stratton) in commenting on the charts relating settlement and log-time asked Mr. Ripley if he had any information correlating laboratory and field data. Mr. Ripley said that he is most anxious to obtain long-term field data to check against laboratory results; as yet these are not available. The Dutch have reported, however, that they are still grading roads over peat bogs one hundred years after they have been built. This may be shear deformation rather than secondary consolidation; there is no long-term data to confirm this. Mr. Keene (Connecticut State Highway Department) asked if a large part of the settlement under the main dyke was shear deformation, with no volume change, would there not have been some bulging? In reply, Mr. Ripley said that volume change would have resulted in upward bulging; there was very little of this. The road was displaced laterally, but was not lifted.

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III. 5. b. THE COMPRESSIBILITY OF PEAT WITH REFERENCE TO THE CONSTRUCTION OF MAJOR HIGHWAYS IN B. C.

S. F. Hillis and C. O. Brawner

The increasing demand for better alignment and the high cost of right-of-way are forcing the highway engineer to construct through extensive peat deposits. In the past, secondary roads were commonly "floated" on top of the muskeg, but for primary highways the peat was generally removed or displaced. These latter procedures are very expensive when the deposit is deep and, at present, methods are being developed for the construction of primary highways on peat which may, in many instances, provide the necessary stability more economically (2) (3) (9).

In order for construction on top of the peat to be successful, an evaluation of fill stability, pavement requirements and probable settlements must be made. This paper deals only with the settlement aspect. It summarizes the compression characteristics of peat, evaluates methods of predicting the field behaviour, discusses factors which influence such predictions, and outlines an approach which is being used to overcome the problem of large settlements.

Factors which warrant further study are also suggested.

It should be noted that the discussion is concerned mainly with the problem of major highway construction and the authors realize that some of the methods suggested may be inapplicable to the construction of lower quality roads or certain building projects.

Compression Characteristics

The one undisputed aspect of the behaviour of peat is that it is highly compressible. However, the mechanism by which this compression occurs is a matter of some controversy. Based on their own experience and a review of the literature, the authors would like to submit their present thoughts on this mechanism.

It appears that after some time, usually relatively short, the settlement of most peats continues as a straight line on a semi-logarithmic plot for as long a time as has been observed, which in some cases has amounted to several years. This applies both to laboratory and field observations (1), (4), (6), (7), (8), (11), etc.

Fig. 1 shows such a plot for a highway fill on a peat deposit near Vancouver. Until $t = 26$ days, and possibly for some time longer, there were still significant pore pressures in the peat. Therefore, the process up to this time could be compared with the usual primary consolidation associated with inorganic clays in that the rate of settlement is principally a function of the decrease in pressure in the fluid occupying the pores (12). There are, of course, some important differences between the behaviour of peat at this stage and that of inorganic soils due to many factors and it is unlikely that the classical theory of consolidation can be rigorously applied even during this stage of compression.

After the pore pressures have dissipated, and probably for a fair time before, the second phase of settlement, known as "secondary compression", occurs. This is the phase which shows on Fig. 1 as a straight line after t = 46 days. The bulk of the evidence indicates that the rate of secondary compression is independent of the drainage conditions and thus theoretically independent of the thickness of peat layer.

Secondary compression may be due to the plastic deformation of the solids and adsorbed layers as postulated by Buisman (4) and the amount appears to increase with increased shear stresses.

The above description is more or less the classical conception of the process of settlement within a peat layer and the authors have as yet found no convincing evidence which would lead them to think that it is not a reasonably accurate picture of the process.

In the past, some misunderstanding as to the nature of the process has been caused by the fact that significant pore pressures have been measured in the peat for a long time after the settlement became directly proportional to the logarithm of time. However, a large number of the piezometers used were of the well point type and in the light of recent experiments on the sensitivity of piezometers and data obtained from instrumentation on a large highway project, the authors are doubtful as to the validity of these measurements at least in B. C. peats. If absolutely reliable, pore pressure data have to be obtained in peat soils, then porous stones with high air entry values, or closed systems, are recommended (14).

Prediction of Field Behaviour

It is appropriate at this point to discuss the factors which will affect the prediction of field behaviour from laboratory tests or other means.

(a) Variability

Peat is extremely variable in most of its characteristics, not only from one location to another but also within the same deposit. Such variations are associated with the type of plant from which it is derived, the amount of humification which has occurred, the mineral content, etc. The variation with depth is often very noticeable, since peat deposits are usually formed in layers which, of course, may be quite different in their nature.

These features affect such properties of the peat as specific gravity (6), permeability (5), degree of saturation, moisture content and ultimately its strength and compressibility. This is a great hindrance to successful highway construction and to laboratory investigation. Since so many aspects of the material are affected it is also very difficult to compare the work of various investigators even with the help of classification systems.

(b) Sampling

It is difficult to obtain sufficiently undisturbed samples of peat - especially if it is at all fibrous in nature. Even if apparently undisturbed samples can be taken, by the time the soil reaches the consolidometer or triaxial machine it is likely to be greatly different from the material in situ. This is due to many factors such as (i) inability of the sample to retain its original volume on stress relief due to the presence of gas (ii) loss of moisture on extrusion and preparation (iii) usual disturbance caused to a soft soil sample as it is forced into a retainer, etc.

(c) Inherent Testing Errors

(d) Errors in Interpretation of Laboratory Results

(e) Shear Movements

Large shear movements are likely to occur in the field. These movements do not appear in laboratory consolidation tests.

(f) Underlying Soils

It is common to find soft soils underlying peat deposits. This is largely due to the fact that peat prevents the formation of a dessicated zone and its low effective weight produces little consolidation in the underlying soil.

However, these soils are usually more susceptible to analysis and by means of laboratory testing and field observations on settlement plates installed at the bottom of the peat, it is possible to sensibly eliminate their effect from the total settlement picture. Nevertheless, the underlying deposit may present great difficulties with regard to pre-consolidation and, of course, stability. However, only the behaviour of peat is considered in the present paper.

In view of the above sources of error, it is unlikely that accurate predictions of settlement can be made. However, in modern highway construction very accurate estimates are usually not required since long-term settlements are generally of such a magnitude that it is necessary to remove potential settlements before the highway is opened to traffic. To this end the principle of pre-consolidation has been applied (2) (3) (9). This principle is illustrated on Fig. 2.

Pre-consolidation

The complete success of pre-consolidation as a highway construction technique in peaty soils has not, to the authors' knowledge, yet been proven. That is to say, observations have not been taken for a long enough period such that it can be maintained that pre-consolidation was successful in preventing distress to the road surface for the period chosen beforehand.

However, laboratory evidence indicates that surcharging could be a successful approach (6) (10) and field results are available which look encouraging. These results are summarized in Table I.

Most peats exhibit little rebound on unloading which is a helpful characteristic.

TABLE I

Project	Max. Depth of Peat	Time - Surcharge removed until latest observation (months)	Remarks
Maillardville Test Section	18'	19	2" of settlement has occurred. However it is considered that this surcharge was removed too soon.
T. C. H., near Maillardville	20'	5	No settlement
T. C. H. Sperling Ave.	15'	36	No signs of distress
T. C. H. Chilliwack	35'	12	No signs of distress
Hart Highway	27'	9	No signs of distress
N. T. P. H. (Tamarac Lake)	13'	10	No signs of distress

The effectiveness of pre-consolidation will of course depend on the fact that a reasonable ratio of surcharge load to final embankment weight can be obtained and that negligible continuing shear movements will occur under the final load. With high design loads it may be impossible to fill to the required surcharge heights. For these reasons it is unlikely that the technique will be successful under high fills without recourse to lightweight materials.

Nevertheless, the information obtained to date indicates that pre-consolidation is likely to be a useful approach to highway problems in peat areas if it is applied to fills of moderate heights.

It is definitely a very economical procedure where deep extensive deposits intersect major highway routes (2).

When pre-consolidation is used, the problem resolves to one of estimating the approximate settlements expected in the design life of the structure. This will suffice for calculating the fill quantities and will be the basis for estimating the surcharge required to remove the settlement in any given time. Field settlement plates and piezometers can then be installed and settlement vs. time curves plotted. As the project progresses, these curves will indicate what adjustments to the surcharge are required.

The successful application of this method hinges on positive answers to the following questions:-

(a) Can even approximate estimates be made of settlement and surcharge to remove this settlement in a given time?

(b) Can the field curves be interpreted with sufficient accuracy so that conservative, yet economical, adjustments can be made to the calculated surcharge?

(a) Calculation of Settlement and Surcharge

Settlement

Since it is likely that all settlement calculations will be approximate, then there are several methods of settlement prediction available:-

1. The first and crudest method is to assume that the settlement will amount to some proportion of the height of fill or thickness of the peat layer depending on the experience with the area. For example, for some projects in British Columbia, it has been found that the settlement amounted to about 50% of equivalent height of the applied load up to a maximum value equal to 1/2 the thickness of the peat layer.

2. In some previous work it had been found that the rate of settlement was approximately proportional to the square of the thickness of the drainage path for the time observed (6) (9). If this were the case, then laboratory and field settlements could be related by means of the "square rule" (9). However, there are no theoretical reasons for assuming that this rule, rather than another, will provide more accurate results and its use can only be justified on empirical grounds.

This approach was applied to one project in the Vancouver area and the results from about 50 settlement plates have been analyzed. The anticipated 25-year settlements have been obtained by extrapolation from the field curves. These anticipated settlements compare fairly well with the values calculated from laboratory tests and the "square rule".

3. There is what is probably the most rational approach available at present, that of separating primary consolidation and secondary compression (c.f. (4) (11).

The primary phase can be calculated from void ratio-pressure curves as for inorganic soils (e vs. log. p curve). In laboratory tests on British Columbia peats, the actual primary consolidation takes place very quickly. Consequently, if the load is left on for 24 hours, the decrease in void ratio will be composed of both primary and large secondary effects. In order to minimize this effect, it may be better to remove the load when the pore pressures are no longer significant and proceed to the next load. This will make it possible for more samples to be tested and give greater statistical significance in this very variable material.

Longer term tests could then be run to properly define the secondary stage.

After the significant pore pressures have dissipated, the laboratory void ratio-log. time plot is likely to become a straight line. From the slope of this line the coefficient of secondary compression, C_s (i.e. the amount of compression per unit thickness of soil occurring in one log. cycle of time) can be obtained. The secondary settlement can then be calculated as follows:-

$$S_{sec} = C_s H \log \frac{t_2}{t_1}$$

S_{sec} = secondary settlement

C_s = coeff. of secondary compression

H = thickness of layer (after primary settlement completed)

t_1 = time for primary consolidation

t_2 = time for total settlement

$t_1 - t_2$ = time over which it is desired to calculate the secondary settlement

Since it is a logarithmic relationship, the calculation is fairly insensitive to errors in the assumed value of the time for primary consolidation.

The total settlement in any given time can be obtained by adding the primary and secondary phases.

The authors have no evidence as yet on the efficacy of this method when dealing with the compression of peats but in principle it is used for inorganic soils which exhibit secondary compression.

Figs. 3 and 4 show the relationship between the compression index C_c and moisture content for three peats. Plots such as these, if also made for C_s might enable laboratory work to be reduced considerably.

Surcharge

As in the case of predicting settlements, it is unlikely that accurate surcharge values can be estimated from the results of laboratory tests. Two possible approaches will be considered:-

(i) The first method is completely empirical and assumes that a surcharge equal to 50% of the final load of the embankment will be required if the 25-year settlement of the embankment is to be removed in about six months under the surcharge loading.

(ii) The second method involves the use of laboratory consolidation tests and has been described at length elsewhere (3) (9). Fig. 5 illustrates the principle of the method.

There is no convincing field evidence which shows that this method gives a more accurate time-surcharge relationship than method (i). In fact it has, in some instances, proved under-conservative where the first approach gave adequate results.

Method (ii) involves extensive laboratory testing and, due to its theoretical deficiencies and the unconvincing field evidence to date, it appears that, generally speaking, a less detailed approach is warranted.

Method (i) has the advantage of being very quick and simple to evaluate and, since the amount and time of surcharge loading is subject to field adjustment, it may be that it is the best approach so far available. A minimum surcharge height of 4 ft. of fill has been adopted in highway construction projects in British Columbia.

(b) Interpretation of Field Settlement Curves

Field loading and settlement curves are shown on Fig. 6. From the settlement curve the extrapolated 25-year settlements can be obtained for three different loads. These values are then plotted as points "A", "B" and "C" on Fig. 7. Line DE is a load line giving the load which would result if grade was maintained at any given settlement. The slope of the line is determined by the weight of fill and the water table conditions.

AC and DE intercept at point F and this represents the settlement which has to be removed in order that no further compression will occur under the design embankment loading.

If, at the time of the calculation, this settlement, plus some tolerance, has actually taken place, then the surcharge may be removed. If this is not the case, then more surcharge must be added or the existing surcharge retained for a longer time. The actual values of additional surcharge or increased time can be obtained from the time-load-settlement relationship.

An attempt should be made to construct the fill as quickly as possible, compatible with stability to height, h, (Fig. 6) so that the straight line relationship can be established under at least this load and the surcharge loading. The example shown has three points, A, B and C but this is not always possible; in fact it is often difficult to obtain two points accurately. If the slope of the line gives an obviously wrong answer, or only one point can be obtained, then a guide as to the real slope (or curve) can be obtained from laboratory tests on samples taken close to the settlement plates in doubt.

This approach is based on the premise that the settlement-log time curve remains straight for 25 years. Eventually it must level off unless the thickness of the peat decreases to zero. However, the assumption that it is straight for 25 years is likely to be somewhat conservative.

Research at Hokkaido, Japan (10) has indicated that the curve obtained from plotting field settlements vs. time (arithmetic scales) is hyperbolic in shape. This curve can then be extrapolated to any time and does not involve the inconsistency which occurs at longer times with the other method.

Shear Movements

During the calculations care has to be taken to separate settlements resulting from primary consolidation and secondary compression and those caused by shear movements which occur due to overstress (i. e. movements which occur when a significant amount of the subsoil is stressed beyond its "creep" strength).

This separation can be effected by means of lateral movement gauges. These gauges are placed at various distances from the toe of the fill and are read for vertical and horizontal movement. The volume displaced can then be roughly estimated and a correction applied to the settlement readings.

On two projects in British Columbia, the lateral movements have been checked by means of a slope indicator (13).

Sand Drains

During the construction of fills on soft compressible inorganic soils of low permeability, sand drains are sometimes utilized. Vertical piles of sand, some 16 ins. in diameter, are driven at close centers to a depth dependent upon the thickness of the compressible layer. The fill is then constructed over top. These drains reduce the length of the drainage paths and are supposed to speed up the dissipation of pore pressures and so increase the rate of settlement and gain in strength. However, in this paper it is postulated that the settlement of a load placed on peat is composed of both primary and secondary effects. It is also suggested that the primary phase occurs quickly (at least in B. C.) and that a secondary compression is not a function of pore pressure dissipation. If this is indeed the case, then it is unlikely that sand drains will have a beneficial effect on the rate of settlement.

The published field evidence on this point is rather vague but the general conclusion seems to be that sand drains should not be used when dealing with highly organic soils. However, if peat exists in conjunction with inorganic compressible soils, some advantage may result.

Based on their own experience with sand drain installations in British Columbia, the authors concur with this general conclusion.

Some Suggestions for Further Study

(i) There is a great need for comparison between settlement predictions from laboratory results and actual field settlements. A considerable amount of laboratory and field data is available but seldom are both for the same job.

(ii) Attempts should be made to correlate C_c and C_s with the more easily measured properties such as void ratio and moisture content.

(iii) All settlement observations should be plotted against depth of peat and superimposed load for various peat classifications. An approximate relationship may become evident which, in some cases, could provide a satisfactory means of settlement prediction, especially if pre-consolidation is to be employed.

(iv) In order to fully establish the effectiveness of pre-consolidation, a careful check should be kept on the condition of all highways where the procedure has been used.

(v) It is essential that accurate records be kept of the ratio of surcharge to design load and time of surcharge application on projects which utilize pre-consolidation. The depth and type of peat and underlying soils should also be recorded.

Conclusions

It is considered that the total compression of a peat layer under load consists of a primary and secondary phase. The primary phase is associated with a dissipation of pore pressure. The secondary phase continues for years after the pore pressures have dissipated. The secondary settlement appears to be proportional to the logarithm of time and may become large when considered over the expected life of a highway.

There are many factors influencing the accuracy of settlement predictions and it is suggested that only approximate settlement estimates can be made. However, this fact is not important if pre-consolidation is employed since, in this case, an attempt is made to remove most post-construction settlement by means of surcharge.

Evidence is presented which suggests that pre-consolidation is a useful technique for major highway construction over muskeg. Three methods of calculating the long-term settlements are outlined although it is admitted that the estimates will only be approximate. Two of the methods described involve laboratory testing.

Two methods of calculating the surcharge required in the pre-consolidation procedure are described. The first method is completely empirical and is recommended since the surcharge is subject to revision in the field.

A method of adjusting the surcharge from field observations is outlined.

It is pointed out that compression and shear movements have to be separated in the settlement readings.

Sand drains are not considered to have much beneficial effect in peaty soils.

Finally, it is suggested that more study of the subject is required, especially study of field behaviour. Some recommendations along these lines are made.

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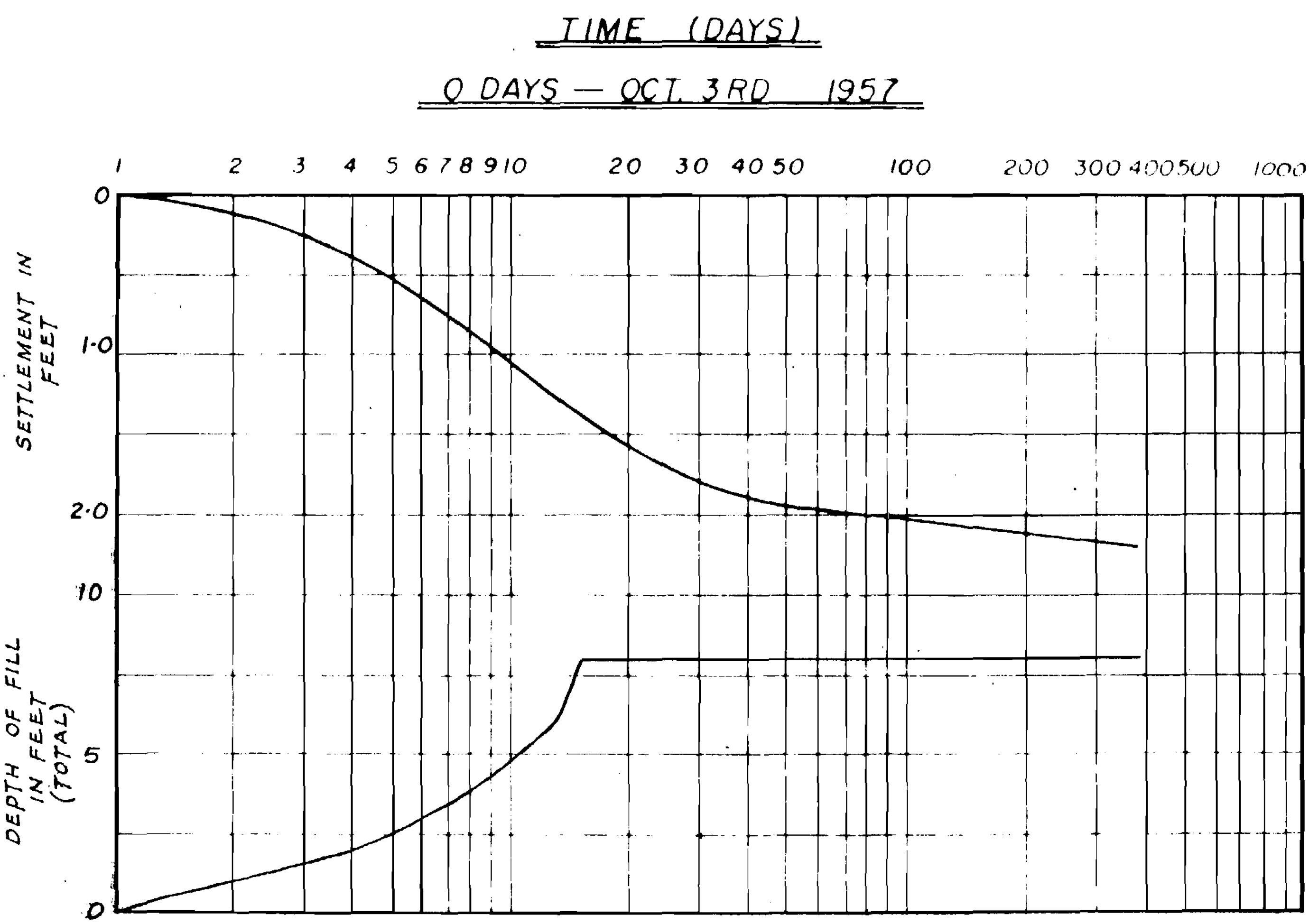


LULU ISLAND TEST SECTION

PEAT SETTLEMENT CURVE

FIG. I.

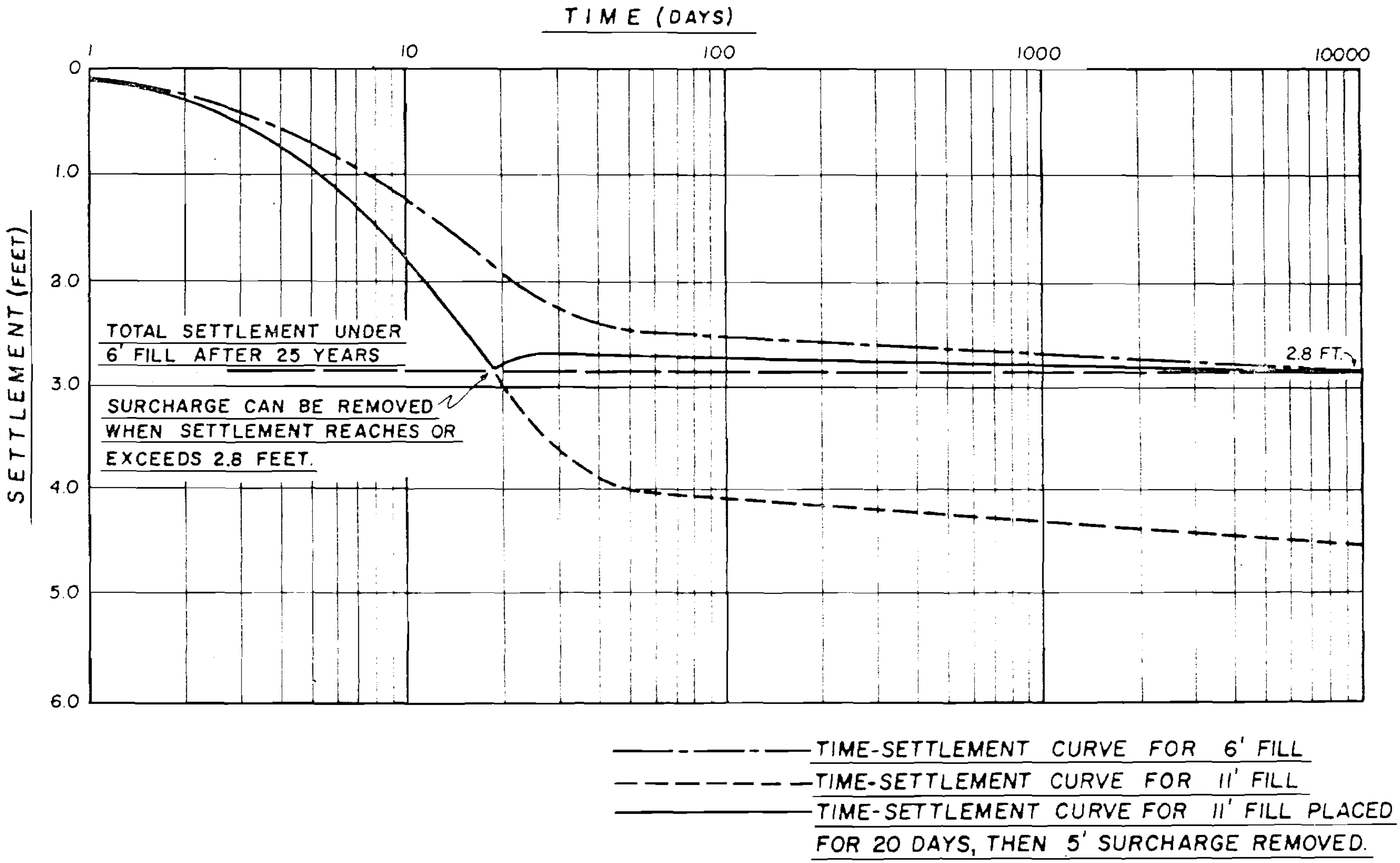


FIG. 2 - TIME SETTLEMENT CURVES ILLUSTRATING PRECONSOLIDATION PRINCIPLE.

VOID RATIO vs. Cc.

TWO — BC. PEATS.

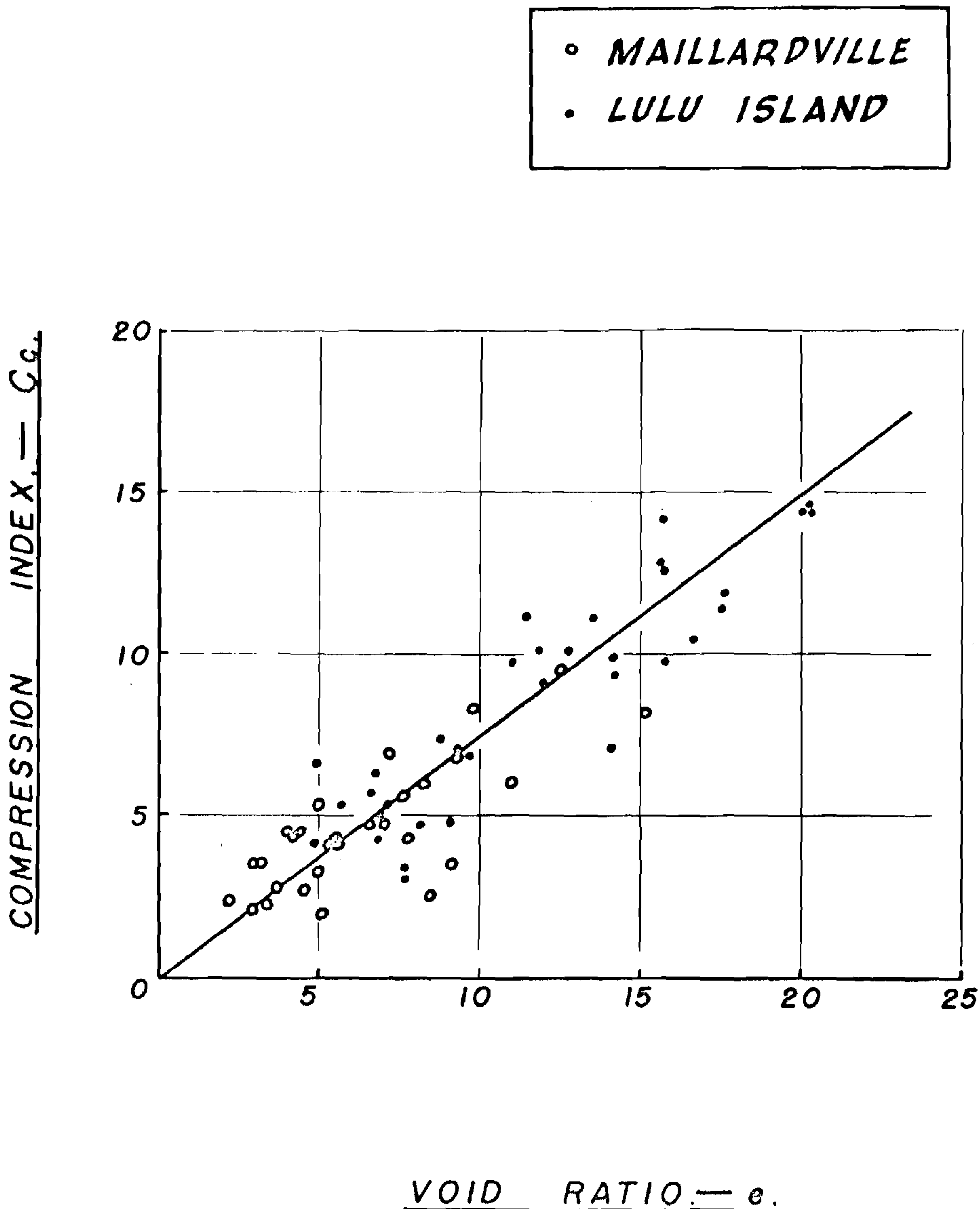
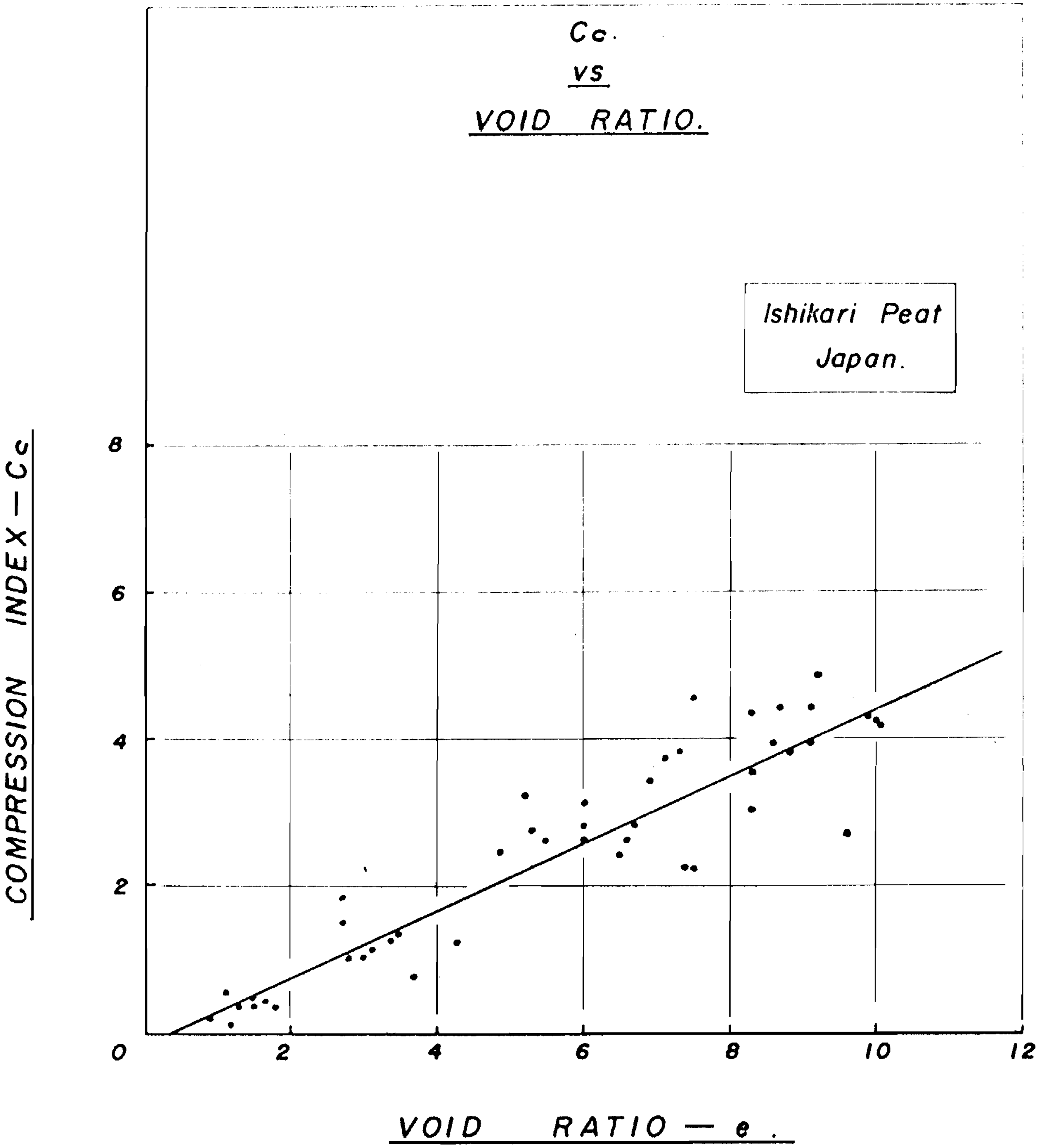
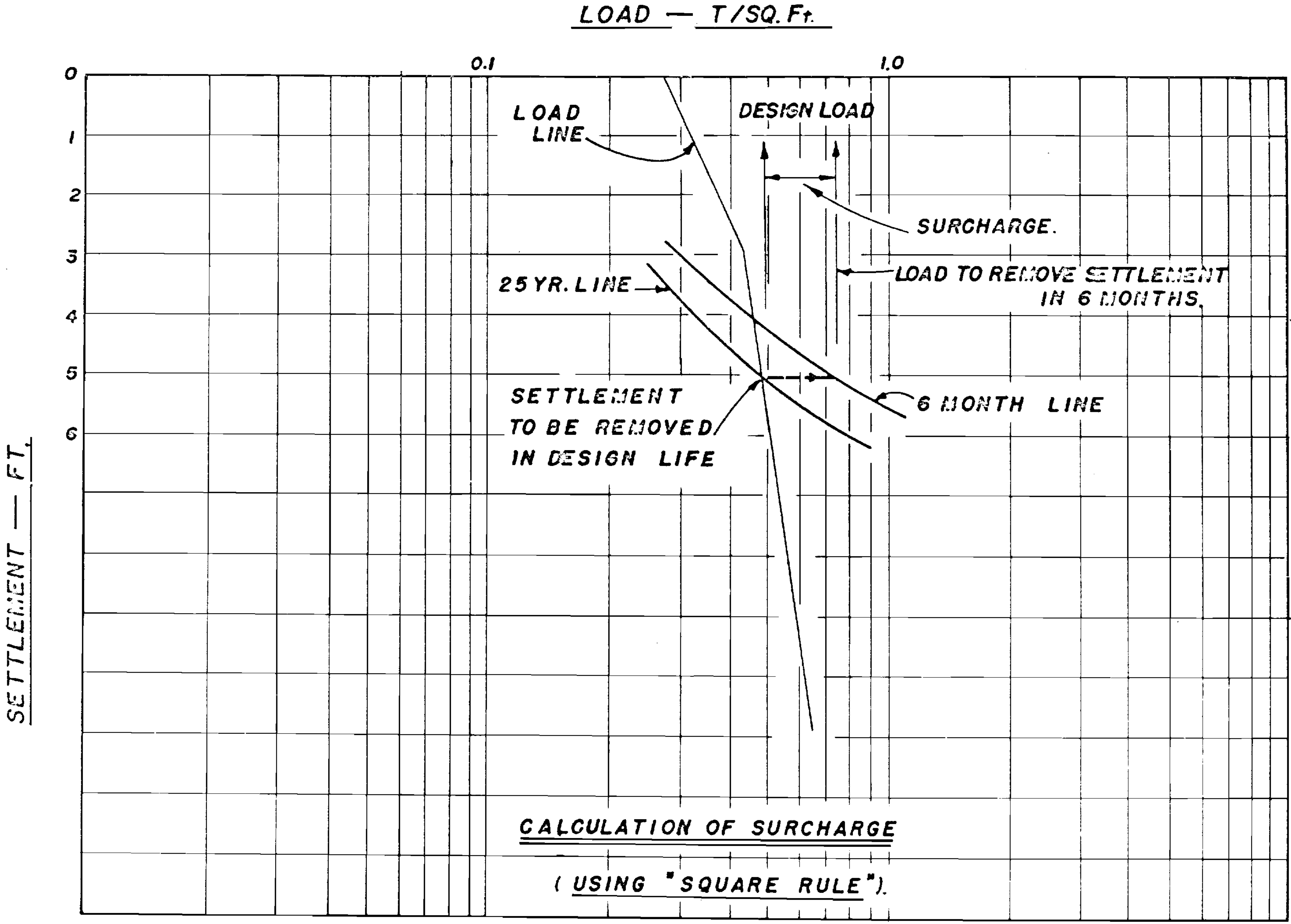


FIG. 3.



Cc.
vs
VOID RATIO.
Ishikari Peat
Japan.
COMPRESSION INDEX — Cc
VOID RATIO — e.
0
2
4
6
8
10
12

FIG. 4.



CALCULATION OF SURCHARGE

(USING "SQUARE RULE").

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FIG. 5

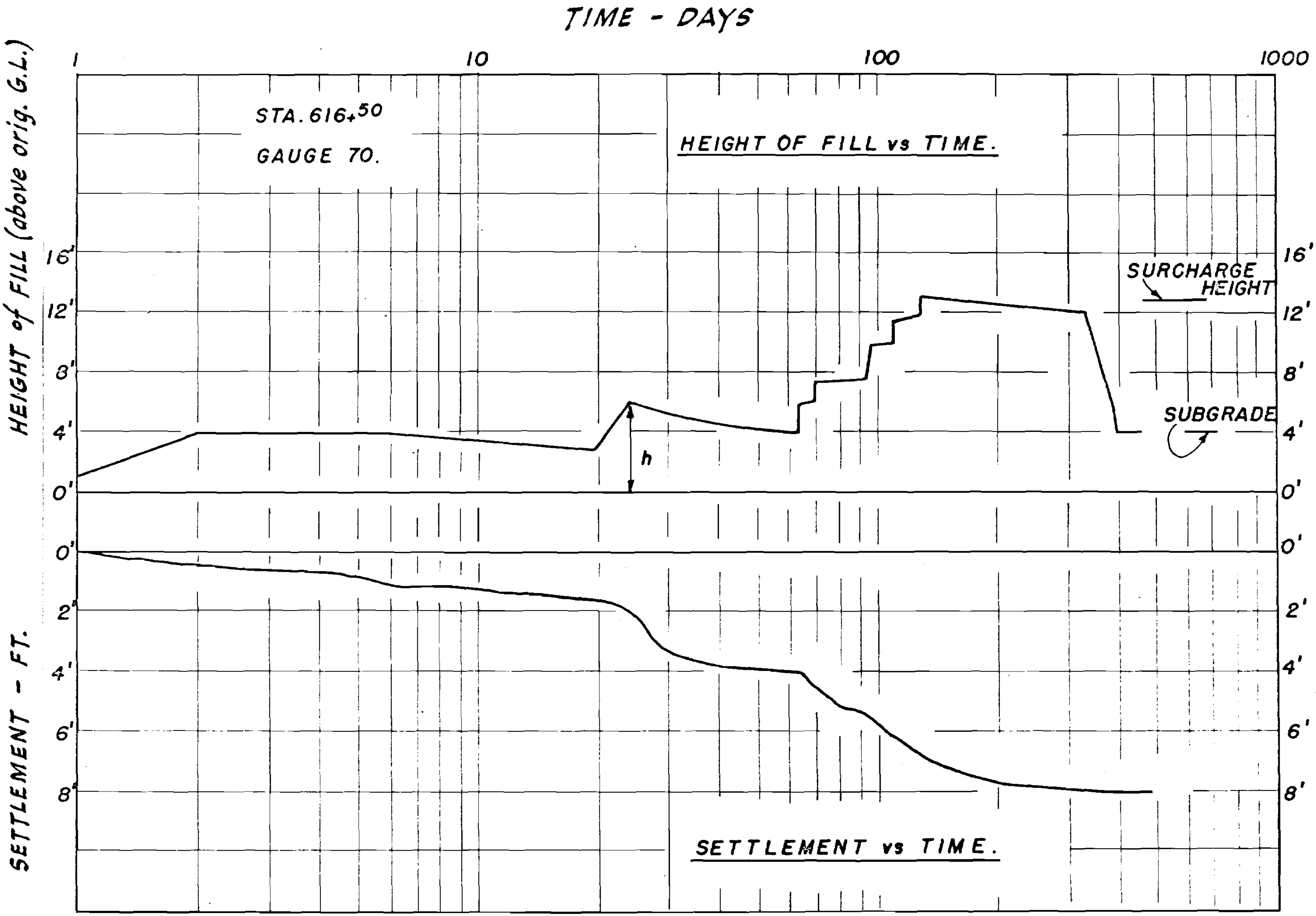
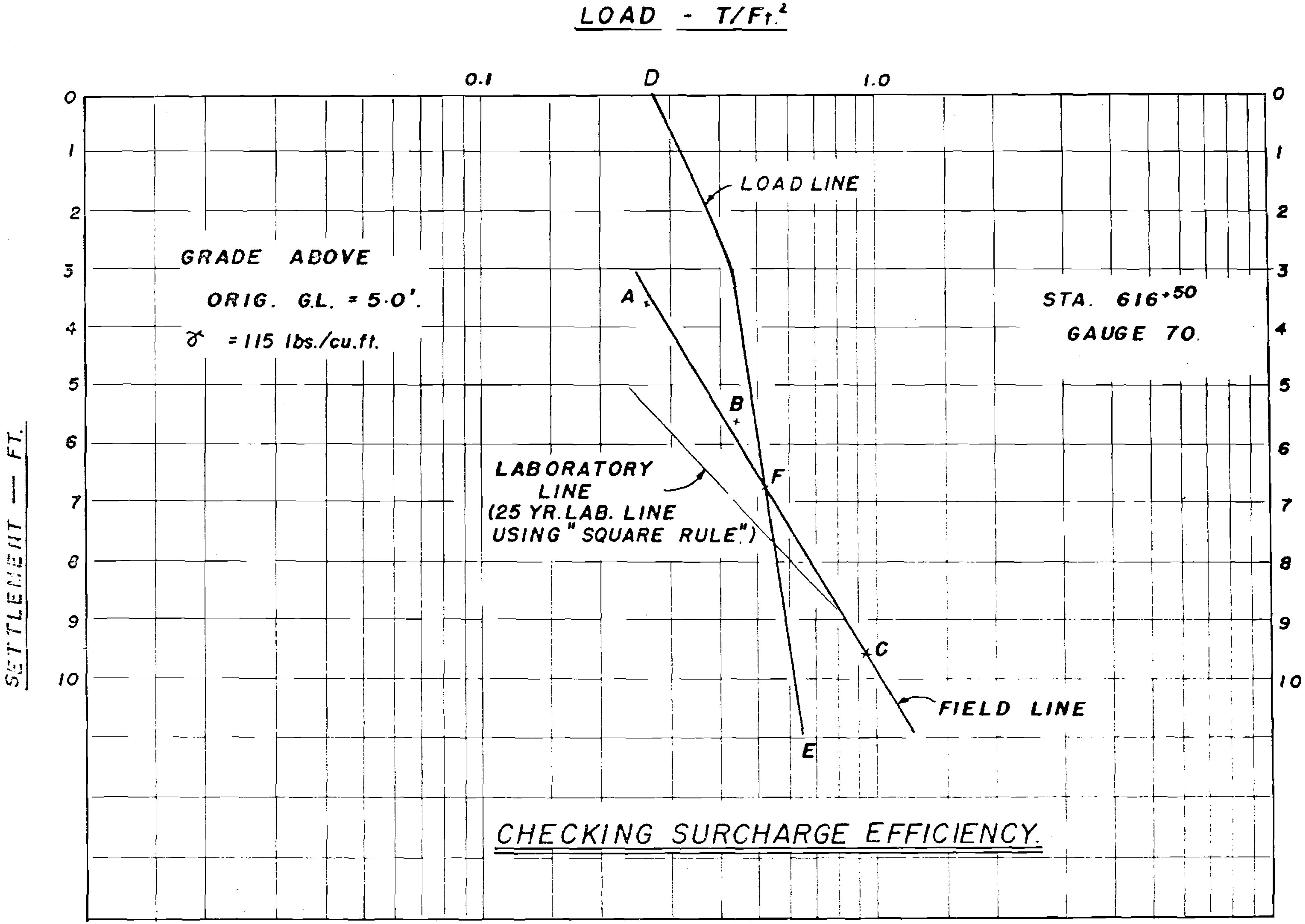

TIME - DAYS
1
10
100
1000
HEIGHT of FILL (above orig. G.L.)
STA. 616+50
GAUGE 70.
HEIGHT OF FILL vs TIME.
16'
12'
8'
4'
0'
SURCHARGE HEIGHT
SUBGRADE
h
0'
2'
4'
6'
8'
SETTLEMENT - FT.
SETTLEMENT vs TIME.

FIG. 6



CHECKING SURCHARGE EFFICIENCY.

FIG. 7.

Discussion

Dr. Chu wondered about differences between the peat types described by Brawner and Ripley, such differences as water content, etc. Mr. Brawner acknowledged that variability was very possible; however, moisture contents of the peats with which he was concerned were in the general range of those mentioned by Mr. Ripley. This is also true for void ratio and compressibility. There is some difference in the material, though, but they do not know what it is.

Mr. Harwood (Defence Research Board) observed that long-term observations initiated by various organizations should be continued by the National Research Council.

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IV. FIELD TRIP

A field trip to the Copetown Bog near Hamilton was arranged in conjunction with the conference and lasted for most of the afternoon. Its purpose was to enable those present to examine a confined muskeg and to have demonstrated the practical use of the Radforth Classification System for muskeg, and also to have a demonstration of some field sampling and testing equipment. The latter was not possible since a very high water table prevented easy access onto the muskeg. About fifty-five people attended the field trip, which was under the direction and guidance of Dr. N. W. Radforth. (For map of Copetown Bog, see Paper I. 3, Fig. 9).

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APPENDIX "A"

LIST OF PEOPLE ATTENDING THE SEVENTH MUSKEG RESEARCH CONFERENCE, HAMILTON, ONTARIO - 18 AND 19 APRIL 1961

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