Proceedings of the Eighth Muskeg Research Conference

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NATIONAL RESEARCH COUNCIL OF CANADA

ASSOCIATE COMMITTEE ON SOIL AND SNOW MECHANICS

PROCEEDINGS

OF THE

EIGHTH MUSKEG RESEARCH CONFERENCE

17 AND 18 MAY 1962

Prepared by

I. C. MacFarlane

TECHNICAL MEMORANDUM NO. 74

OTTAWA

OCTOBER 1962

FOREWORD

This is a record of the Eighth Annual Muskeg Research Conference which was held in the Arts Building of the University of Saskatchewan, Saskatoon, on 17 and 18 May 1962. 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.

A wide variety of subjects was considered, including road and railway construction over organic terrain, trafficability studies, density and water measurements in peat by radioactive methods, factors governing the selection of culverts for muskeg areas, corrosion problems and compression characteristics of peat, together with the fundamental question of why and how does muskeg occur. In Session I, under the chairmanship of Mr. C. O. Brawner, three papers were presented. Four papers were presented at Session II, Dr. J. B. Mawdsley, chairman. The chairman of Session III was Mr. I. C. MacFarlane; four papers were presented. Mr. R. A. Hemstock was chairman of Session IV - - a conspectus on the question of why and how does muskeg occur.

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INTRODUCTORY REMARKS

Dr. N. W. Radforth introduced Dr. J. B. Mawdsley, Dean of Engineering of the University of Saskatchewan, who spoke briefly of his experiences with muskeg in his capacity as Director of the Institute for Northern Studies. He then introduced Dr. J. W. T. Spinks, President of the University, who extended a cordial welcome, on behalf of the university, to delegates to the conference. He expressed his pleasure that the Associate Committee on Soil and Snow Mechanics had decided to bring the muskeg conference to the University of Saskatchewan. Dr. Spinks spoke briefly of the facilities provided by the university and stressed that a strong emphasis is placed on an ambitious research program in order to develop a high degree of academic excellence.

Dr. Radforth then briefly explained the background of the Muskeg Subcommittee and of the Associate Committee on Soil and Snow Mechanics and read a telegram of greetings and best wishes from Dr. R. F. Legget, chairman of the Associate Committee, who sent his regrets that he was unable to be present.

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I. 1

MUSKEG TRAFFICABILITY RESEARCH PROGRAM
VEHICLE MOBILITY LABORATORY, CARDE

by

W. J. Dickson* and E. G. Leger**

INTRODUCTION

The Vehicle Mobility Laboratory (VML), was set up at the Canadian Armament Research and Development Establishment (CARDE) at Valcartier, Quebec, in 1958. The first three years were spent in planning a test program for mineral soils and in the design and construction of the necessary test facilities. That phase of the work is now well established and planning is underway to tackle the organic terrain problem.

A literature review reveals that relatively little organized effort has been expended on basic research into the muskeg trafficability problem. Up until 1955 when Mr. MacFarlane prepared a bibliography (1) there had been only two references to this problem in the open literature. These appeared in the Petroleum Engineer of Aug. 1954 (2) and in Civil Engineering of Aug. and Oct. 1954 (3). They dealt with the Gulf Oil Company's "Muskeg Buggy" which was built by Bombardier. It carried a 2, 500 lb payload, had a curb weight of 5, 000 lb and was powered by a 116 H.P. Crysler Industrial engine. Its nominal ground pressure was 1. 5 psi. The vehicle was 7 ft wide and 12 ft long.

The Canadian Army carried out a project following World War II. The object of this project was to determine the traversing ability of vehicles, available to the Canadian Army at that time, over terrain around Churchill, Manitoba, in summer. The terrain was variously described as muskeg, floating bog, marsh, sloughs, etc. most of which would now be called muskeg of one classification or another. The trial team concluded that wheeled vehicles were unusable on organic terrain, - as were high pressure tracked vehicles such as bull-dozer tractors. Low ground pressure (3 psi) tracked vehicles could, however, negotiate much of the terrain provided they avoided open water, but they suffered breakages and rough riding in wooded and hummocky areas. Also it was found that the drivers needed considerable experience. These facts are now axiomatic.

Although the Vehicle Mobility Laboratory had been functioning in Ottawa until 1958, when it was moved to CARDE, the main emphasis had been placed on mineral soils and snow. To date, organic terrain has received little attention by the Dept. of National Defence from the point of view of vehicle-terrain interaction. A few trials were carried out in the summer of 1960 to assess the ability of a specific vehicle on muskeg. That vehicle, the "RAT, " was

* See Appendix "A" for Affiliation

** Canadian Armament Research and Development Establishment tested for mobility and reliability on muskeg in the Kapuskasing area. However, the main effort of DND has been to support Dr. N. W. Radforth in his work on aerial interpretation of muskeg (4, 5).

OIL INDUSTRY WORK ON MUSKEG

The greatest contribution to the practical problem of mobility on muskeg in this country has been made by the Oil Industry. In conjunction with the efforts of Oil Companies several vehicles have been designed and built in Canada and the USA to traverse muskeg to a limited degree. These vehicles include:

Nodwell Scout Car
Nodwell Transporter
WNRE Musk-Ox
WNRE Terrapin
Spalding Centipede
Bombardier.

In Scotland a vehicle has been built to travel on the peat bogs. This vehicle, the Water Buffalo, also has seen service on Canadian muskeg.

From the beginning of the Muskeg Research Conferences progress in the access problem has been reported annually by Dr. Radforth (6), by Messrs. A. Hemstock (7, 8), J. Thomson (9, 10) and L. Keeling (11) of Imperial Oil Ltd., and others (12, 13). Techniques have been developed for route selection using air photographs to permit limited travel over muskeg in virgin territory with exceptionally heavy loads (14).

The oil exploration work is divided into three phases: reconnaissance, geophysical exploration and wildcat drilling. For the first two operations the oil industry has found aircraft, helicopters and commercially available vehicles to suit the needs. For hauling 25 to 30 ton components for drilling operations, however, a special vehicle was required. Thomson has concluded that it is impracticable to design a vehicle for one set of circumstances. One must usually contend with hilly country, flats, river crossings and muskeg all in one route. Wheeled vehicles are most economical up to the road head. Beyond that point, tracks must be employed to bring down the ground pressure. It seems, therefore, that several kinds of vehicles are required; but each vehicle should embody the requirements of as broad a spectrum of terrain as possible.

An off-road vehicle capable of traversing muskeg and soft clay requires wide tracks as well as considerable power. If it is to traverse rough terrain it must also have a deep soft suspension system and be very rugged in its construction. This same vehicle is overpowered, cumbersome and uneconomical on firm agricultural terrain and is not at all suited to road travel. One must agree with Thomson that there can be no universal vehicle, but rather a hybrid which can negotiate as wide a set of terrain conditions as possible.

During the summers of 1957 and 1958, Thomson (9, 10) undertook a set of critical experiments to evaluate the effect of several specific parameters, namely nominal ground pressure and position of center of gravity. He found that considerable improvement in performance was achieved by moving the center of gravity forward of the center of track area. In fact, he was able to increase the track loading by about 50 per cent. There is no satisfactory explanation for this phenomenon as yet but it is interesting to note that Dr. Tsumenatsu and Dr. Matsui (15) of Japan also found an advantage in shifting the center of gravity forward. Thomson also found that removing every second grouser from the track improved performance. It is not known whether this is due to less cutting and tearing of the mat. Dr. Tsumenatsu and Dr. Matsui found that it was definitely an advantage to use a track shoe which did not cut the mat.

DESIGN PRINCIPLES

Great strides have been made in the muskeg access problem but one cannot be satisfied until complete access on this terrain has been attained. This may be an impossible goal; but from the military point of view the effort must be made. In the event of military operations on this terrain it is essential to have a higher capability for mobility than the enemy. Research must continue if a vehicle is to be designed to go on the worst type of organic terrain. From his long experience in this field, Dr. Radforth (16) has deduced seven fundamental requirements or principles for a vehicle to go on muskeg. Accordingly a vehicle must have:

1. Full buoyancy in open fresh water.
2. A winch co-ordinated with the track action.
3. An adequate approach angle on the tracks.
4. Appropriate flexibility in the lower run of track.
5. Optimum thrust relative to the structural cohesion of the terrain.
6. Center of gravity and load distribution appropriate to the amplitude of irregularity in the terrain.
7. Lowest possible ground pressure.

Thomson (17) has set down a suggested design procedure. He recommends that the designer should:

"1. Identify the role of the vehicle accurately in terms of mobility and function.
2. Establish the range of conditions through which the vehicle will be required to operate and then determine:
   a. Allowable ground pressure for the softest terrain.
   b. Power requirement for the highest motion resistance.
   c. Structural ruggedness required in the machine for the roughest terrain.

3. Conduct field work with existing vehicles to establish the feasibility of the solution devised from 1 and 2 above.
4. Design and construct a prototype vehicle and apply to field service. Proving ground evaluation does not duplicate field conditions and is therefore not a useful development step."

DEFENCE RESEARCH BOARD PROGRAM

The principles cited by Radforth and Thomson provide a guiding light. While they offer only qualitative information, these principles suggest what must be evaluated to put them on a quantitative basis. The need for buoyancy and a synchronized winch on the very weak muskegs are self evident when one observes current vehicles operating on such terrain; but the optimum geometry of the track, power requirement and location of center of gravity for maximum mobility on a given type of muskeg must be evaluated. These parameters must be related in some way to the geometry or microtopography of the terrain and to the strength of mat and underlying peat. Presumably size is also a parameter. To a large vehicle, hummocks and mounds are only ant hills; but to a small vehicle they may look like drumlin size hills.

It is generally accepted that sandy soils can tolerate a larger unit ground pressure from a large vehicle than from a smaller one. Most loam types show an increase in strength with depth and can also be expected to tolerate larger unit pressures under large vehicles. It is interesting to note, however, that a theoretical analysis at CARDE suggests that a deep homogeneous cohesive material may tolerate a smaller ground pressure as the size of vehicle increases. If this is true, - and peat is analogous to weak cohesive mineral soils, - there may be some limiting size of vehicle that can negotiate deep peat bogs. This is pure speculation with no factual evidence to support the theory. Nevertheless, it may be that there is some optimum size of vehicle for travel on muskeg. It must be large enough to crawl over the vegetation; yet it may be limited in size by the underlying peat.

The question inevitably arises as to how to attack the problem. Can it be solved in the laboratory or must it be done in the field? Industry finds it is not economical to employ the long term terradynamics laboratory approach. Industrial operations require an immediate solution. So do military operations in time of war; but during peace time the Defence Department is able to use the laboratory approach which it is hoped will complement the work done by industry. There is now a vehicle mobility laboratory (VML) set up at CARDE which includes a soil bin and dynamometer carriage. Work is being carried out on sand and on soft clay. The soft clay is in some ways similar to peat. Muskeg, however, is more than peat. It is now believed at the VML that even DRB work on muskeg must be done in the field.

The VML program has been planned in four phases:

1. Select areas of each representative type of muskeg according to Radforth's System of Classification (18).
2. Make a microtopographic survey of the surface of each type.
3. Measure the surface and sub-surface strength of each type.
4. Finally attempt to determine the optimum vehicle geometry and power requirement to negotiate each type.

The first step has begun. During the summer of 1961 an air and ground reconnaissance was made of organic terrain north of Quebec City. Patches of 13 types of muskeg have been located on which to launch the program. Air photos of each type will be taken at different altitudes to provide graphic illustration of the undisturbed terrain for later reports. This is intended to provide a link with the two Handbooks (4, 5) prepared by Dr. Radforth and his staff for DRB.

As soon as this is completed the next step will be to make a survey of the surface of each type so that a microtopographic map can be prepared. A sampling and testing program will then be carried out to determine the strength and other engineering properties of the mat and peat. Close co-operation with Mr. MacFarlane of the NRC is planned during this phase. He has done considerable work in this regard aimed primarily at road building and construction on muskeg (19).

This preliminary work is intended to be carried out before vehicles are allowed on the ground. This precaution is taken to ensure adequate illustration of, and measurements on, the undisturbed terrain. When this data is collected the last phase can be planned scientifically. It may require construction of a vehicle test bed consisting only of the running gear and engine, but so constructed that running gear geometry can be varied. The optimum configuration may possibly be evaluated experimentally.

It is believed that one criteria for straight going ability is the product of payload and speed divided by power input. This has been defined as the load carrying index and has units of tons payload x miles per hour per horsepower.

$$M = \frac{W_p V}{P} \qquad (1)$$

where

M is load carrying index tons payload, mph/hp

W_p is payload, tons

V is velocity, mph

P is power input, hp.

It is the intention to install self-contained instrumentation in the vehicle under test and to measure torque at the sprocket and vehicle speed from which to calculate the load carrying index. It may not be practicable to carry out drawbar pull tests on all classes of muskeg.

Therefore, by measuring the power input on level grade it is hoped to obtain the equivalent of rolling resistance which is the power input devided by speed. At the same time it is intended to measure fuel consumption to obtain a measure of the efficiency of the whole vehicle.

CONCLUSION

The CARDE program is at the end of the first phase. Arrangements have been made to have air photographs flown over the area. It is hoped to carry out some of the survey work during the summer of 1962. It should also be mentioned that the U. S. Army Corps of Engineers, Waterways Experiment Station at Vicksburg is carrying out a muskeg research program. This work was started in the summer of 1961 near Parry Sound, Ontario and will continue in 1962. It is intended to maintain close co-operation between the CARDE and WES efforts. A major step toward such co-operation was made in October 1961 when a co-ordinating conference was held at Quebec City. Those attending were representatives of all the interested military agencies in Canada and the USA who are concerned with organic terrain.

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DISCUSSION

Mr. Harwood wondered about the advantage of articulated vehicles as compared to non-articulated vehicles. Major Dickson replied that he was not yet sure that there was any advantage with respect to straight going ability. However, there is an advantage with regard to steering. In articulated vehicles, steering is accomplished by bending the vehicles about a pivotal point. There is no problem of cutting off power on one side to steer. Theoretical analyses thus far, however, have not shown any obvious advantage to articulation. Major Liston was interested in the comments regarding the relation between ground pressure and vehicle size and wondered if this has been borne out by the RAT and other vehicles. Major Dickson said that he had no factual evidence for his conclusions in this regard; they were purely from analytical soil mechanics. Major Liston also wondered how the RAT and the TRANSPORTER compared in muskeg. Major Dickson replied that they have not been compared in the field. Dr. Radforth pointed out that Major Dickson was with a group near Parry Sound last fall and observed a vehicle bog down in a confined muskeg with a depth of about 8 ft, and then get itself out. If it could not have extricated itself, it would have had to stay there, as the nearest assistance was at least 4 miles away. This aspect obtains all across Canada; the vehicle must be able to extricate itself when it becomes bogged down, which points up the importance of a synchronized winch and track. Dr. Radforth announced that the Organic and Associated Terrain Unit of McMaster University is undertaking the sponsorship of a trafficability conference in Parry Sound in mid-August, at which it is hoped some of the problems raised here will be shown. It is hoped to get together the people who designed and planned the early phases of various muskeg vehicles and encourage them to tell why they took a particular approach. Dr. Radforth then wondered if it would be appropriate to ask why is it necessary for the center of gravity to be moved forward, with respect to ordinary circumstances. Major Dickson replied that this was his own opinion and was not based on experimental results. The tolerable ground pressure is greater at the front than at the rear of the vehicle, since the pressure is not uniform over the track area. When one considers the force triangle of drawbar pull, weight and soil reaction, the resultant probably passes ahead of the center of track area for equilibrium. Mr. Harwood commented that several years ago at Kapuskasing a WEASEL was used which had an adjustable weight which could be moved forward and backwards as desired. The vehicle in motion tends to tip backwards. Also, on muskeg the vehicle throws up a bow wave and is always trying to get over this wave which further aggravates the tendency to tip backwards. Consequently, the ground pressure on the rear grouser plates is greater than the ground pressure on the front grouser plates. He suggested that perhaps the ultimate solution is to have a hydraulic pump and ballast and "trim-up" the vehicle as it goes along.

The question was raised whether Major Dickson had done - or if he contemplates doing - any work on the Rolligon principle in muskeg. Major Dickson replied in the negative, stating that his research program was a study of basic principles and that in field studies tracked vehicles would be used.

Mr. Farmer remarked that the theory has been advanced that for large ground pressure vehicles of considerable length, the foundation will be compressed, thereby making its own firm mat. This is true for snow and he wondered if it held true for muskeg. Major Dickson said that this may be so, but in deeper muskegs, the vehicle may sink excessively and will develop an extremely high rolling resistance.

Professor Goodman inquired if any work has yet been done on the shearing resistance and settlement (or sinkage) characteristics of the soil to be used in the research program. Major Dickson answered that the project is still in the very preliminary stages and investigations of this sort have not yet been carried out.

I. 2 MUSKEG AND THE QUEBEC NORTH SHORE AND LABRADOR RAILWAY

by

B. M. Monaghan

INTRODUCTION

The Quebec North Shore and Labrador Railway, a subsidiary of the Iron Ore Company of Canada, links the mining town of Schefferville with Sept-Iles, a seaport on the north shore of the Gulf of St. Lawrence. It was constructed at a cost of $325, 000. 00 per mile, is 360 miles in length, has been in operation since 1954, and was the first railroad - indeed, the first surface transportation route of any description - to serve the interior of the Labrador-Ungava peninsula.

Lack of interest in the exploration and development of Labrador-Ungava can be attributed to a number of factors, most of them related to the rigorous climate of the region and to the problem of access. Because of its unnavigable rivers and streams, the rugged terrain which extends inland from the Gulf of St. Lawrence for almost 100 miles presents a natural barrier to the interior plateau. Another obstacle occurs on the plateau itself in the form of extensive areas of muskeg.

When reconnaissance surveys for the Q. N. S. and L. Railway began in 1945, it was generally believed that muskeg would be responsible for serious and persistent difficulties in connection with railroad location, construction, and maintenance. This paper describes the nature of the organic terrain crossed by the Railway, and includes a brief account of the construction procedures adopted.

THE CLIMATE OF THE REGION

Labrador-Ungava has a number of climatic features which distinguish it from other sections of the country. Some of these features are noteworthy in view of their influence on muskeg development and their effect on construction operations.

Sept-Iles, the southern terminus of the Q. N. S. and L. Railway, has a mean annual temperature of 34 degrees Fahrenheit, almost identical with that of Saskatoon. However, the maritime climate of the north shore of the Gulf of St. Lawrence causes midwinter temperatures at Sept-Iles to be five to ten degrees warmer than those at Saskatoon, while midsummer temperatures tend to be about five degrees cooler. The annual snowfall at Sept-Iles is approximately 170 inches, and the annual total precipitation is 42 inches - more than three times that at Saskatoon.

At Schefferville, the northern terminus, the climate is sub-arctic and the area is one of sporadic permafrost. Here, the mean annual temperature of 23 degrees Fahrenheit is comparable with that of Fort Simpson, 150 miles west of Great Slave Lake in the North

West Territories. Again, however, there are important differences between temperature extremes at the two localities; the range from midwinter low to midsummer high temperatures being much less marked in the case of Schefferville. The annual snowfall and annual rainfall totals at Schefferville are 130 inches and 14 inches respectively. The corresponding figures for Fort Simpson are 55 inches and 7 inches.

Climatic conditions at Schefferville are typical of those which prevail over most of the muskeg area traversed by the Railway. Thus, this region is characterized by long severe winters with heavy snowfall - the mean annual maximum depth of snow cover on the ground is 60 inches - and cool summers with persistent light rain.

DESCRIPTION OF THE TERRAIN

The concept of muskeg as "organic terrain" rather than a troublesome type of water-logged organic matter - or "muck" - is a useful one since it involves a consideration of the physiographic features of the areas in which muskeg occurs, and the subsurface materials upon which the organic cover develops (1). These are important considerations to those concerned with problems of access, construction, and drainage in muskeg areas.

The Q. N. S. and L. Railway crosses three of the major physiographic subdivisions of Labrador-Ungava (2): The Labrador Trough, The Lake Plateau, and the Laurentide Massif. To these might be added a fourth - the narrow North Shore Lowland. With the exception of the Laurentide Massif - which poses its own problems of access - muskeg is one of the important features of each of these areas (Fig. 1).

The Labrador Trough, or Labrador Geosyncline, is the "iron belt" of the peninsula; the new mining communities at Schefferville, Gagnon, and Labrador City are all situated in this area of Precambrian sedimentary and volcanic rocks. As a geologic unit it extends in a southwesterly direction from the west side of Ungava Bay for a distance of more than 400 miles and has a maximum width of about 50 miles. Physiographically, the term "Labrador Trough" is used to describe the ridge and valley type of upland which is characteristic of most of the Labrador Geosyncline north of Schefferville. The more resistant formations form the ridges, while valleys have developed in the weaker slates and shales.

The route of the Q. N. S. and L. Railway follows the valleys of this physiographic subdivision for about 30 miles through an area with a mean elevation of 1800 ft and local relief in the order of a few hundreds of feet. Ridges are generally bare of tree cover, and valleys are occupied by many elongated lakes, ponds, and expanses of muskeg. Here, as over much of the Labrador peninsula, an established drainage pattern has been deranged by a mantle of glacial drift. Its clayey texture distinguishes the drift in this area from the sandy and silty soil formations in many other parts of the region.

The Lake Plateau occupies a large drift-covered area in the south central section of the peninsula. The salient feature of the area is the one from which it derives its name - the profusion of lakes, ponds, and string-bogs. The extent to which bodies of open water dominate the landscape is particularly impressive when the ground surface is viewed from the air or examined in airphotos. Another important feature, also conspicuous in airphotos, is the large number of eskers oriented in a roughly north-south direction.

The presence of eskers and scattered outwash deposits, and the occurrence of glacial drift in drumlins (or drumlin-like landforms) does much to mitigate the problems of access presented by great expanses of open water and muskeg. Generally the drift is silty textured and excessively wet; in some locations, particularly in the vicinity of larger lakes, it may be overlain by a deposit of silt which rarely exceeds one or two feet in depth.

The Railway crosses this physiographic unit for a distance of 250 miles, about half of which is predominantly bog. Most of the organic terrain occurs to the north of Milepost 150 where the route crosses the low divide (elevation 2066) separating the headwaters of south-flowing rivers from those of the upper Hamilton River.

The Laurentide Massif is an area of rugged hills situated between the Lake Plateau and the Gulf of St. Lawrence. Summits approach an average elevation of 2500 ft and, apart from the major river valleys, local relief is in the order of 500 ft. The larger streams occupy deep gorges in the northern part of the section and deep glacial troughs in the south. In its ascent to the Lake Plateau the Railway follows the valleys of the Moisie River and several of its tributary streams. The nature of the terrain precludes the development of important areas of muskeg.

The North Shore Lowland is a narrow coastal strip which extends inland to the steep south-facing Laurentide Escarpment. Except where river terraces form northward extensions, the width of this physiographic subdivision rarely exceeds five or six miles.

The north shore of the Gulf of St. Lawrence is an emerging coastline. In the vicinity of Sept-Iles, marine and estuarine sediments deposited during the period of submergence overlie a rocky foreland of low hills, and the coastal plain rises in a series of beach ridges to an elevation of about 400 ft. Scattered areas of muskeg occur in the depressions between adjacent ridges. The Railway crosses the coastal plain for a distance of twelve miles before entering the valley of the Moisie River.

MUSKEG FEATURES

Although aerial survey and airphoto interpretation techniques have done much to simplify and improve the quality of route location work, reconnaissance and location parties must still devote a considerable amount of time to on-foot surveys of the terrain. This was even more necessary in 1945-46 than it is today. Consequently, the early civil engineering workers in Labrador-Ungava soon became familiar with muskeg in the field and, locally, the term came to be applied more or less exclusively to areas of open or lightly timbered bog which could be crossed only with the greatest difficulty - or not at all - except during the winter months.

The depth and area of bog formations were matters of primary interest to those engaged in reconnaissance and preliminary surveys. Since the airplane, or floatplane, generally provided the means of access, there were ample opportunities to view the terrain from the air. This circumstance led to the increasing use of airphotos and to efforts to relate conditions on the ground to the patterns revealed from the air. Three important types of bog were recognized; (a) string bogs - readily identified from the air or in airphotos by the striped appearance of areas in which they predominate ; (b) marshes, in which expanses of grassy vegetation surround pools of open water; and (c) raised bogs, a feature of the North Shore Lowland. The information accumulated on the depth and composition of these formations, although fragmentary, was of value during final location and construction phases of the railway project. In recent years, the results of detailed studies have been published (3), and helpful information on the identification and classification of string bogs is now available (4).

String bogs excited much interest during the initial surveys for the Railway; the original cause for concern being the extent to which they hampered movement on the Lake Plateau. Figures 2 and 3 are, respectively, air and ground views of bogs of this type.

The amount of water associated with string bog areas may be large or small, and local descriptions are likely to vary accordingly. In some situations an observer on the ground might be confronted by a "lake crossed by many narrow bands of waterlogged vegetation;" elsewhere, a "series of parallel ridges separating elongated pools of water" would be a more appropriate description. Occasionally, where grassy vegetation or sedges encroach upon the pools, open water is absent altogether. The "strings" themselves are stable ridges of peat which support a growth of sphagnum moss and low shrubs. Stunted trees are a feature of some bogs.

Efforts were made to determine the more important features of string bogs when it became apparent that construction across them could not be avoided without resort to lengthy diversions or a tortuous alignment. In many instances it was found that the depth to a firm stratum, measured from water level, did not exceed about four feet. Since this was considered to be a tolerable depth of organic material, alignment changes were made only when soundings indicated an appreciably greater depth. Practical importance was also attached to the observation that string bogs occur on gentle - often imperceptible - slopes rather than in topographical depressions. The variation in water levels suggested that drainage measures might meet with some success.

Marshes cover an area comparable with that occupied by string bog. As used here, the term refers to the variety of basin bog which develops as vegetation encroaches on the shores of a lake. On the Lake Plateau and in the Labrador Trough, marshes are characterized by a cover of sedges surrounding scattered pools of open water; the surface mat is frequently underlain by water or semi-fluid organic material, and the depth to a firm stratum is commonly in the order of 15 ft. The term "Sedge Meadow" has been used to describe bogs of this type in the vicinity of Schefferville (3). In the North Shore Lowland area the vegetative cover may include such woody members as alder and the Labrador Tea Plant. From the standpoint of access, marshes are particularly treacherous. During location of the Q. N. S. and L. Railway they were regarded as a more serious obstacle than string bogs and were avoided whenever possible.

Raised Bogs are a feature of the North Shore Lowland where they have developed on relatively flat marine terraces. Figure 4 is an aerial view which shows part of a typical bog in the vicinity of Sept-Iles; it is roughly circular in shape and a little over a mile in diameter. The plotted elevations indicate a drop of approximately 10 ft between the center of the bog and its margin. Precipitation provides the only source of water while a single small stream serves as the only permanent drainage outlet. Sedges and low, woody, shrubs are the predominant forms of vegetative cover.

Noteworthy features of raised bogs include: the variety of materials upon which they develop - sands, silty clays and even rock outcrops; the peculiar drainage characteristics, which are occasionally responsible for the flooding of adjacent areas; and the deep accumulation of peat which generally underlies the surface.

RAILWAY LOCATION, CONSTRUCTION AND MAINTENANCE

Because of the large tonnages involved and the long haul from mine to loading dock, the single track railroads serving the interior of the Labrador-Ungava peninsula must operate long, heavy trains. In the case of the Q. N. S. and L. Railway which hauls up to 13 million tons of ore annually in an operating season of approximately 200 days, nine loaded trains and as many empty trains may occupy the track at one time. It is evident, therefore, that mining railroads in this region must be constructed and maintained to high standards - they are in no way comparable to the secondary highways designed to serve small communities in some remote areas.

Location. Field surveys covered a total distance of more than 1500 miles. They were complicated by the grade and curvature requirements dictated by such economic considerations as optimum train lengths, weights, and speeds. Subsurface studies were confined to bridge sites, and terrain conditions were evaluated on the basis of information derived from airphotos or from the soundings and observations of survey parties. Since it was recognized that construction across the Lake Plateau would necessitate a departure from the traditional method of balancing cut and fill quantities, efforts were made to locate within short haul distances of sources of suitable borrow material. Efforts were also made to avoid encounters with muskeg, particularly those of the marsh variety, but wide detours were held to minimum.

During the years 1945 and 1946, field work was restricted to the summer months. Later, it was undertaken on a year-round basis using dog teams and ski-equipped airplanes during the winter months; floatplanes and canoes in the summer. Helicopters, which were used to great advantage during the construction phase of the operation, have been employed on more recent reconnaissance and location studies, and are now considered to be indispensable for this type of work.

Construction. In this region, construction of a railroad is likely to be the major item involved in bringing a mining property into production. It is of particular importance, therefore, that the task be completed in the shortest possible time, and serious consideration may be given to methods which would otherwise be regarded as extravagant. For example, it may be judged expedient - as it was in the case of the Q. N. S. and L. Railway - to move tractors by air, or to fly 190,000 bags of cement a distance of 300 miles rather than move them overland. As might be expected under these circumstances, the approach to the problem of construction across muskeg areas was a straightforward one.

On the Lake Plateau construction operations were deferred until the winter of 1952-53. By this time a railhead had been established at Milepost 108, an airstrip was in operation at Milepost 134 and others were under construction, and a large force of personnel and equipment became available as the heavy earthwork was completed south of Milepost 100. During the winter months the greater part of this force, which included 56 shovels and 180 tractors, was concentrated in the most inaccessible bog areas. By June 1953, the railhead had been advanced to Milepost 153 and grading operations were completed to Milepost 224.

During the following summer many construction difficulties were averted by close attention to every possible drainage improvement. Offtake ditches were often effective, and in a few instances all surface water was drained from string bogs or shallow lakes and ponds. Drainage ditches were blasted using ditching dynamite or excavated by means of light draglines supported on timber mats (Figs. 3 and 5). Airphotos were of value in determining the most suitable location for many of the longer offtake ditches.

The most satisfactory construction procedure in string bog areas involved the use of draglines working in pairs on opposite sides of the roadbed. These machines - often preceded by ditching parties using dynamite - were used to excavate longitudinal drainage ditches to the full depth of the organic cover. Organic matter and silt was wasted along the edges of the right-of-way, and the underlying granular material was cast onto the centerline of the roadbed to form a trafficable road (Fig. 6). Haulage units working from suitable borrow areas were then used to complete the subgrade. Settlement caused by heavy construction traffic and consolidation of the underlying peat usually necessitated minor amounts of additional fill; this was placed during final grading operations immediately prior to the laying of track.

The few encounters with deep muskeg occurred mainly in the Labrador Trough area where their depth occasionally amounted to more than fifteen feet. Embankments were constructed by end-dumping, and the toe-shooting method of fill settlement was used in some cases.

To varying degrees, all construction operations were complicated by problems of access and transportation. Muskeg received the major portion of the blame for these difficulties, but a number of other factors were also involved. Because of the heavy snowfall and the late date to which snow cover persists, much of the available granular material was placed in a saturated condition. This difficulty was aggravated by the cool, wet, summer weather which made it almost impossible to achieve adequate compaction. Thus, even when the completed (or partially completed) railway subgrade was used as a haul road, construction traffic was frequently in distress. This condition occurred in both cut and embankment sections and was particularly troublesome wherever silty textured soils were involved.

The thawing of ground ice after the insulating cover had been removed was another cause of distress to traffic during the summer months. Scattered islands of permafrost in the northern part of the route sometimes made it necessary to drill and blast frozen ground to depths of at least six feet. More commonly, shallow ground ice was found at sites where conditions were favourable to unusually deep frost penetration. The removal, or compaction, of snow and organic cover during winter haul operations was the most obvious cause, and muskeg sections in which a shallow depth of peat was underlain by silt were the areas generally affected.

The difficulties associated with muskeg led to the adoption of a variety of schemes to minimize road traffic. Over much of the Lake Plateau it was possible to take advantage of the larger lakes; some construction camps were serviced almost entirely by floatplanes or amphibious airplanes; 40-ft boats with an eight-ton capacity served the purpose of trucks in transporting fuel and supplies; and personnel were sometimes transported to work sites by means of boats and canoes. Under these circumstances, helicopters were invaluable for inspection and expediting.

Maintenance. Sections of the Railway constructed over organic terrain required fairly extensive maintenance during the first one or two years of operation; drainage, differential settlement, and frost damage were the principal features.

No important drainage problems were involved, but there was a need to complete work which would have been costly and time-consuming if undertaken during the construction period. In contrast to highway and airfield pavements, the track structure of a railroad is not seriously affected by differential settlement. Train fill methods were used to provide additional coarse-textured fill on which to raise track levels, and also to widen embankments in order that they might accomodate deep sub-ballast and ballast sections.

Frost damage was responsible for heaved track which required attention until June or July, and for soft subgrade conditions that persisted throughout the wet summer months. It was, and continues to be, the most serious maintenance problem in muskeg areas. The frost-susceptible soils responsible for the difficulty are always associated with a surface cover of peat. Since this insulating material reduces the depth of frost penetration, the sections of the Railway most severely damaged by frost during the first years of operation were those in which the organic cover was shallow (or was removed altogether), and where embankment heights were low. Ground temperature measurements made toward the close of the construction stage provided useful information on the depth of frost penetration under various conditions; they were of value in determining the extent to which embankment heights should be increased. Track lifting operations have produced a marked decrease in the incidence of frost damage with the result that the problem is now confined to shallow cuts and to sections where frost-susceptible soils were incorporated in the subgrade. Muskeg areas are entirely free of the slope stability, drainage, and icing problems which account for much of the roadbed maintenance activity in other sections of the route.

During the early life of a new railroad, an increasingly high proportion of all maintenance of way activity involves the requirements of the track structure and ballast section. Heavy loads and the intensity of traffic cause this proportion to be unusually high in the case of the Q. N. S. and L. Railway. From the standpoint of rail wear and track maintenance, sections located in muskeg areas are considered to be relatively trouble-free - a feature attributed to level grades and the absence of sharp curves.

CONCLUSIONS

Interest in Labrador-Ungava has been stimulated to a remarkable degree by the success of this undertaking. Recent achievements in the field of railroad and highway construction include: a 200-mile access road and railroad to serve the iron ore development of the Quebec Cartier Mining Company; an additional 70 miles of main line railroad linking other mining areas and dock facilities with the Q. N. S. and L. Railway; extension of the Provincial highway system by 175 miles in order to serve the growing communities along the north shore of the Gulf of St. Lawrence; and a 105-mile all-weather access road between the Q. N. S. and L. Railway and the site of major hydro-electric scheme on the Hamilton River. The mining industry provides the basis for the economic life of the region, and the Labrador Trough will probably continue to be the primary producing area. Future developments may eventually create a need for highway access to the new mining communities; they will almost certainly necessitate the extension of existing railways or the construction of new ones. Secondary activities such as the development of sources of hydro-electric power and the exploitation of forest resources will continue to require the construction of transmission lines and access routes. It seems inevitable, therefore, that difficulties caused by muskeg - often compounded by the presence of permafrost - will continue to beset the civil engineer involved with construction in this region. The Radforth Classification System, which appeared as a "Suggested classification" during the closing stages of construction of the Q. N. S. and L. Railway, is likely to be of great practical value in future location studies.

It is to be hoped that the results of the current program of research into the engineering properties of muskeg will be equally useful to the engineer in the field.

ACKNOWLEDGMENTS

The author is grateful to Dr. N. W. Radforth for the invitation to present this paper, and to Mr. R. W. Pryer for assistance in its preparation.

Special acknowledgment is due to Professor K. B. Woods and Dr. R. F. Legget. During his long association with the Railway as a soils consultant, Professor Woods has been responsible for many of the techniques used to overcome construction and maintenance difficulties. His contributions are highly valued. Dr. Legget has long been interested in construction problems in Labrador-Ungava, and the members of his staff at the Division of Building Research have provided much useful advice and assistance.

Thepaper is presented with the permission of Mr. W. D. Durrell, Executive Vice-President of Hollinger-Hanna Limited.

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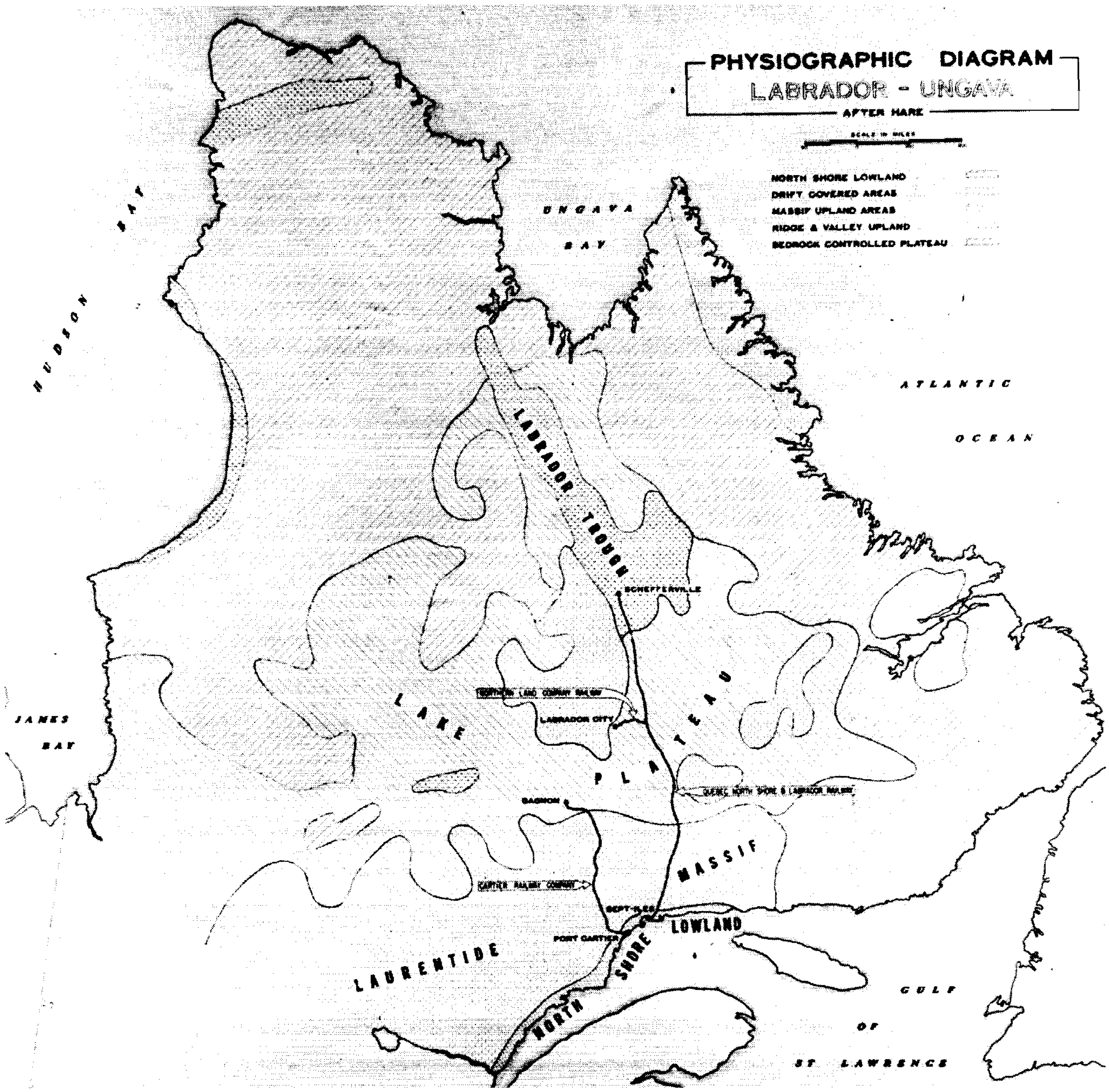

FIG. 1 RAILWAY ROUTES IN RELATION TO PHYSIOGRAPHY OF LABRADOR-UNGAVA

FIG. 2 EMBANKMENT SECTION ACROSS STRING BOG

FIG. 3 EXCAVATION OF OFFTAKE DITCH IN STRING BOG

FIG. 4 RAISED BOG DEVELOPMENT ON MARINE TERRACE IN THE NORTH SHORE LOWLAND AREA

FIG. 5 INTERCEPTING DITCH AT MARGIN OF RAISED BOG IN THE VICINITY OF SEPT ILES

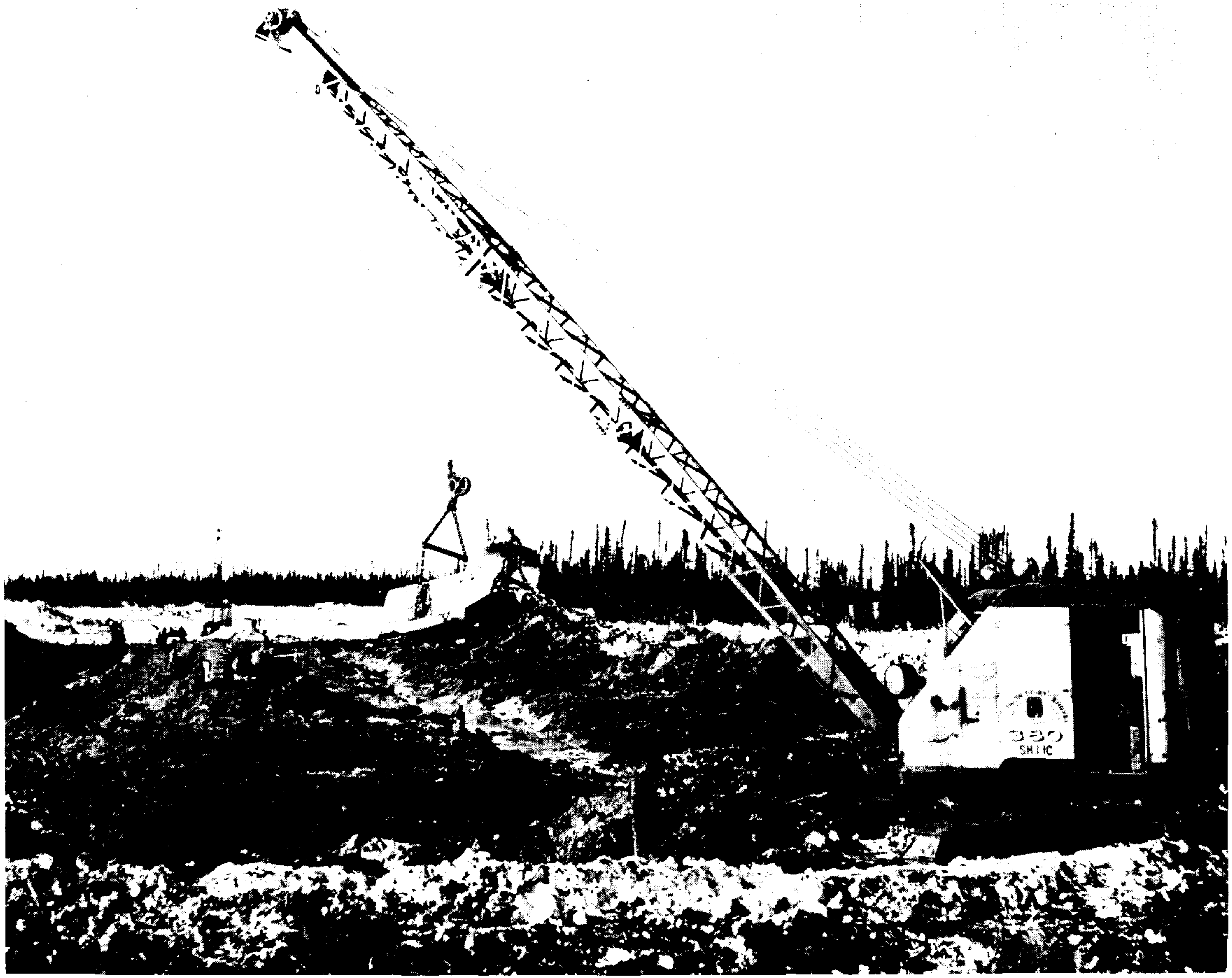

FIG. 6 DRAGLINES WASTING ORGANIC MATERIAL AND CONSTRUCTING TRAFFICABLE ROAD ON CENTRELINE

DISCUSSION

Mr. Hansen referred to mile 198 where permafrost was encountered and asked if this had to be dug out later. Mr. Monaghan replied that this was dug out before the track was laid. The frost-susceptible soil was identified and excavated to 8 ft below rail elevation.

Mr. Evans wondered what approach Mr. Monaghan would take now in muskeg areas on the basis of his past experience. Mr. Monaghan referred to the recent construction of a branch line on which fills were built over muskeg and permafrost areas. Ridges were cut through, the frost-susceptible material excavated to 8 ft below rail elevation and backfilled with granular material. Dr. Radforth observed that he has been anxious over the years that experiences across the country would be shared and co-ordinated for the advantage of all. Consequently, he was grateful to Mr. Monaghan for his paper. He pointed out that a classification system for muskeg has been of interest ever since the first conference. The early work on the Q. N. S. and L. Railway was done in 1945, well before the NRC work on classification was published. Dr. Radforth stressed, however, that the important thing about any classification system is that it be properly defined and is understood by everyone. He said that "string bog" is a good term and in fact is the same feature as the vermiculoid pattern as observed from 5000 ft.

Mr. Harwood asked if the railway was operated during the winter and if snow plowing was carried out. Mr. Monaghan replied in the affirmative to both questions. Mr. Harwood wondered further about muskeg when encountered, if it was filled over or excavated. Mr. Monaghan said that in general the fill was placed directly on the muskeg. Mr. Harwood observed that muskeg adjacent to the track would not be frozen to any great depth, but muskeg under the track would freeze when the snow is removed. He inquired if any differential heaving was experienced in long sections of track over muskeg. Mr. Monaghan said that they do get heaving but no differential heaving. Frost penetration in the muskeg adjacent to the track is 1 to 2 ft, but under the track it is 10 ft. Differential heaving does occur, however, in the transition zones as the track goes from the muskeg to mineral terrain.

Professor Anderson raised the question of suitable borrow material for muskeg and referred to discussions at the last Highway Research Board meeting regarding the use of fine material below water level. He wondered if there had been a set criterion for borrow material. Mr. Monaghan stated that they followed rigidly a soil specification which permitted the use of silt and silty clay after the first 8 ft of fill, which had to be granular. Granular material was always used below water level.

Mr. Keeling was interested in the string bogs with the water at slightly different elevations between the strings. He wondered what has been the experience over the past 10 years in draining these. Mr. Monaghan said that in some locations the drain level was such that the embankment was kept rather close to the water level of the terrain. One string bog area has been completely drained and is relatively dry. The most successful condition is in string bogs with a gradient and with a ditch dug along the line of the gradient. Raised bogs are much more difficult to drain.

Mr. Schlosser asked about the depth range of raised bogs, relative to adjacent organic terrain. Mr. Monaghan replied that raised bogs always occurred in locations by themselves and were surrounded by mineral terrain. One raised bog had a depth of 26 ft. String bogs, on the other hand, very rarely exceed 10 ft in depth and are generally 2 to 4 ft deep.

I. 3 DENSITY AND WATER MEASUREMENTS IN PEAT BY RADIOACTIVE METHODS

by

N. W. Radforth and K. H. Ashdown

INTRODUCTION

It was suggested in a recent paper (1) that density and moisture measurements might be obtained from organic terrain by the use of methods utilising radioactive materials.

These two factors, density and moisture, are basic to any quantitative assessment of peat, but it is difficult to obtain a reliable measurement of either if peat is removed from its in situ position and methods that obviate this bear investigation. The methods are essentially those used for determining density-moisture relationships in mineral soils.

A source of gamma radiation and a suitable detector, both contained in a probe unit, may be lowered into an access tube inserted into the soil to be investigated and the passage of radiation through the soil used to measure density. Similarly, a fast neutron source and a slow neutron detector may be used to measure the hydrogen content of a soil. In the latter case it is then possible to arrive at the moisture content of mineral soils because all or most of the hydrogen present may be assumed to be associated with water. However, because of the bound-hydrogen content of organic matter, a different approach is needed if the moisture content of organic soils is to be determined and the main purpose of the previous paper on this topic was to suggest a method by which this could be accomplished.

This paper is essentially a progress report on this application.

CONSTRUCTION

It was thought preferable to design and construct equipment to personal specifications rather than to procure commercially available instruments because these are generally designed for use in materials of the density range 50-150 lb/cu ft whereas the density of peat usually lies in the range 10-30 lb/cu ft (2). Also, they are usually calibrated for a lower moisture content range than it is expected to be found in peat and, of course, on the assumption that there is no hydrogen content in the material to be measured except that associated with water. Recalibration for both ranges might be feasible but such instruments may weigh as much as 130 lb and it was thought that an instrument designed for use in muskeg areas should weigh considerably less than this.

An instrument of the type required has five basic units: a counting device or scaler, a shielded source of gamma radiation, a shielded fast neutron source and two probe units containing appropriate radiation detectors.

The shielded sources contribute most weight to such an instrument and means were sought to reduce this factor without sacrificing efficiency of operation or increasing radiation hazards or costs. In the case of the gamma source this was facilitated by the need to provide for a relatively low density range - the source does not need to possess the high emissivity of sources used for materials of higher density. Sources of fast neutrons used in commercial instruments have extremely long half-lives but because gamma radiation is also emitted they require shielding material as heavy as, or heavier than that of the gamma source. By choosing a fast-neutron source of little or no gamma emission it was possible to use paraffin wax as the shielding material and so reduce weight considerably. The fast-neutron source that was obtained has a half-life of about three months but this is not considered to be disadvantageous as this relatively inexpensive source can be purchased as and when required for a season's work and again reduce initial expense.

The scaler is the heaviest single piece of equipment and the major weight contributing item is the battery power source. However if a reasonably long period of stable operation is desired, little can be done to reduce the weight. Nickel-cadmium batteries were chosen for their high power/weight ratio.

The completed instrument weighs 85 lb - a reduction of 30 per cent from the weight of a commercial model.

Several delays in assembling the equipment were incurred because of the need to obtain some essential components from overseas. Thus, having decided to use photomultiplier tubes and scintillating crystals as the detector units and wishing to provide as slender a probe unit as possible, it was found that the smallest diameter photomultiplier then available was manufactured in Holland. Similarly, the most suitable nickel-cadmium batteries were of French and British manufacture. The solid Caesium gamma radiation source also came from Britain as the form in which it was obtainable in Canada, a nitric acid solution in a small glass bottle, seemed too fragile for the kind of field-work contemplated.

After the initial assembly the instrument required several modifications in order to improve its operating performance before attempting calibration. This is the present stage of development.

CALIBRATION

As mentioned in the previous paper it is proposed to use water and various forms of cellulose acetate as the calibration media so as to duplicate as nearly as possible the bulk density and hydrogen content ranges that it is expected to be found in peat. Three forms of cellulose acetate have been obtained: flake, pellets and powder and these will provide well-spaced calibration points for both ranges.

The calibration chamber is a thirty-five gallon capacity plastic drum with an aluminum access tube - similar to that which will be used in the field - placed centrally.

The fast neutron source has not been used for hydrogen content calibration procedures but work has been confined to tests with the density probe. As calibration is incomplete it is not possible at this time to present final data.

CONCLUSION

It would be premature to consider the suitability of the suggested method of interpreting data obtained with the instrument before calibration procedures are complete but the possibility will first be investigated of relating density measurements to the sixteen categories of peat suggested by the senior author because the various combinations of such structural features as fibrosity, granularity, non-woodiness and woodiness that are emphasized would seem to encourage such an approach.

It may be that seasonal variation in moisture content will render investigation in this direction inappropriate but the instrument may then find continuing use in *in situ* investigations of peat.

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DISCUSSION

Professor Goodman asked Mr. Ashdown if he had any data available to show comparisons in the water content using the neutron moisture meter and the standard system of drying samples in the laboratory. Mr. Ashdown said that several of Dr. Radforth's students have worked on the problem of removing samples and drying them. In obtaining samples, however, invariably some water is lost so the laboratory result may not reflect the true water content *in situ*. In any case, Mr. Ashdown reported that he does not have any comparative information. Professor Goodman expressed concern with the standard laboratory procedure for determining the water content of peat. He mentioned that in his laboratory a drying temperature of 85°C is used to prevent charring of the material and wondered if anyone was doing any research into this. Mr. MacFarlane said that he is currently engaged in a laboratory study to determine the appropriate temperature to use in water content determinations for peat.

Mr. Farmer asked if anyone had tried quick freezing as a method for obtaining samples. Mr. Ashdown explained that this approach had been tried by the McMaster Muskeg laboratory staff in the Copetown bog. A small pipe, 10 ft long, closed at the bottom end, was pushed into the ground and filled with a mixture of alcohol and dry ice. A sample was obtained about half an inch thick surrounding the lower 18 inches of the pipe. Mr. Harwood said that he has had some experience using liquid nitrogen in the probe and has obtained samples this way. Mr. Brawner commented that in B. C. they have retained 6-in. diameter samples (unfrozen) which were quick frozen and stored in that state until ready for use. Dr. Radforth thought that more attention should be given to procuring peat samples in winter. The peat can be sawed out in chunks when it is frozen. An excellent sample can be obtained in this manner and can be stored frozen for an indefinite period. He felt this method has not really been given a trial as yet. Mr. Harwood commented that he had tried this method of sampling about 10 years ago, near Kapuskasing. He pointed out it was necessary to keep the snow scraped off the surface to permit the peat to freeze to any great depth. Mr. Fox wondered what drying temperatures for water content were used in B. C. Mr. Brawner replied that they used the standard temperature of 105°C for organic soils as well as inorganic soils. He admitted that this is a controversial question and suggested that it be clearly established exactly what we mean by moisture content of peat. Dr. Radforth agreed that the matter of temperature of drying is most important as it is undesirable to char the peat. The kind of water held in the peat is also important as is the type of peat relative to the water it contains.

II. 1 CONSTRUCTION OVER MUSKEG ON THE RED DEER BYPASS

by

K. O. Anderson and R. C. G. Haas

INTRODUCTION

The reasons for the construction of highways on muskeg and the resultant problems encountered or likely to be encountered have been well discussed by authors in papers presented to previous conferences (1, 2, 4, 5). Stability and settlement during construction and throughout the service life of the highway embankment appear to be the most pressing problems that have to be faced when the method of construction without excavation or displacement of the underlying peat is decided upon. Accumulation of experience gathered by various agencies during highway embankment construction on different types of muskeg can be very useful in helping to develop methods and procedures to help solve these problems. This report will relate some of the information gathered and observations obtained during the construction of a major highway across a muskeg area west of the City of Red Deer, Alberta, during the summer of 1961.

The scope of this report will be limited to one part of a continuing project studying problems related to muskeg, or organic terrain, as it could more precisely be called, as part of the co-operative Highway Research Program sponsored by the Department of Highways of the Province of Alberta; the Research Council of Alberta, Highways Division; and the Department of Civil Engineering of the University of Alberta.

PROJECT DESCRIPTION

The muskeg section is located on a terrace on the north side of the Red Deer River approximately three miles west of the City of Red Deer. This terrace was at one time a part of the river channel, however the present channel located approximately 2, 000 ft south of the muskeg, has degraded from 10 to 20 ft. Because of the particular bridge location crossing the river the route of the four lane divided highway bypassing the City of Red Deer, of necessity, had to cross this particular muskeg.

Referring to Fig. 1 it can be seen that approximately 2, 300 ft of this alignment crosses the muskeg of varying depth up to a maximum of 11 ft. The peat is underlain by approximately 4 ft of soft silty blue clay, before encountering a firm layer of silty sand and clay.

The surface cover according to the Radforth Classification System could be described as DFI at the deeper sections with a transitional boundary to AFI at the edges of the muskeg.

In preparing the grading contract for this project, the Department of Highways made provisions for the fill to be placed in successive stages and to place a surcharge in order to preconsolidate the underlying peat before proceeding with the base course contract planned for the early summer of 1962. Since this would afford an excellent opportunity to observe the field consolidation characteristics of a particular peat and to perform stability analyses based on field vane shear strength measurements, a detailed instrumentation and observation program was undertaken as part of the co-operative research program.

PRELIMINARY INVESTIGATIONS

Following the preliminary soil survey taken by the Department of Highways during the winter of 1960-61 a more detailed investigation was undertaken by Research Council personnel during May and June of 1961. The investigation consisted of taking vane shearing strengths, and the determination of corresponding moisture and ash contents, at frequent intervals along the right-of-way. A summary of the results obtained is shown on Fig. 2. As can be seen from this figure, there is considerable scattering of results. However, general trends can be detected. It can be noted that the ash contents of the peat are relatively high, generally ranging from 50 to 85 per cent. It is considered that these high ash contents are due in part to contamination by silty erosional products and partly due to calcium carbonate concentrations appearing in the form of shells. The subsurface constitution could be described as ranging from non-woody fine fibrous peat to amorphous peat having no recognizable structure. The specific gravity of an air-dried sample taken from about mid-depth was 2. 3, again a relatively high value for peat, but consistent with the high ash content of the material.

The shearing strength obtained seemed to be highest at mid-depth with a definite layer of lower strength material at the interface between the peat and the soft underlying clay. The sensitivity of the peat, (that is, the ratio of the undisturbed strength divided by the remoulded strength) was generally about 3. Stability analyses by both the sliding block and circular arc methods (3), using a shearing strength of 250 psf, gave a maximum height of fill of 15 ft assuming a factor of safety of 1. 0. Since assumptions concerning piezometric pressures greatly influence any stability analysis, it was assumed that the piezometric pressure level was coincident with the top of the fill. This was verified by later observations as being a reasonable assumption.

Since the design height of embankment at this cross-section was 11 ft, this meant that a surcharge height of only 4 ft was possible before reaching the maximum height of 15 ft. Calculation of a surcharge height based on laboratory consolidation tests and using the method of analysis as used in British Columbia, indicated that in order to get the predicted 25 year settlement in a period of three months a height of 8 to 10 ft of surcharge would have been necessary. It could be argued that this height of fill could have been increased due to the probable increase in shearing strength by consolidation. It was thought, however, that the strength of the clay layer provided a large portion of the resisting moment and since the gain in strength through consolidation of this material would be at a relatively slow rate, a height of fill greater than 15 ft was not considered justified.

INSTRUMENTATION

It was decided to install settlement platforms and piezometers on two cross-sections at the deepest parts of the muskeg. In order to establish the shape of the interface between the fill and the muskeg a decision to install 5 settlement platforms at each cross-section was made with locations as shown on Fig. 3. The settlement platforms and piezometers were combined at each installation. The settlement platform consisted of a 3- by 3-ft by $1\frac{1}{2}$-in. wooden plate with a rigidly attached 2-in. diameter nominal size pipe riser. The piezometer consisted of a porous stone Norton tube 12 in. in length. A plastic tube as a standpipe was brought up inside a $1\frac{1}{2}$-in. diameter pipe which in turn telescoped within the 2-in. pipe attached to the settlement plate, thereby allowing for differential movement between the plate and the piezometer without displacing the piezometer.

Lateral movement stakes of dimensions 2 in. by 2 in. by 6 ft were driven into the muskeg so as to extend 2 ft above the surface. These stakes were placed at 200 ft intervals at a distance of 30 to 40 ft from the toes of the embankment.

PERFORMANCE OF TEST INSTRUMENTATION

The major difficulty encountered with the settlement platforms and piezometer installations was the tilting of the complete unit during the placing of the fill. This was due primarily to the fill being placed by end dumping in one direction only, thus causing a wave to develop at the toe of the advancing fill. In addition to the tilting of the platform due to this wave, there was an actual forward displacement of the complete installation. Despite these difficulties it appears that satisfactory settlement readings were obtained and that the piezometers functioned satisfactorily. The sensitivity of the piezometers was evident from observation of the rapid rise and fall of the water level in the plastic tube with the passing of nearby construction equipment. Unfortunately most of the installations have since been destroyed by construction equipment.

OBSERVATIONS

Typical settlement and piezometric pressure curves are shown in Fig. 4. Due to the varying depth of peat it was considered advisable to express the settlement as a percentage of the depth of peat, which in this case was 10 ft. It is apparent that the piezometric pressures were up to and slightly above the height of fill and that the dissipation of pressure was very slow.

Lateral movements were observed as high as $3\frac{1}{2}$ ft but no general failure occurred.

It is felt that these large lateral movements appear to be in the surface layer rather than a complete movement to the full depth of the muskeg.

The procedure used of varying the height of surcharge from 0 to 4 feet, depending on the depth of peat, seems to have been satisfactory. Observations made this spring show that uniform settlement appears to have taken place for undulations in the road surface due to differential settlement are not evident.

CONCLUSIONS

The following general conclusions can be drawn on the basis of field observations of the fills constructed last summer and cursory observations this spring:

1. The entire project, including instrumentation design, performance and observations, and subsequent surcharging of the fill, is considered to be successful insofar as the actual information gathered and the guidance for future projects of this type is concerned.
2. It is most desirable to have settlement platforms and piezometers installed on a project if preconsolidation techniques are to be used. They are very useful in order to determine when to remove the surcharge and also to warn of impending failure.
3. It would appear that stage construction techniques are desirable features to be used along with preconsolidation techniques. This allows for the use of a lower surcharge by extending the period of time between placing of surcharge and subsequent removal.
4. For this particular type of deposit and method of embankment construction it was possible to use the vane shearing strength measurements to predict a safe height of fill. It is not possible, however, to state precisely just what the factor of safety against shear failure was, although since the fill did not fail it must have been greater than unity.

ACKNOWLEDGMENTS

The authors wish to acknowledge the assistance given by Department of Highways personnel who were on this project, particularly Resident Engineer L. N. Whitson, for his co-operation in all aspects.

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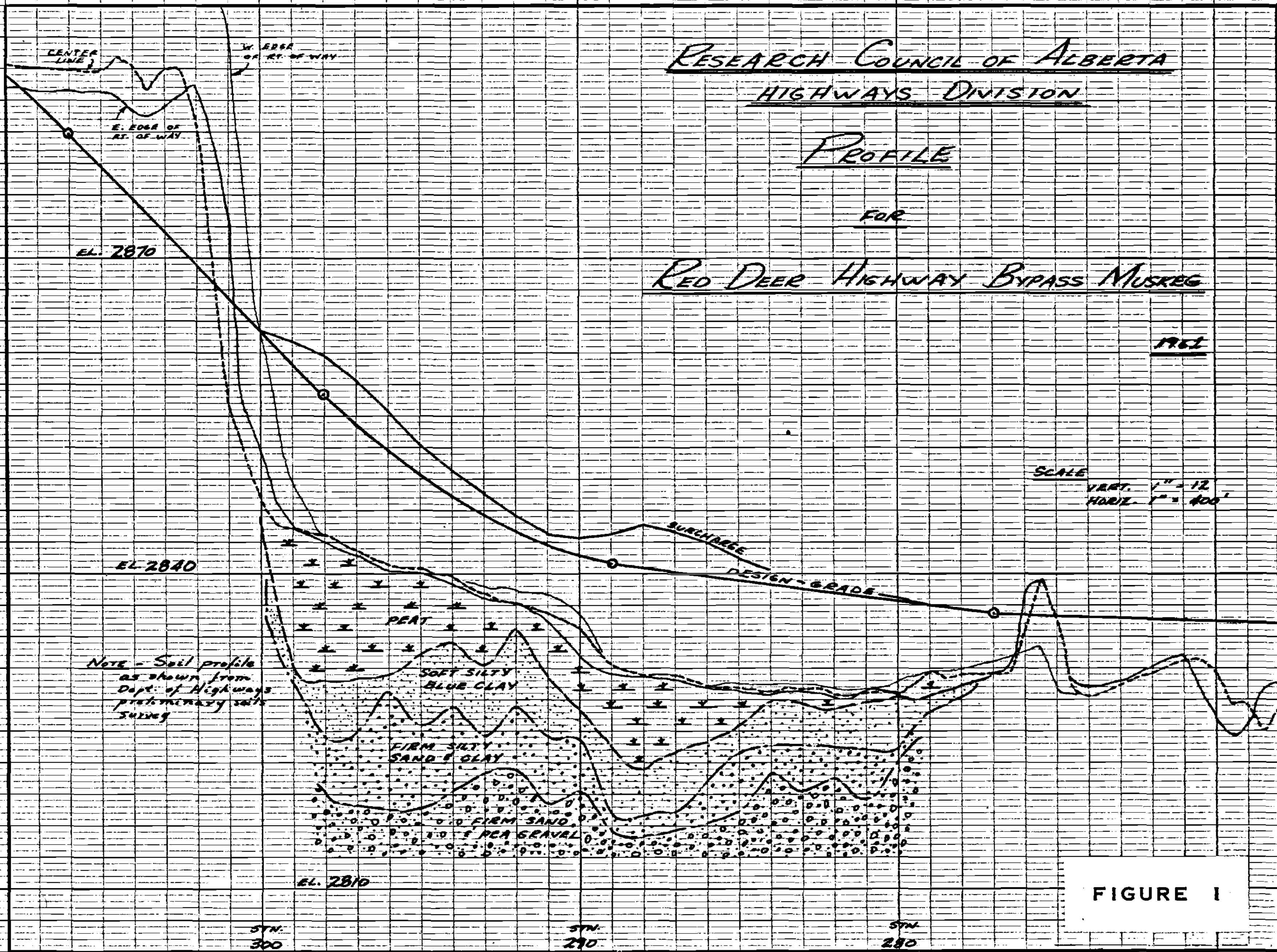
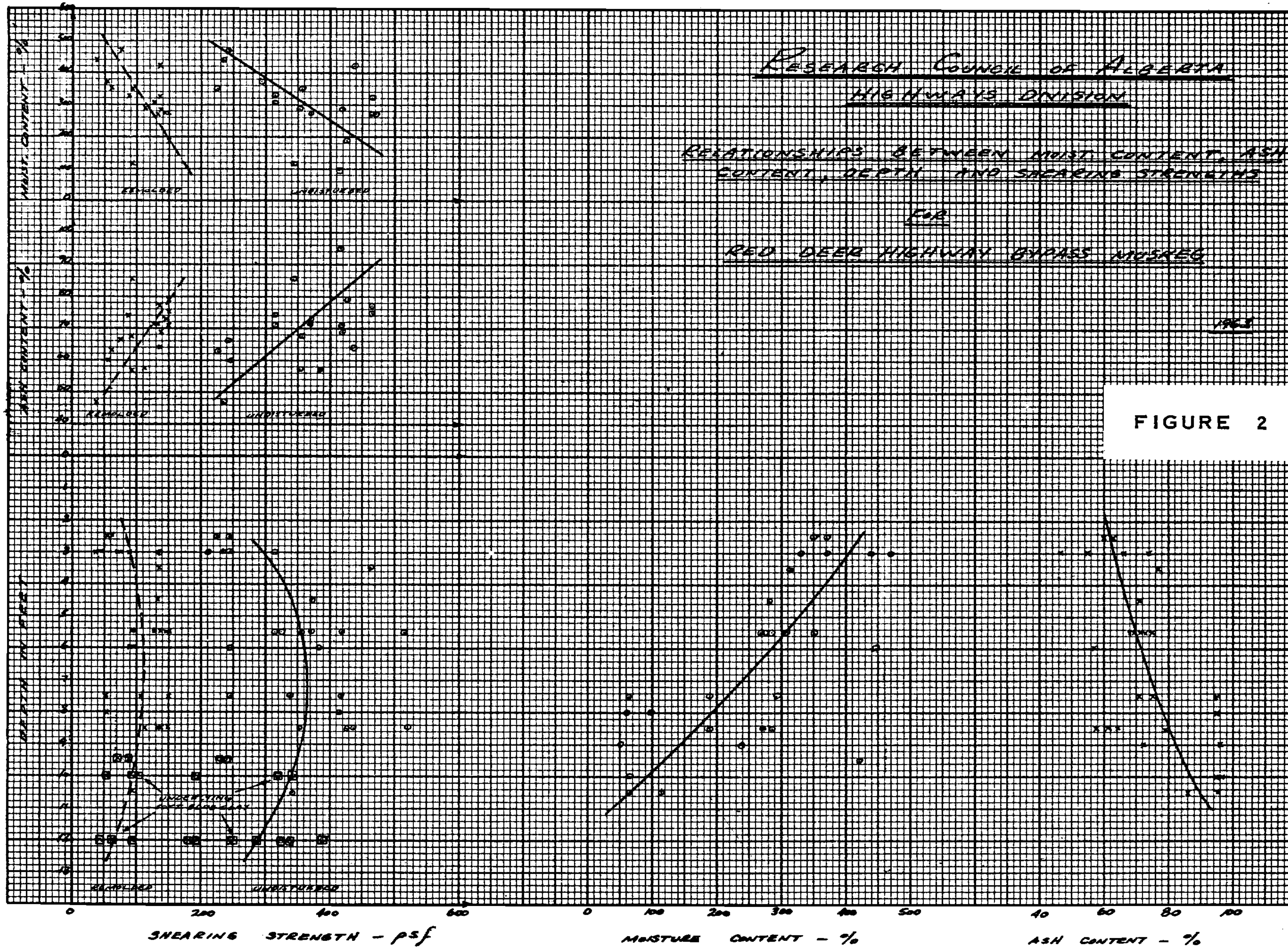

RESEARCH COUNCIL OF ALBERTA
HIGHWAYS DIVISION
PROFILE
FOR
RED DEER HIGHWAY BYPASS MUSKEG
1961
SCALE
VERT. 1" = 12'
HORIZ. 1" = 400'
CENTER LINE
W. EDGE OF RT. OF WAY
E. EDGE OF RT. OF WAY
EL. 2870
EL. 2840
EL. 2810
SURCHARGE
DESIGN-GRADE
PEAT
SOFT SILTY BLUE CLAY
FIRM SILTY SAND & CLAY
FIRM SAND & PEA GRAVEL
NOTE - Soil profile as shown from Dept. of Highways preliminary soils survey
STN. 300
STN. 290
STN. 280

FIGURE I


RESEARCH COUNCIL OF ALBERTA
HIGHWAYS DIVISION
RELATIONSHIPS BETWEEN MOIST. CONTENT, ASH CONTENT, DEPTH AND SHEARING STRENGTHS
FOR
RED DEER HIGHWAY BYPASS MUSKEG
1963
FIGURE 2
MOIST. CONTENT - %
ASH CONTENT - %
DEPTH IN FEET
REMOLDED
UNDISTURBED
UNDERLYING SOFT BLUE CLAY
0
200
400
600
SHEARING STRENGTH - psf
0
100
200
300
400
500
MOISTURE CONTENT - %
40
60
80
100
ASH CONTENT - %

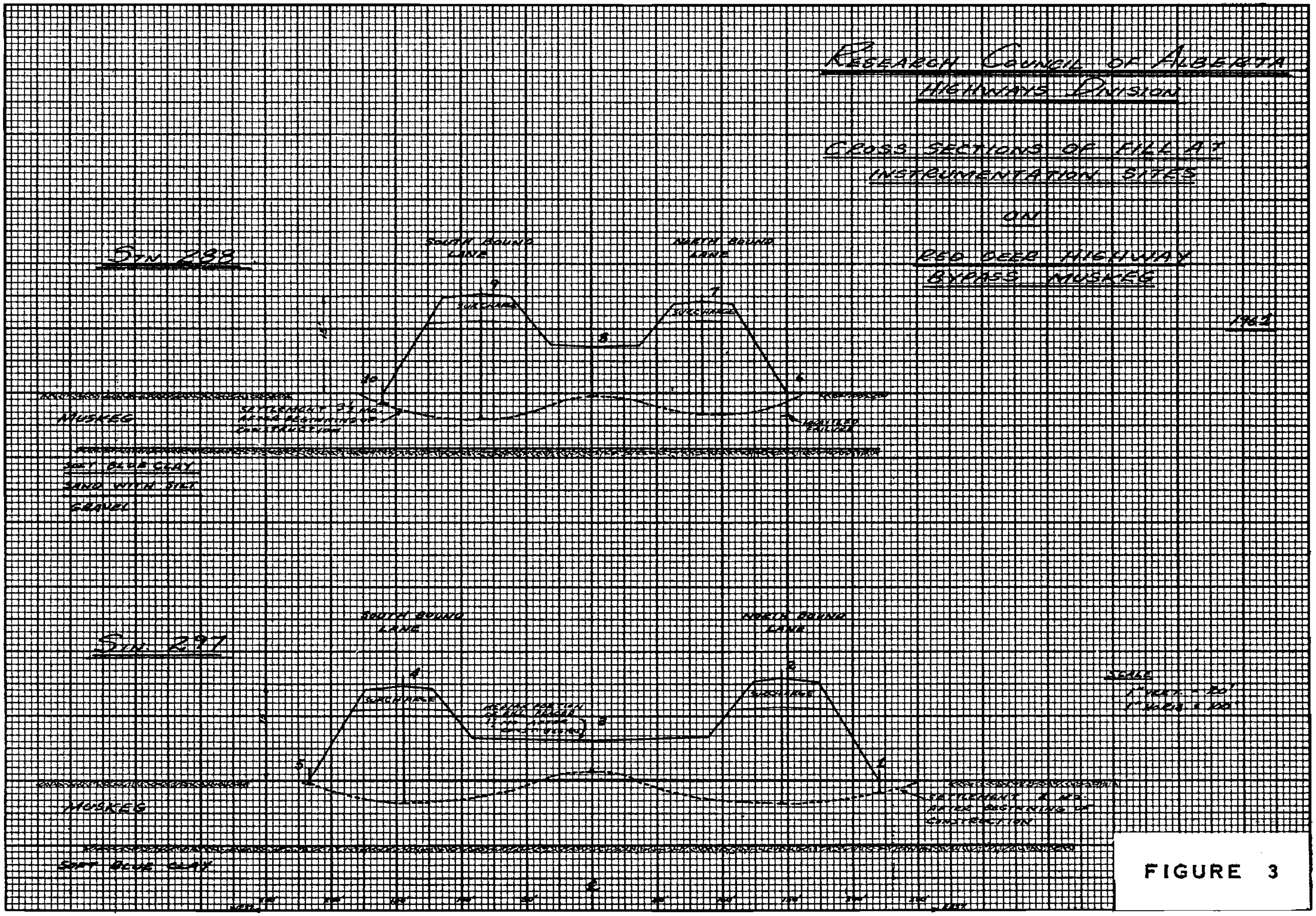
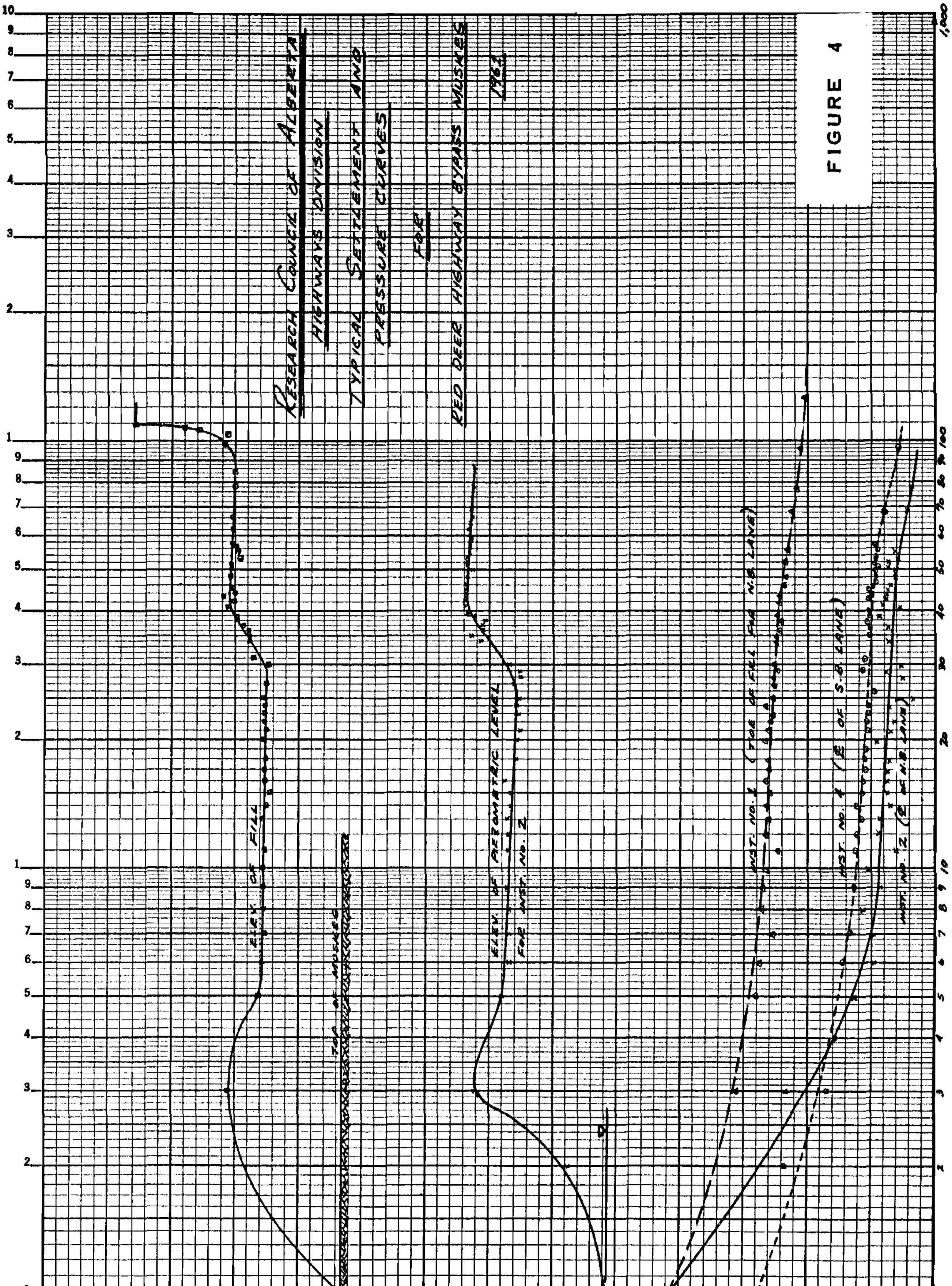

RESEARCH COUNCIL OF ALBERTA
HIGHWAYS DIVISION
CROSS SECTIONS OF FILL AT
INSTRUMENTATION SITES
ON
RED DEER HIGHWAY
BYPASS MUSKEG
1962
STN. 288
SOUTH BOUND
LANE
NORTH BOUND
LANE
SURCHARGE
SURCHARGE
9
7
8
10
6
MUSKEG
SOFT BLUE CLAY
SAND WITH SILT
GRAVEL
STN. 297
SOUTH BOUND
LANE
NORTH BOUND
LANE
SURCHARGE
SURCHARGE
4
2
3
5
1
MUSKEG
SOFT BLUE CLAY
SETTLEMENT
AFTER BEGINNING OF
CONSTRUCTION
SCALE
℄

FIGURE 3


RESEARCH COUNCIL OF ALBERTA
HIGHWAYS DIVISION
TYPICAL SETTLEMENT AND PRESSURE CURVES
FOR
RED DEER HIGHWAY BYPASS MUSKEG
1961
FIGURE 4
ELEV. OF FILL
TOP OF MUSKEG
ELEV. OF PIEZOMETRIC LEVEL FOR INST. NO. 2
INST. NO. 1 (TOE OF FILL FOR N.B. LANE)
INST. NO. 4 (℄ OF S.B. LANE)
INST. NO. 2 (℄ OF N.B. LANE)
ELEV. FT.
ELEV., FT.
SETTLEMENT AS PERCENT OF MUSKEG DEPTH
TIME ELAPSED IN DAYS

DISCUSSION

Mr. Bergan wondered how Professor Anderson would explain $3\frac{1}{2}$ ft of lateral movement without shear. Professor Anderson compared it to putting a cork in a bowl of jelly. Plastic deformation takes place quite rapidly. The peat - like jelly - was plastic enough that it did not shear.

Mr. Fox asked if a maximum rate of movement had been established at which point construction would be stopped. Professor Anderson replied that if such a cut-off point had been established, in all likelihood construction would have been halted sooner than it actually was. He said, however, that the material itself has to be taken into account. In a relatively dry peat, for instance, $3\frac{1}{2}$ ft of lateral movement could very likely not be permitted. Mr. Evans remarked that in a good number of bogs the microtopography varies widely. As a result of the "hills and valleys" in a bog, cracking would occur at the surface of the fill at the "hill" and at the bottom of the fill in a "valley." He wondered if these cracks occur in the pavement surface. Professor Anderson explained that the normal practice in Alberta is to allow time between placing of the fill and the stabilized base course (a year later) and of the wearing surface (a year after the base course) such that most of the irregularities have been ironed out. Mr. Evans suggested that a plastic core in the fill would eliminate transverse cracking. Professor Anderson wondered how this would be done. He pointed out that in stability analyses, no shear strength was assumed for the fill due to the likelihood of tension cracks arising from extreme differential settlements.

Mr. Schlosser inquired about surface treatment before the fill was laid. Professor Anderson said that trees were hand cut and burned. The trees, brush, etc. were not used for matting. Mr. Monaghan asked if any cross-sections of the embankment were taken after completion. Mr. Haas replied that a centre-line profile was taken last fall and again this spring, but time had not yet permitted a close examination of these. Mr. Monaghan also inquired if the toe of the slope was designed to terminate on the muskeg surface or on the base of the organic material. Mr. Haas said that it was designed to terminate at the base of the organic deposit. Mr. Monaghan asked if there was any record of compression of the peat and how much total consolidation is expected. Professor Anderson said that it is difficult to predict how much the ultimate settlement will be. However, up to the present time, the original 11-ft depth of peat has compressed by 3 or 4 ft.

Dr. Radforth asked what sort of drainage techniques were used. Professor Anderson replied that the muskeg was on a slight slope and no lateral or transverse drains were put in. Mr. Mickleborough wondered if the consolidation tests were one-dimensional or triaxial. Professor Anderson stated that one-dimensional consolidation tests were carried out. Samples were obtained by a 2-7/8 inch I. D. stationary piston sampler and transferred to the consolidation ring, which had a height greater than one inch. Mr. Fox inquired about installation of culverts. Mr. Haas explained that in some cases the peat was excavated down to clay, the trench backfilled with gravel, and the culvert placed on the gravel. In the case of the Red Deer muskeg, the culvert was located at a point on the edge of the muskeg where the mineral soil comes up to the surface.

II. 2 THE MUSKEG FACTOR IN THE LOCATION AND CONSTRUCTION OF AN ONTARIO HYDRO SERVICE ROAD IN THE MOOSE RIVER BASIN

by

C. T. Enright

INTRODUCTION

For several years the Commission's power requirements from water resources in comparatively large consignments have pushed the frontiers of Ontario farther and farther north. This is particularly true since the large blocks of hydraulic power available in Southern Ontario have been developed. "Farther north" in most cases has meant, and will mean, encroaching more and more into those areas of the Canadian shield where muskeg becomes more prevalent. One of these areas is known as the Moose River Basin - a huge area of Northern Ontario comprising some 35,000 square miles drained by the Moose and its tributaries, the Abitibi, Mattagami and Missinaibi emptying in James Bay at Moosonee.

Heretofore many of the concentrations on these rivers have been considered too costly to develop until load appears nearby. Transmission of power at 460 kv (extra high voltages or EHV as it is commonly called) has altered this. Transmitting power at higher voltages has reduced transmission and transformation costs to the extent that energy from many of these far northern sites delivered to southern load centres is now within economic limits.

The initial requirement for the construction of a power development is access. This may be by existing rail or road facilities. Extension of one or the other or both of these is sometimes all that is necessary. In other cases, extensive road building is required. Modern construction practice requires that access be of a high standard. In addition, many of the plants are remote controlled, and therefore non-attended after construction. The construction access set-up may not be suited to adequately fill the need for periodic inspection and emergency access for breakdown and repair. An operational service road may then be desirable. This type of road need not be of as high standard as that required for construction since it will not be subject to the same continuous heavy traffic. It may best be described as a road capable of supporting moderate to heavy movement of light traffic with the occasional transport of heavily loaded vehicles. The completed road must afford all-season travel with a minimum of maintenance. Driving width may be single-lane with drive-outs at appropriate intervals - about every half mile. These roads may be the only direct access other than by air. Drive-outs may serve the dual purpose of passing traffic and as emergency landing bases for helicopters.

A service road of this type will link the Mattagami River plants at Little Long

Generating Station with Pinard Transformer Station near the Abitibi River which is designed as the control centre for all plants in the area and as the gathering point for EHV transmission to the south. The Little Long Generating Station is located on the Mattagami River 45 miles north of Kapuskasing, while the Pinard Transformer Station will be located about one-half mile east of the Ontario Northland Railway at Fraserdale, the latter being about 70 miles north of Cochrane.

LOCATION

The final road location is approximately 27 miles in length and is generally through wooded country with soils comprising silts, silty sand and clay, and sand and clay. The route is generally on the height of land between the north and south flowing streams. (Refer to Key Plan, Plate 1.) No rock in place is evident on the surface. Boulders are found on the surface or mixed in with the various soils. Of particular interest is the fact that approximately 5 miles of the route is over organic terrain. Included in this are some 4,000 feet in which depths of muskeg vary from 3 to 14 feet.

Location methods followed standard highway practice with the addition of muskeg interpretation from air photos by the Radforth method. Initially a route was chosen on 8 miles to the inch topographic maps after an aerial reconnaissance by helicopter had yielded the general characteristics of the area. Basically, this route was the most direct to be found taking into consideration the fundamental obstacles to be avoided in road location. Air photo cover sufficiently wide to take care of alternative routes was then obtained and a photogrammetric location was projected thereon. Muskeg typing was also derived from the photos. Dr. Radforth was consulted and confirmed the location, subject to a field check in which he participated. In late spring the projection, with some revision during the field check, was blazed on the ground. Subsequently a survey line was cut and the survey and plotting carried out. The field check generally substantiated the assumptions made from the photo typing, and any revisions were the normal refinements usually made in this type of location. During the survey, soil and vegetation were typed and muskeg probing for depth and character was carried out.

DESIGN

The road is designed with a one-lane surface and granular base 16-feet wide between shoulders at subgrade or profile grade with drive-outs one-half mile apart. The subgrade is to be constructed to provide for future widening to two lanes should conditions warrant.

In muskeg sections where corduroy is specified, i.e., muskeg generally over three feet in depth, the road will be 24 feet wide between shoulders, with appropriate tapering to the 16-foot width. In muskeg sections under three feet in depth, the muskeg will be removed to mineral soil and the excavated areas backfilled with suitable embankment material. The excavated muskeg is to be used to flatten embankment slopes.

Clearing will be 100 feet in width throughout except where excavation or embankment slopes and side borrow require extra width. Grubbing will be carried out on all areas, except where corduroy is required, to a width sufficient to accommodate the road section and the ditches. In muskeg sections where corduroy is specified, close cutting will be carried out in lieu of clearing and grubbing and the natural mat must not be disturbed. In these sections every care must be taken not to fracture this mat and no vehicles will be permitted inside the 100-foot right-of-way until the corduroy is laid. Traffic will then be permitted on the corduroy and subsequent fill. Embankments generally throughout the muskeg will be of the order of three feet in height above the original ground level.

The fill up to the bottom of granular base may be any suitable road fill free from organic material. This will be compacted from the bottom up by the six-inch layer compaction method except that the initial fill over corduroy may be 12 in. in depth. The succeeding layers shall adhere to the six-inch layer method.

Overlying the common fill a 12-in. compacted depth of selected granular base course "B" is specified to the elevation of the grade line shown on the profile. Pipe culverts are specified throughout, either plain galvanized or asphalt coated depending on the acidity factor of the drainage.

The reasons behind this design are particularly pertinent to road construction over organic terrain. No particular difficulty is foreseen in building on or through the mineral soils. However, considerable thought was given to organic terrain areas which comprised about twenty per cent of the route.

For a distance of some four miles, the eastern end of the route is almost continuously in shallow muskeg varying from one to three feet in depth. This whole area is predominantly ADE and AEI with occasional patches of BEI and DFI. Plate 4 shows typical ADE terrain. Plates 5 and 6 show AEI terrain from the air and on the ground. Plate 7 shows AEI and BEI terrain. It should be noted how one blends into the other. The majority of the areas of BEI and DFI were avoided by positioning of the line during location. The areas of ADE cover could support a fill directly on their surface with allowance being made for settlement. Unfortunately, the areas are covered with considerable down timber and without grubbing a properly compacted road bed would have been next to impossible to achieve. Also, elevation differential is slight and with a high water table it would be very difficult to obtain borrow pits of workable dimensions. However, with reasonable weather conditions and proper drainage, considerable shallow side borrow could be obtained after removal of the muskeg. Though this material is of a silty nature and of high moisture content, when properly handled it will provide suitable road fill. Replacement of excavated muskeg on the embankment slopes will deter, and possibly prevent, washing and gullying. Corduroy throughout, in lieu of excavation, would have provided a suitable foundation for the embankment. However, from present knowledge the point of economic balance between the cost of grubbing and excavation and subsequent backfilling, and the cost of the supply and placement of corduroy falls at about a depth of three feet of muskeg. The drainage gradient, though slight, is adequate and orthodox lateral ditching with suitable offtakes is specified.

Near the westerly end of the route, an entirely different condition was faced. Here there are EI, DFI and EFI, and BDE and BDF intermittently over a distance of about 4000 ft. In the EI, DFI and EFI cover the muskeg has a depth of up to 14 ft. Plate 10 shows the critical line of relocation. EI and FIE cover adjoin and it is significant that the difference in cover can be readily seen from aerial photographs. Functionally the root and fibre structure of the woody E cover gives a much greater degree of support than the non-woody grassy F cover. In the BDE and BDF cover, muskeg varies from one to eight or ten feet in depth. The structural constitution of the peat foundation is woody fibrous elements in close continuous mesh for a depth of over four feet. It is reasoned that this natural mat will adequately support an embankment of the designed type on corduroy and afford a stable roadway capable of carrying gross loads up to 60 tons. Some settlement is expected but with the design height of the embankment, it is not considered to be critical. Plates 11 and 12 show the standards governing corduroy. Longitudinal stringers are to be spiked to each log with half-inch drift pins. All logs must have a minimum tip diameter of six inches. Joints in the stringers and courses are to be staggered. The corduroy specified is designed to produce a dense compact mat which will remain as placed indefinitely. It will be completely covered by the embankment material so that the air is excluded and so should remain sound. As an added precaution, coniferous wood only is specified for its construction. Branches and smaller trees will be placed on top of the logs to prevent the embankment material from passing through to the muskeg and creating voids and developing pot holes. The embankment might be constructed without the use of corduroy but maintenance would be costly until final settlement was achieved. Settlement and deformation of section on corduroy will be minimal.

No attempt will be made to drain the deep muskeg areas, since drying to any extent would weaken its structure. Drying affords aeration which in turn rapidly breaks down the woody fibre - the desirable constituent of this particular peat type. The only way to preserve this fibre and the strength of the mass of peat is to keep it wet - in its natural condition. However, provision is made to pass water through the road by culverts at critical locations. These culverts will be placed at either end of each corduroyed section on mineral soil or on suitably constructed foundations, so that settlement will be kept to a minimum. This will alleviate ponding on the upstream side of the embankment during high water periods. It is expected that the existing subsurface drainage will not be critically disturbed.

ARRANGEMENT OF CONTRACT

The work in hand covered by the present contract consists of general grading which is let on the unit-mile basis and includes clearing, grubbing, close cutting, excavation and embankment and ditching. Other items are the supply and placement of corduroy, the supply and placement of culverts and the supply and placement of the 12-in. depth of selected granular base course. Though it is not expected that rock will be encountered, a nominal quantity has been inserted in the price schedule in order to obtain a unit price. For the purpose of comparison of costs, in modern construction practice, alternative tenders were required for 16- and 24-ft roads throughout. This provided a comparison of the economics of single vs two-lane construction in northern bush roads. It was found that the composite standard which has been adopted provided the best economy having in mind ultimate widening some time in the future. The differential between the cost of a subgrade to accommodate two-lane traffic and that to accommodate one-lane traffic was less than 4 per cent of the total cost of the road.

After the present contract is completed it is proposed to place a 4-in. layer of compacted crushed rock of minus $2\frac{1}{2}$-in. crusher run grading. This contract will be let separately, or completed by the Commission's personnel and the material will be supplied by the Commission. It is expected that a great part of it will be crushed from tailrace excavation at the Little Long Project. This crushed surface will be filled by the addition of crusher fines to create a dense compacted surface.

INSTRUMENTATION

Instrumentation will be carried out in the deepest muskeg as a joint effort between the Research and Engineering Divisions. A single line of piezometers for measuring pore pressure will be installed. Also, steel base plates will be pinned to the top of the corduroy for settlement measurement. This is shown on Plate 14. Periodic measurements during and after construction will be taken. The points will be permanently referenced so that results may be obtained over a long period of time.

Chainage 198+00 represents a typical section of average fill of three feet over muskeg. This is considered to be a desirable height for embankments over fairly deep muskeg. It will not overload the peat structure but at the same time is adequate to carry the traffic load. It is high enough to permit proper snow removal.

Chainage 208+00 represents a $6\frac{1}{2}$-ft fill over about 6 ft of muskeg. This is the highest embankment over muskeg on the road.

ABITIBI EXTENSION

Adjoining the eastern end of this route a three-mile section has been completed from Pinard Transformer Station to the Abitibi Generating Station. This road will serve as access from the Abitibi colony, ultimately eighty houses, to the Pinard Transformer Station and to the Ontario Northland Railway at Fraserdale. In the interests of economy it was decided to construct this section adjacent to the grade of the existing railroad spur which was required to be kept in service. Most of this route was in muskeg having AEI and BEI cover with short sections of DFI and FI cover, the latter, of course, denoting the deeper muskeg. Clearing only was carried out on this section. A rock spoil pile was used for common fill and was dumped adjacent to the existing grade to form the base of a two-lane road. This road will be completed with the addition of granular base and surfacing to the same standard as the Little Long section.

No particular difficulty was experienced on this section because of the nature of the fill material. The source was a rock dump which was the spoil from the excavation during the construction of the Abitibi Canyon station during the 1920's. This was composed of broken rock and overburden varying in size up to a cubic yard or more which had been washed and weathered during the years since placement. Because of its unit weight it settled to the bottom of the muskeg and there is little evidence of lateral displacement. Some trouble was experienced because of muskeg ooze to the surface through the voids between the larger fragments but this is not serious and can be taken care of by blading over the side slopes.

The major factors in determining the design of this section were the material available from spoil and that little natural borrow is obtainable within reasonable haul limits. Corduroy was not used, although the muskeg generally would have supported it, because experience has shown that rock fill on corduroy is far from satisfactory. Due to unequal loading, uneven settlement of the corduroy results, followed by distortion of the surface of the cross section. Tipping of the corduroy may occur with the resultant sliding of the embankment.

GENERAL CONCLUSIONS

Because of the varying conditions and their treatment it is intended that the entire road will serve as basic reference in future road-over-muskeg development for the Commission.

Various features relative to the location and design have been established already:

1. Photogrammetric interpretation using the Radforth method of classifying muskeg has proven itself. Field check has substantiated this. Much time can be saved, particularly in the road location involving considerable distances by using small scale photography. This has the advantage of adding breadth to the area to be investigated, thus giving greater scope.

2. The use of a recognized code which is widely understood and in which the

elements have an established significance promotes the exchange of ideas between the various interests involved.

3. The significance attached to the coding letters and the combinations thereof has a particular value in the determination of muskeg suitability for various uses. This of course is related to the sphere of reference.

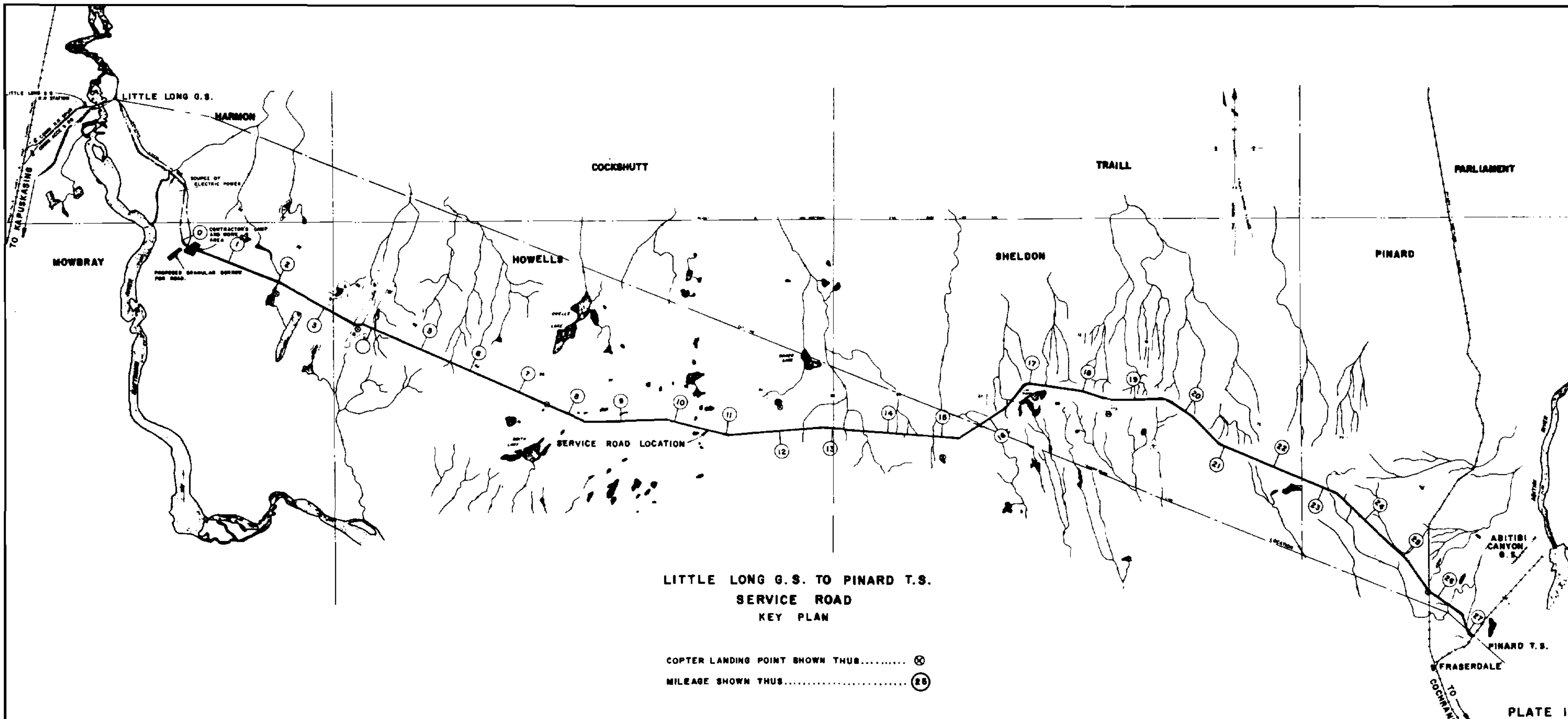
Specifically the reference to DFI and EFI cover signifies that although this is muskeg capable of supporting a road on corduroy with the organic material left in its natural state, it would be ruinous to fracture this natural mat; hence the prohibition of traffic within a reasonable distance before the corduroy is laid, in this case within the 100-ft right-of-way.

Similarly it may be assumed that the ADE cover denotes muskeg that is usually shallow in depth which in its undisturbed form would accept an embankment of approximately three feet in height. This would be economically sound if borrow were within reasonable haul limits.

4. Careful documentation of this and other road construction will help to build a storehouse of knowledge about this particular terrain feature which in the future development of this province, and indeed the country as a whole, will either act as a barrier or prove to be one of nature's gifts once we have discovered its secrets.

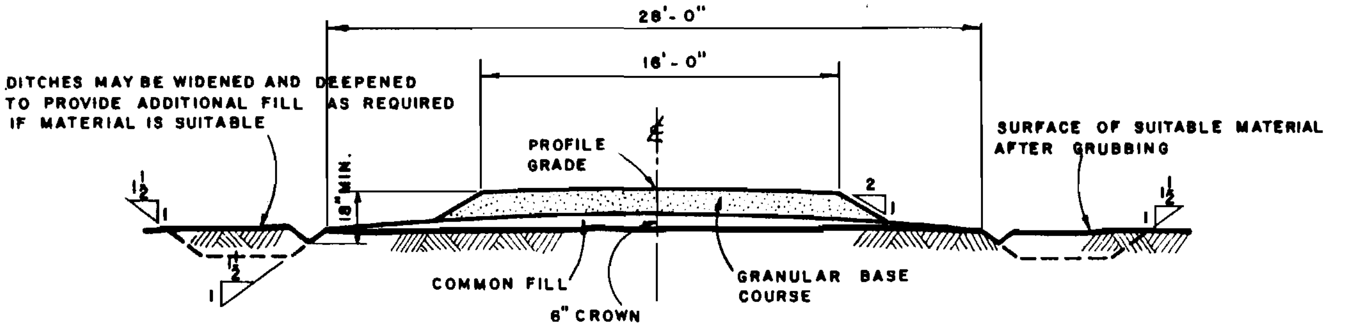
ACKNOWLEDGMENTS

The author would like to take this opportunity to thank Dr. Radforth for his help and understanding, not only during the location of the road, but also during the preparation of this paper.



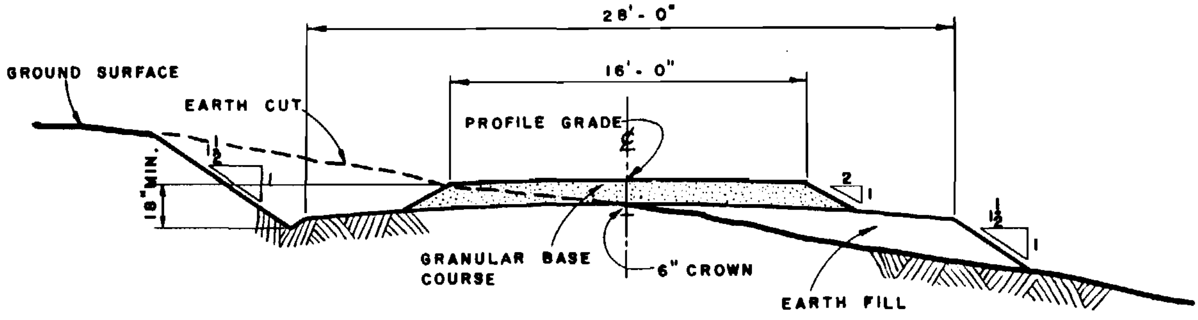
LITTLE LONG G.S. TO PINARD T.S.
SERVICE ROAD
KEY PLAN

PLATE I



GRADERWORK SECTION

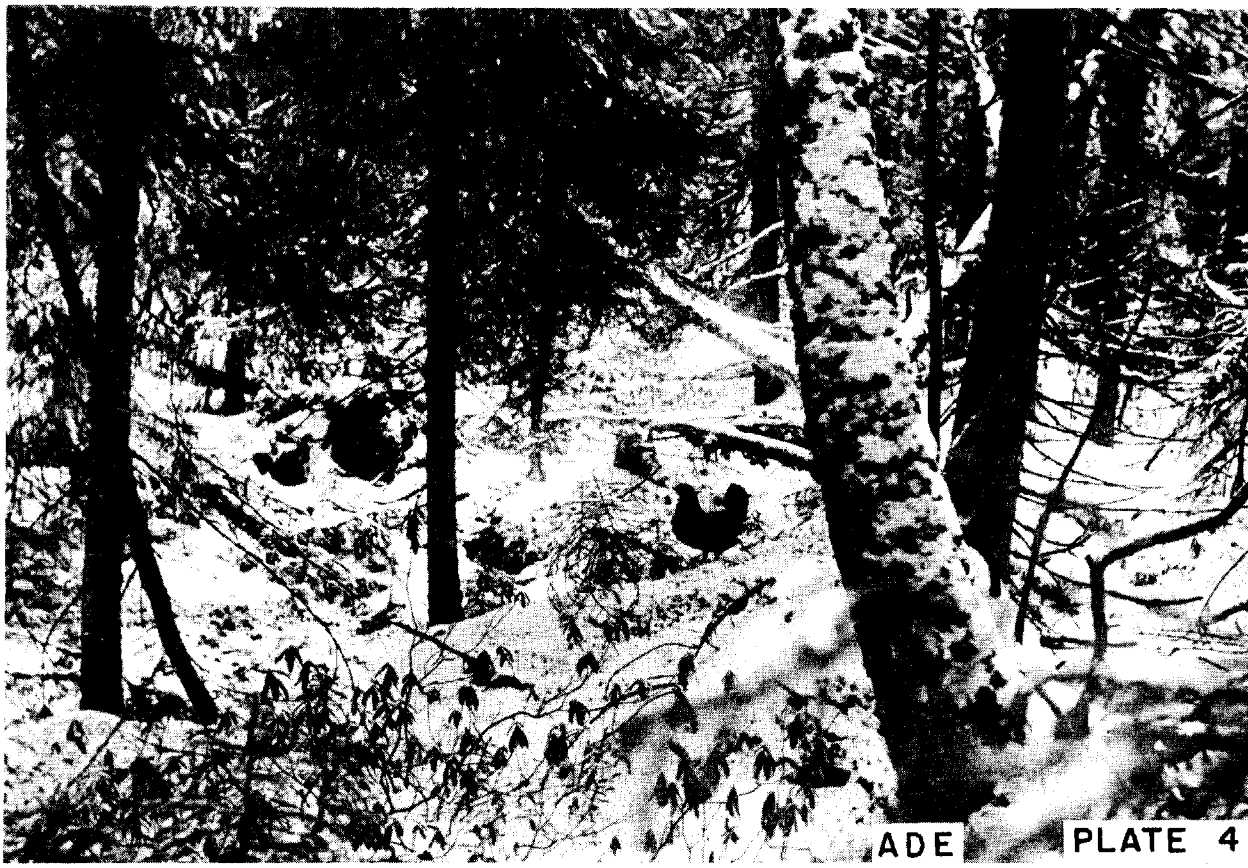
PLATE 3



NOTE: FILL MATERIAL MUST BE APPROVED BY THE ENGINEER

EARTH CUT & FILL

PLATE 2


ADE
PLATE 4


AEI
PLATE 5

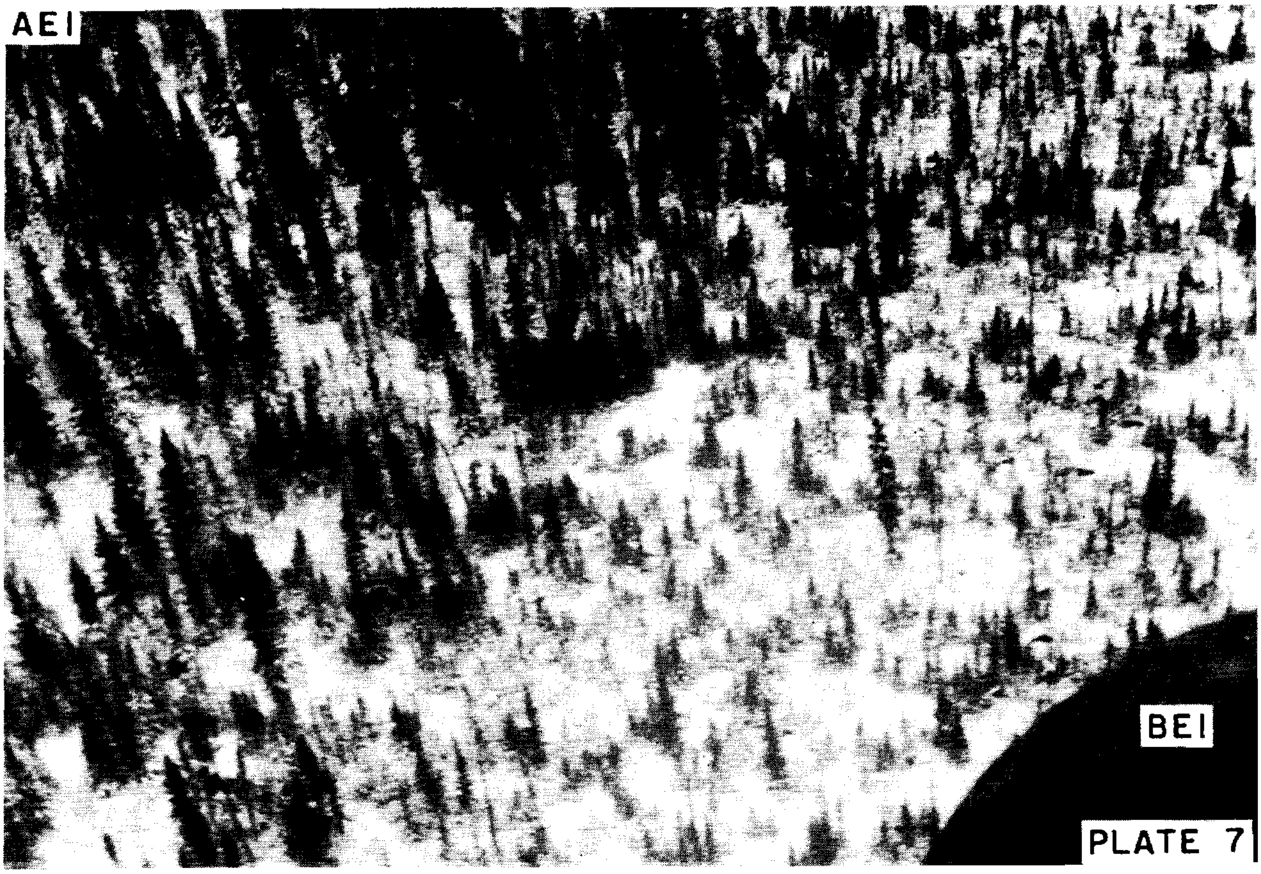


PLATE 7

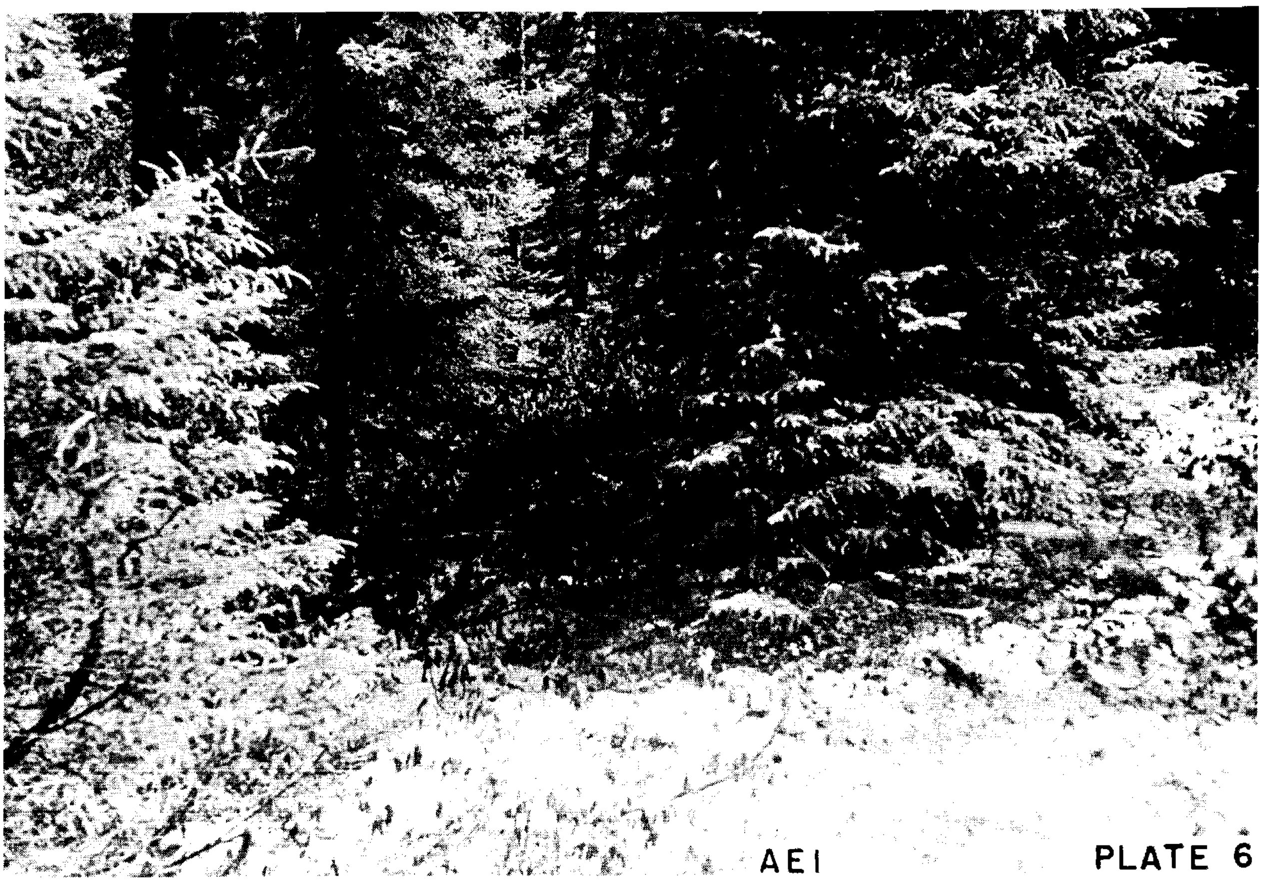
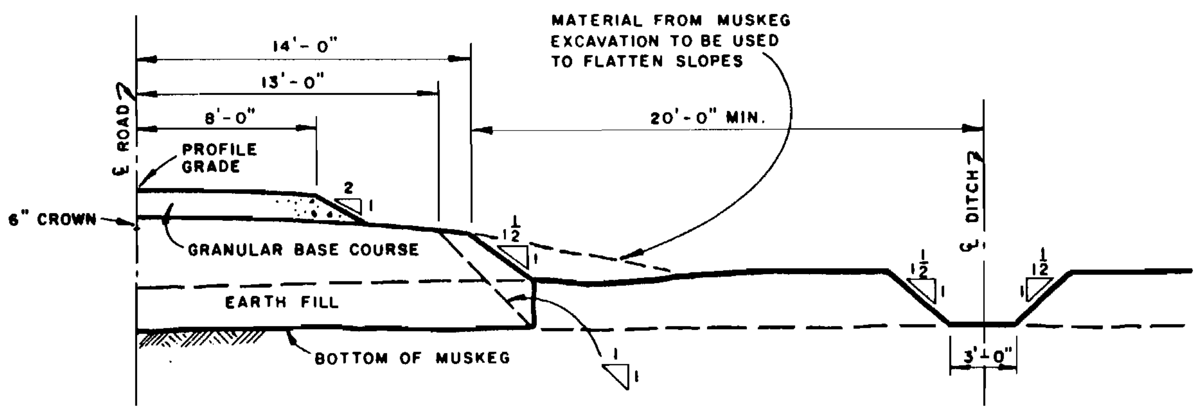


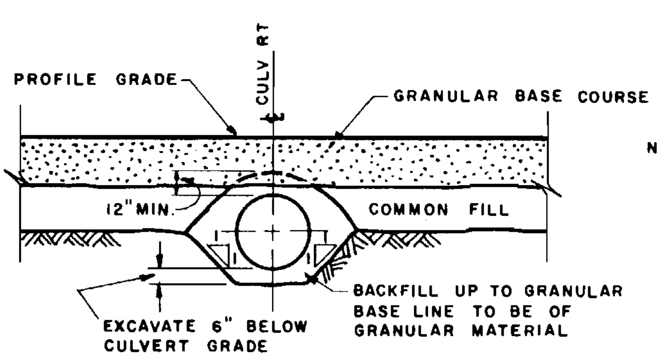
PLATE 6



MUSKEG EXCAVATION

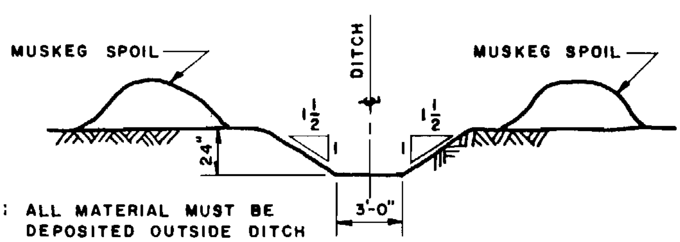
HALF SECTION

PLATE 8



NOTE: TOP OF CULVERT TO BE AT LEAST 12" BELOW TOP OF GRANULAR BASE COURSE

SECTION THROUGH CULVERT

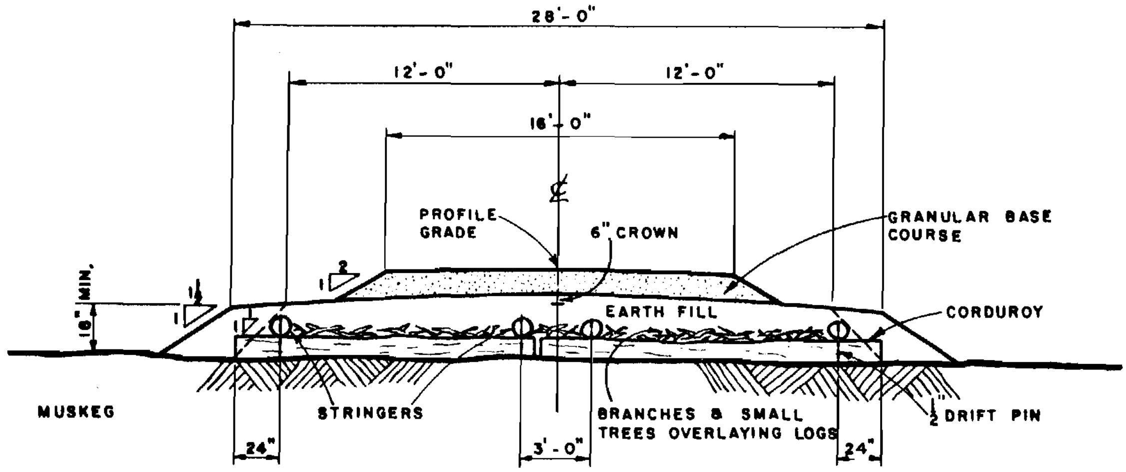


NOTE: ALL MATERIAL MUST BE DEPOSITED OUTSIDE DITCH AND LEVELLED

OFF TAKE DITCH

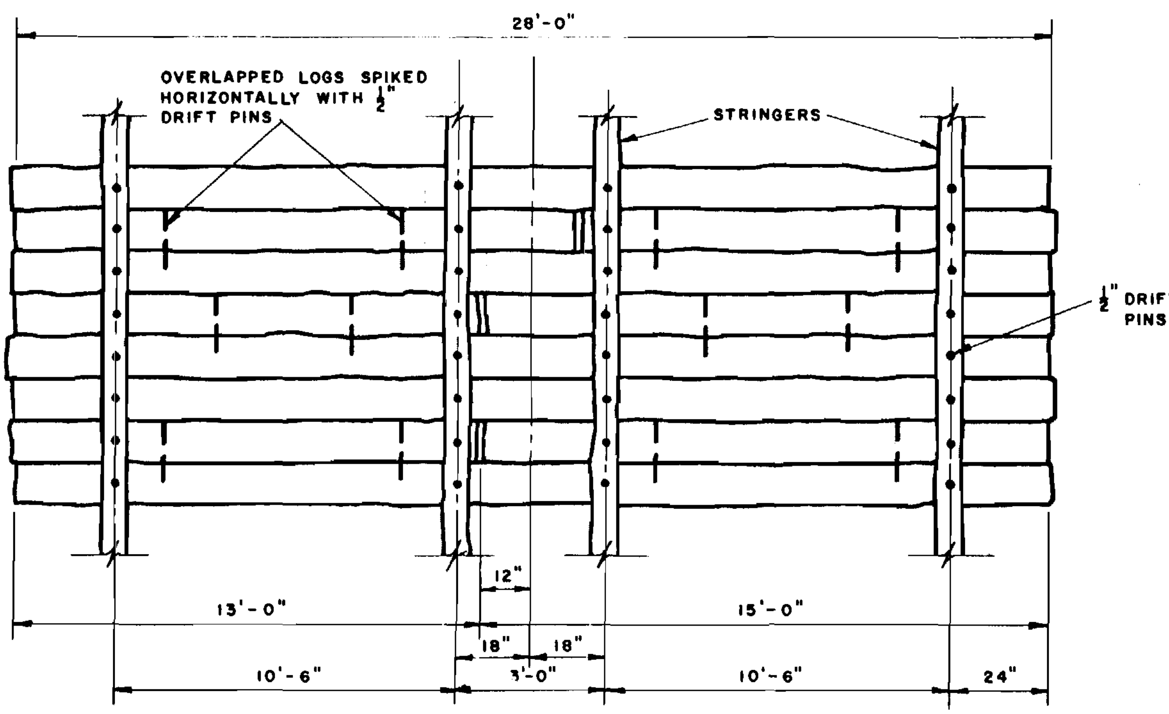
PLATE 9

CRITICAL LINE IN RELOCATION SHOWN



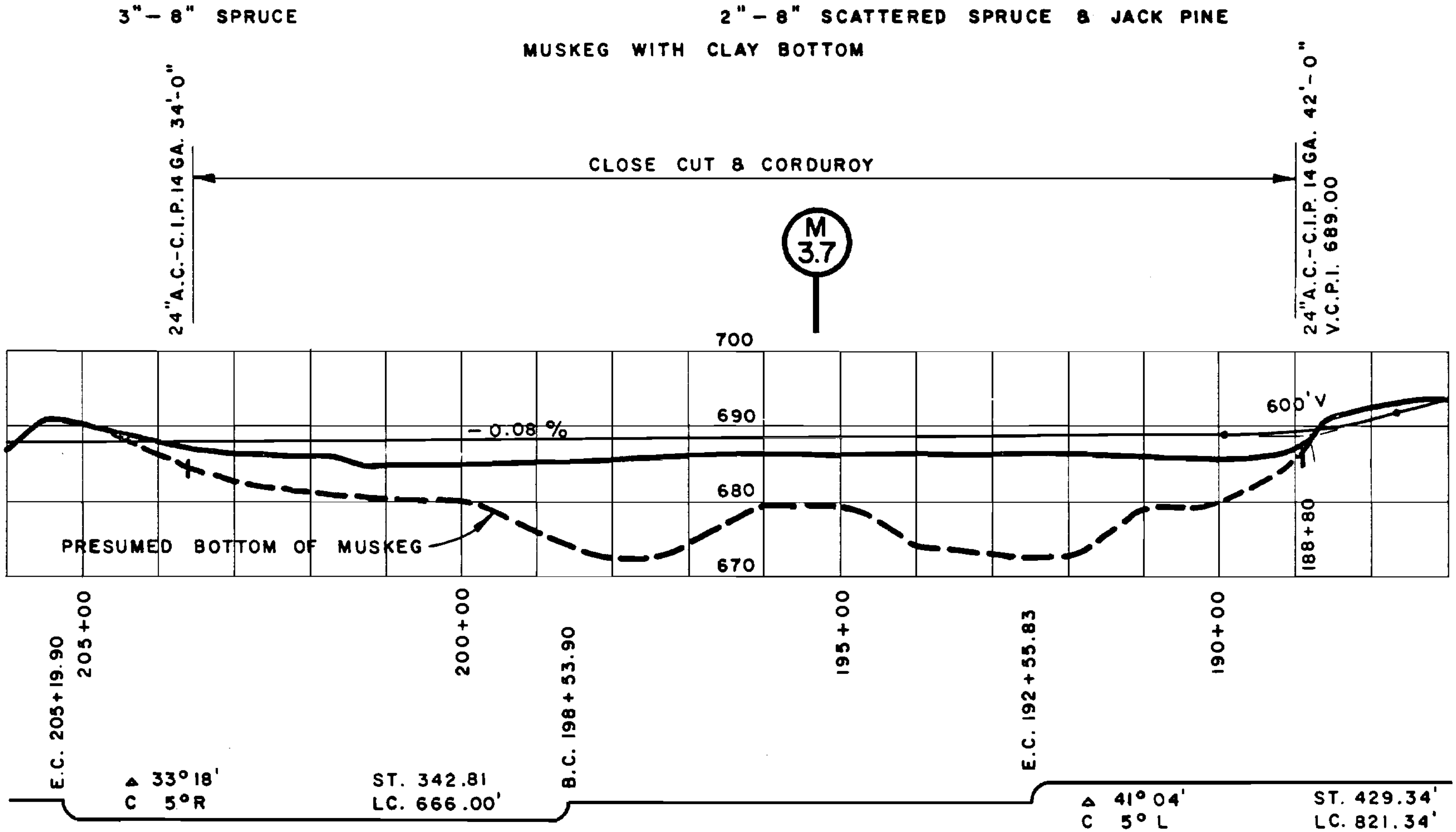
EARTH FILL ON CORDUROY

PLATE 11



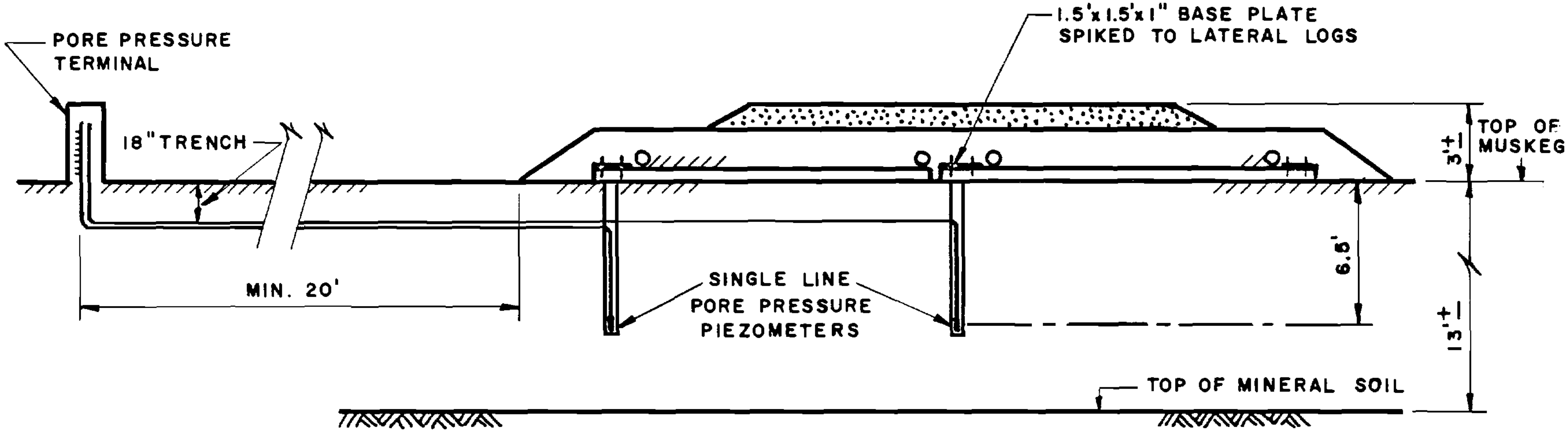
PLAN OF CORDUROY

PLATE 12

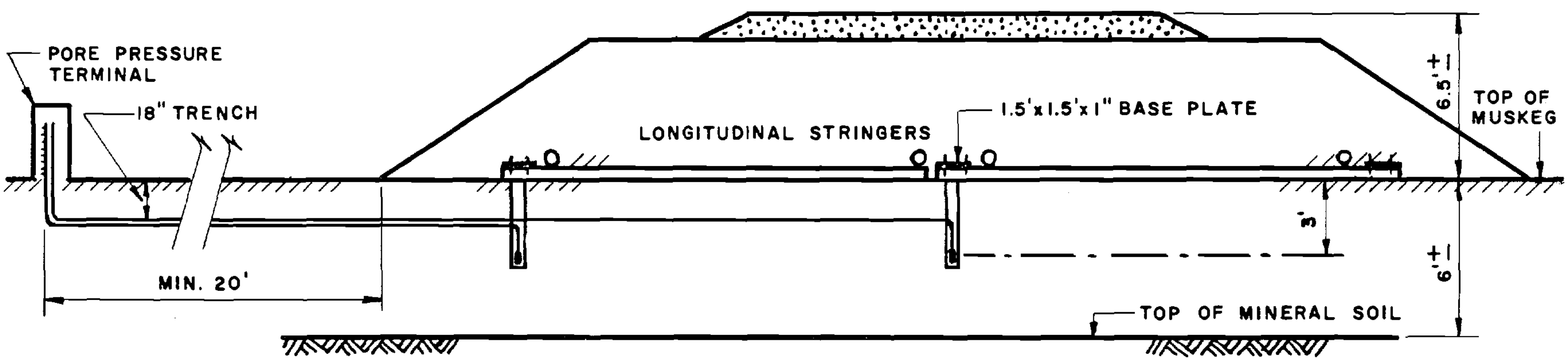


DEEP MUSKEG SECTION PROFILE

PLATE 13



CHAINAGE 198+00



CHAINAGE 208+00

PROPOSED INSTRUMENTATION
ON MUSKEG SECTION

PLATE 14

DISCUSSION

Mr. Savage asked about the type and width of structures to be used. Mr. Enright pointed out that there are no water crossings on this road; consequently, no major structures will be used. Mr. Savage inquired further about the availability of cost figures. Mr. Enright explained that he had only tender costs, which indicated $30,000 per mile. The corduroy price is $3 per lineal foot. A question was raised with regard to the depth of compacted fill over the corduroy. Mr. Enright replied that it is of the order of 3 ft.

Mr. Monaghan wondered about the water level in areas adjacent to the road right-of-way. Mr. Enright said that there is very little fluctuation in the water level throughout the year in the deeper muskegs. The shallow muskeg area is flooded over a 4-mile stretch and the water level may vary 1 to $1\frac{1}{2}$ ft throughout the year. In this area, the fill will be 2 to 3 ft above water level. Mr. Enright pointed out that there is a good gradient in this 4-mile long area and good drainage will be obtained once it is initiated. If it were not drained, he thought that trouble might be encountered during run-off.

Professor Goodman raised the question of the relative efficiency of single tube vs double tube type piezometers. Professor Anderson said that he used the single tube type piezometer. When the water level was above the casing, variations could be observed visually. When it fell below the casing, measurements were taken using an electrical contact. Professor Goodman remarked that he has had trouble with the Bourdon gauge type of measurement.

II.3 RECENT EXPERIENCES IN MAJOR HIGHWAY AND CONSTRUCTION DEVELOPMENTS OVER PEAT LANDS NEAR VANCOUVER, B.C.

by

N. D. Lea

Four years ago the author was requested to recount some of his experiences on construction of highways over peat in the lower B.C. mainland and subsequently some notes appeared in the Proceedings of the 4th Muskeg Research Conference (1). During the past four years considerable further experience has been gained. For instance, the Burnaby Freeway project is nearing the completion of the construction phase and there has been a great deal of data gathered. This has not yet been analysed sufficiently and put into adequate form for a proper technical paper but it is hoped that this will be accomplished before the end of this year. In the meantime, however, it is possible to present an interim and extemporaneous discussion on work in progress even though a formal technical paper cannot be forthcoming at present.

The Vancouver area has as many different types of terrain as one would expect to find in a large country. The developed area now covers not only a substantial part of the Fraser River Delta with its erratic fluvial deposits, but also covers a large area of varied till deposits and glacial outwash and bedrock. Some developments are below sea level and protected by dykes whereas other developments go up the mountain sides to over 1000 ft elevation. In the early stages of development in Vancouver City itself and in the city of New Westminster the large areas of swamp land were completely avoided, but now open spaces are disappearing and developments are coming more and more into contact with the peat lands. Some of the major peat deposits are marked on Fig. 1. Quite an intensive study has been made of the peat deposits in the municipality of Burnaby. There are two such deposits: the Central Burnaby peat area and the South Burnaby peat area. Together these cover approximately 20 per cent of the area of the municipality or about seven square miles. Richmond has a couple of peat areas, one of which is known as the Richmond Bog. Together, they constitute approximately 30 per cent of the land area of Richmond or about 12 square miles. Delta also has the Delta Bog - some 30 per cent of its area or about 15 square miles. There are other scattered patches of bog, in Maillardville and in the south of New Westminster for instance, bringing the total amount of peat lands in the Metropolitan Vancouver area to about 40 square miles.

Figure 2 shows the Burnaby Freeway section of the Trans-Canada Highway which connects the Second Narrows bridge with the Port Mann bridge. This is a 12-mile stretch of ultimate 8-lane freeway, designed to the highest freeway standards, fully grade separated and with some 22 grade separation structures in these 12 miles. Between Port Mann and Vancouver a number of peat deposits are traversed. The first of these is the Maillardville peat area - almost two miles of which is traversed - but which is only about 10 ft deep. It was here that the Maillardville test section - the first of a series - was built some four years ago. The Cariboo interchange, which is adjacent to Burnaby Lake, just comes to the edge of the peat with some of the interchange loops being on peat. The Central Burnaby peat bog is skirted as much as possible but at the Sperling-Sprott interchange construction is right in the bog. This was not the original intention. A line was laid out initially which was the best route through this difficult section of terrain, avoiding the deepest peat areas which are the most troublesome sections. However, the municipality objected to that location on the basis that it used their best land and they wanted to use it for something else. They suggested that the highway be relocated to the poorer terrain. The provincial government agreed to do this, even though it was recognized at the time that it would involve considerable extra cost. Consequently, in the final location there is a mile of centre-line crossing peat in the Sperling-Sprott area, as well as about a mile of secondary roads and interchanges. In the Willingdon Avenue area, furthermore, there is about 3/4 mile of freeway across peat as well as 3/4 mile of secondary roads. In the First Avenue area peat is again encountered for about half a mile. A total of about 4 miles, therefore, of this 12-mile stretch of freeway is across peat in addition to another couple of miles of secondary roads. This highway, as has already been pointed out, is under construction. It should be partially paved this year, the pavement completed next year and the road open to traffic in 1963.

This discussion will concentrate on the section embodying the Sperling-Sprott interchange. All peat sections have been well instrumented but there is more instrumentation at the Sperling-Sprott interchange than elsewhere and it also presents the most difficult problems. There were three test sections at this location. Surcharge is on throughout and has been partially removed.

The stratigraphy in this area is briefly as follows: The top 0 - 14 ft is a loose fibrous peat. From 14 to 27 ft is a loose amorphous peat. Underlying this, from 27 to about 60 ft, is an extremely soft, sensitive silty clay, and below 60 ft is firm soil. The fibrous peat has a moisture content of 400 to 1200 per cent; the amorphous peat a moisture content of 200 to 600 per cent. This gives void ratios in the fibrous material of 8 to 17, in the amorphous material of 3 to 8. The shear strength is more or less constant throughout the peat with some slight increase with depth, and is in the order of 0.05 tons per square foot. The consolidation properties are perhaps more interesting. The magnitude of the gross settlements in the peat have been about what would be predicted from consolidation tests in the laboratory. Figure 3 shows excellent agreement between predicted settlement and actual settlement in the peat under a load of about 6 ft of material. The lines are the predictions from laboratory tests in the Sperling-Sprott and Willingdon areas and the dots are actual measured settlements in the peat strata alone. It will be observed that data is presented for thicknesses ranging from 10 to 30 ft. The magnitude of the settlement under a half-ton load is in the order of 30 per cent of the thickness. This is fairly consistent, except for a couple of points.

The rate of the settlement is also very interesting. This is more difficult to predict than is the magnitude of settlement in organic as well as in inorganic soils. Experience in inorganic soils has been that usually the observed rate in the field is greater than that predicted from laboratory tests, simply because the drainage conditions in the field are usually different from the laboratory. At the 1961 Muskeg Conference, J. R. Lake from Scotland presented some rather surprising results which purported to show that the rate was independent of the thickness (2). This does not agree with observations made during these investigations. Theoretically, applying only Darcy's Law, the time should be proportional to the square of the thickness. Lake observed from his laboratory tests that the time rate was independent of the thickness. The author has assessed data on thicknesses ranging from 1 in. to 30 ft. Data is available on laboratory consolidation tests at 1 in., $1\frac{3}{4}$ in., $2\frac{1}{2}$ in. and 6 in. as well as data from a number of field installations ranging in depth from 5 or 6 ft up to 30 ft. When this information was all plotted and compared, it was found that for a given degree of consolidation instead of t = constant (as given by Lake) or t = constant x H^2 (from Darcy's Law), it was something in between. Actually, on the basis of the laboratory data alone, with barely any field observations, the author's data comes fairly close to the square relationship (3). However, taking into account the whole range of data on time-settlement available to the author, it becomes something more like t = constant x H. There is, of course, considerable scatter and results must be analysed very carefully before any indication is obtained regarding the basic laws which may be governing the settlement of peat. The presence of gas, three-dimensional drainage and non-isotropy are all complicating factors. Possibly one such complicating factor influencing Lake's laboratory tests is a film of gas bubbles. This sometimes happens in laboratory work on peat and would, of course, explain the results which he has reported.

Under a load of 6 ft of fill there was a predicted - and an actual - settlement in the peat of 7 ft. In addition, there was a settlement in the underlying clay of the order of 3 ft. In this section, therefore, with an applied load of 6 ft of normal fill material, there is a total settlement of about 10 ft. It is plainly evident that this creates a problem, particularly since the water table is right at the ground surface. In essence, the solution to this problem was that prior to filling with the weighty granular material, some 10 ft of lightweight fill - sawdust - was added to take up the settlement. Under the weight of the necessary base course material plus the surcharge, it was proposed to push the top surface of the sawdust down to about the water table.

This has been the subject of a great deal of study. It has involved the three test sections referred to above. In general, the results have been quite gratifying, although there have been some problems. One of the first things investigated with regard to the sawdust was its durability. It was found, of course, that it will rot unless it is kept continuously submerged. Consequently, as a basic requirement, it appeared that the sawdust should be continuously submerged. It was also found that there is very definitely a fire hazard in rotting sawdust and a number of incidents are on record where there has been spontaneous combustion of sawdust piles. This was another good reason why it was necessary to keep the sawdust below the ground water table. It will be immediately appreciated, therefore, that this meant working between two rather narrow limits. The top surface of the sawdust had to be pushed down below the water table, yet the surface of the road had to be kept above the water table. In fact, it was desirable to keep the base course out of the water. Consequently, it was necessary to predict the amount of settlement with considerable accuracy. The correct amount of sawdust must be placed initially because once the granular material has been added and then the surcharge material on top of that, the sawdust thickness can only be changed after removing the fill. Incidentally, a reasonable price was obtained - 50 cents per cubic yard, in place including supply.

The sawdust has performed very well in construction. It could be worked in any kind of weather. The compactive effort used did not seem to be critical, and it was simply compacted with trucks. The moisture content also did not seem to be critical. Under the half-ton loading which has been added for preconsolidation, the sawdust has compacted to some 80 - 90 per cent of its original thickness. This has been fairly uniform, however, and its trafficability has been excellent. Not only could the contractors drive over it equipment but it could be easily driven over with an automobile, which showed excellent trafficability and quite good stability and strength. Therefore, it does add considerably to the strength of the base course.

One of the biggest problems with this procedure, however, is the clay which underlies the peat. When the project was started it was thought that the peat was the big problem and attention was concentrated on that. As the project has progressed it has been found that the clay underlying the peat actually is a far greater problem than the peat itself. This clay is a sensitive material with a moisture content considerably above the liquid limit. It is normally loaded under perhaps 20 ft of peat, but the effective weight of the peat is only 5 lb per cu ft. The clay, therefore, is normally loaded under approximately 100 lb per square foot or less and thus it is in an almost fluid condition. Sand drains have been used with some success but there is still a significant amount of settlement. It is a very considerable problem to predict with confidence just how much this settlement will be. Stability is also a problem. Lateral movements have been substantial but this is not regarded as alarming. Indeed, if one is to obtain the most economical design for a project of this type some slip failures are to be expected during construction. Some have occurred on this project, usually when the rate of construction was too high. In most instances this rate was higher than the inspectors had instructed; in some instances it was as instructed. There are a couple of locations where it is necessary to excavate the surcharge - in one case to add more sawdust, in the other to remove sawdust. However, these represent a very small proportion of the over-all area of sawdust treatment - something less than 10 per cent - so that it can be stated that by and large the prediction of the amount of settlement has been correct.

Another problem which is worthy of mention is the effect of the surcharge. This is something which is difficult, and yet extremely important, to predict. It was attempted first of all to predict the surcharge effect by laboratory tests and then by full-scale test sections. Now the performance of the actual highway is being observed in the different locations which are instrumented. In one test section - which was the guiding one for the Maillardville area - the surcharge was left on for one or two months. At the end of the surcharge period, the rate of settlement was about 2 ft per yr. After the removal of surcharge, the rate immediately dropped to several inches per year and after a period of several years, it has now reduced to something like 1 in. per yr. The surcharges are now being left on longer - until the rate of settlement reduces to about 6 in. per yr. It is expected that in this way the rate of settlement will decrease immediately after the removal of surcharge to a fraction of an inch per year and that after a few years this will become insignificant. This is the hope and expectation after quite a thorough study, but there are still some uncertainties involved. We shall certainly soon have an opportunity to see how accurate the predictions have been. This is one sobering thing about engineering works - they remain for all to observe and to criticize. The author predicted several years ago that in those sections where the highway is being built over peat, even though the most extensive precautions are being taken that can be justified, it still might be expected that there will be some slight unevenness in the road surface. It is certainly not expected, however, that there will be any unreasonable amount of maintenance required.

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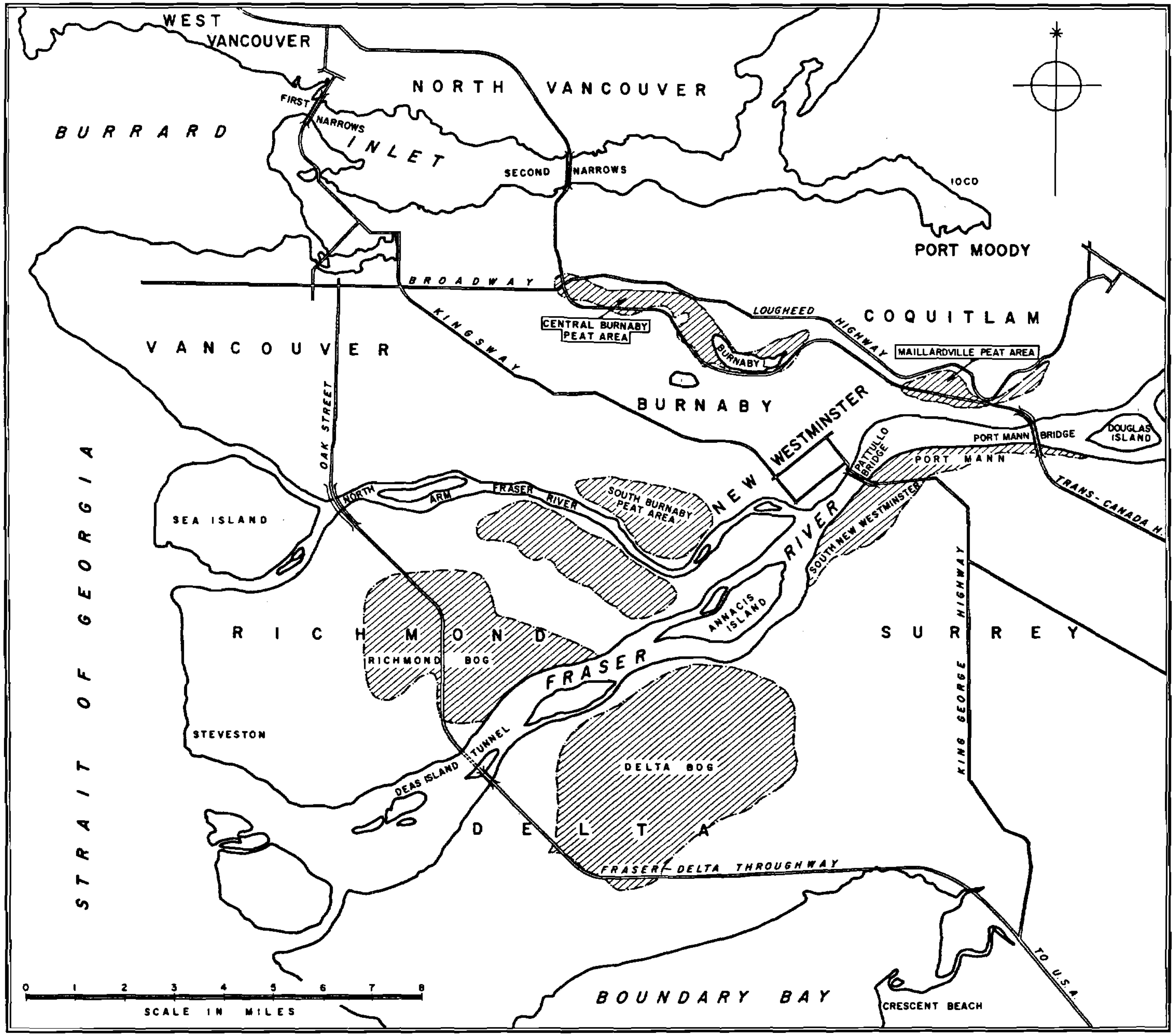
2. Lake, J. R. Investigations of the Problem of Constructing Roads on Peat in Scotland. Proc., Seventh Muskeg Research Conference, National Research Council, Associate Committee on Soil and Snow Mechanics, Tech. Memo. No. 71, p. 133-148, Ottawa 1961.

3. Lea, N. D. and C. O. Brawner. Foundation and Pavement Design for Highways on Peat. Proc., 40th Convention, Canadian Good Roads Association, Vancouver 1959, p. 406-424.



GREATER VANCOUVER PEAT AREAS

FIG. I

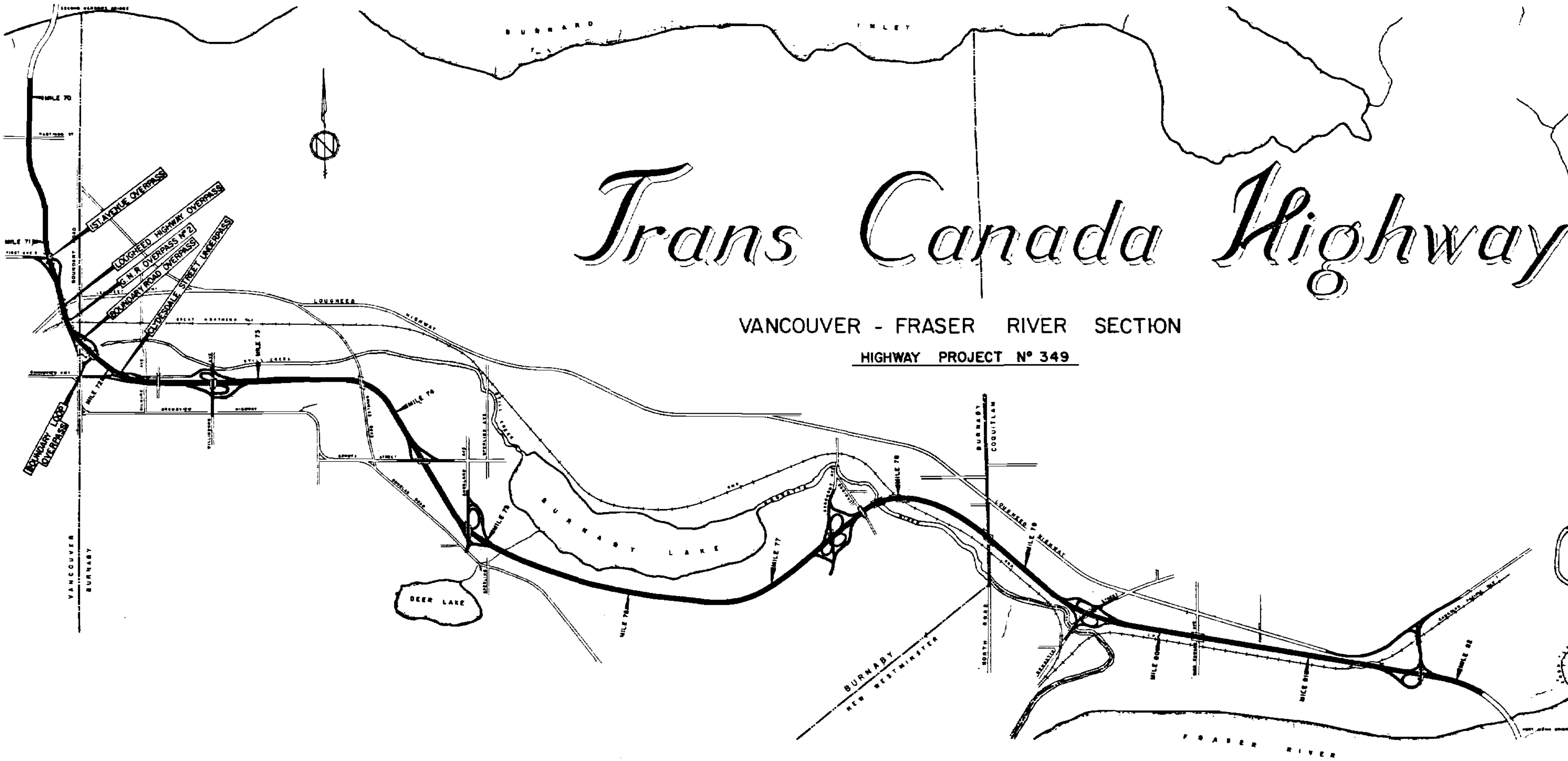
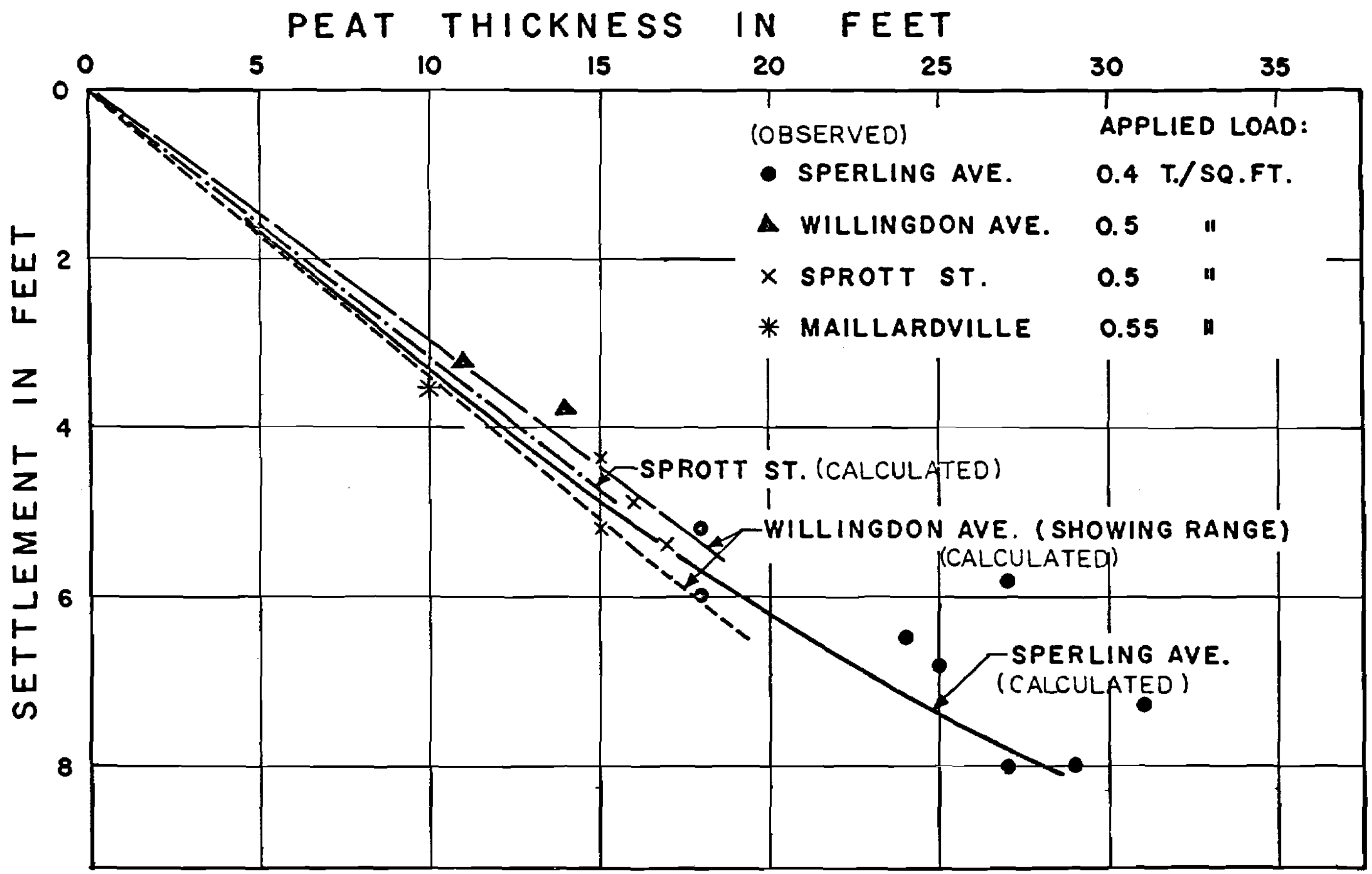

Trans Canada Highway
VANCOUVER - FRASER RIVER SECTION
HIGHWAY PROJECT Nº 349
BURRARD INLET
ST. AVENUE OVERPASS
LOUGHEED HIGHWAY OVERPASS
G. N. R. OVERPASS Nº 2
BOUNDARY ROAD OVERPASS
CLYDESDALE STREET UNDERPASS
BOUNDARY LOOP OVERPASS
VANCOUVER
BURNABY
BURNABY LAKE
DEER LAKE
BURNABY
COQUITLAM
BURNABY
NEW WESTMINSTER
FRASER RIVER

FIG. 2



CENTRAL BURNABY PEAT AREA:

OBSERVED & CALCULATED SETTLEMENTS UNDER ℄ OF TRANS-CANADA HIGHWAY EMBANKMENT LOADS.

FIG. 3

DISCUSSION

Mr. Enright asked about the slope of the embankments. Mr. Lea replied that the slopes are quite variable. In places there is an extensive system of berms (4 or 5, going out as far as 600 ft) where needed for stability.

Dr. Hardy asked if Mr. Lea had any information on indications of secondary consolidation to be expected, if any. Mr. Lea demurred, indicating that secondary consolidation is difficult to define. Mr. Fox wondered if the high lateral movements did not indicate secondary consolidation. Mr. Lea thought not, since these horizontal movements occur without volume change. They are plastic deformations or shearing deformations.

Professor Goodman requested clarification on the size of peat samples obtained for laboratory consolidation tests and also on the time element used in the laboratory for load increments. Mr. Lea explained that his best results were achieved using the Swedish Foil Sampler (2.8 in. diameter) as well as from chunk samples from test pits. Three-inch diameter Shelby tubes with and without pistons have also been used. He has concluded that sample disturbance is not an important factor and that scatter in the results is caused by the non-homogeneity of the material. With regard to load increments, Mr. Lea stated that most of their laboratory tests were a single load increment put on and left on for a long period of time. Professor Goodman asked if a rapid consolidation (for the single increment) was observed, followed by a gradual falling off. Mr. Lea replied that the curve in a general way follows the normal curve for inorganic soils, with more scatter perhaps. He mentioned that originally he was concerned about the turn-down of some curves on the log plot, but came to the conclusion that this is simply a function of the log paper and does not signify anything fundamental. The rate of settlement may be decreasing, or it may remain the same, and the curve will still turn down. Mr. Lea maintained, therefore, that plots on arithmetic paper give much better results.

Mr. Hughes questioned Mr. Lea about differences in experience (such as rate of loading, etc.) at the Maillardville section and the sawdust section. Mr. Lea said that there are many differences in the different sections and emphasized that each particular section has to be carefully studied to see what rate it can take. Where trouble has been experienced, Mr. Lea stated that it was because they had been loading too quickly. The trouble has been with the soft clay underlying the peat, rather than the peat itself, which picks up strength quickly as it is consolidated.

Dr. Golder, referring to the question of secondary consolidation, pointed out that he has discussed this with the Dutch who get 80 per cent of the total consolidation in the secondary phase. They define secondary consolidation as "anything that cannot be explained by Terzaghi." Dr. Golder raised a number of questions as follows:

1. Shear strength was quoted as 0.05 T/sq ft. How was this obtained? Do tests

for mineral soils work for organic soils?

2. For the sawdust which has to be below the water table, how does Mr. Lea know where the water table will be in 50 - 100 years time? The same sort of problem exists in Amsterdam, where the water table has to be maintained at a fixed level, or the tops of piles will deteriorate.
3. Was any attempt made to raise the price of sawdust after it was found to be useful?

Mr. Lea replied as follows:

1. Several methods were used to determine shear strength, such as the in situ vane test and the laboratory triaxial test, but the shear box was not used. The most confidence is placed in field observations, however. One advantage of slip failures is that they give a full-scale measure of the strength of the peat. Analyses of these agreed very well with vane test results.
2. Variation in the water level is a very important factor. In the area in question, the water level will be controlled by the water level behind a dam on Burnaby Lake. Application has been made to the Water Resources Board that the water level will not be raised or lowered so as to affect the sawdust fill, without advance notification of the Department of Highways.
3. The price for the sawdust is based on very large quantities and has remained stable.

Mr. Fox queried Mr. Lea about the construction of culverts. Mr. Lea said that, in general, culverts were placed in the embankment material so that they would move with it. Mr. Fox wondered if accurate estimates could be made of the final elevation of the culverts Mr. Lea replied in the affirmative, stating that it was necessary to do so, especially in the sawdust area.

Mr. Schlosser requested clarification on how slipping was prevented. Mr. Lea explained that this was accomplished by berms and a slow rate of loading.

II.4 CONSTRUCTION OF THE PORT PERRY CAUSEWAY

by

G. A. Wrong

INTRODUCTION

One of many interesting highway construction projects undertaken in the Province of Ontario within the last five years has been the rebuilding of the two causeways across the southern portion of Lake Scugog. As illustrated in Figure 1, this lake is located 28 miles east of Metropolitan Toronto and 18 miles north of Lake Ontario.

These causeways form part of the feeder King's Highway No. 7A which provides a direct route for the residents of Port Perry and the surrounding smaller communities to the larger industrial towns of Lindsay and Peterborough. During the summer months this highway also provides access to the Kawartha Lakes resort area located to the northeast.

About the year 1830 the early settlers of what is now known as the town of Lindsay dammed the Scugog River to harness its power for the purpose of operating a saw mill. This dam resulted in the flooding of 60,000 acres of land and the forming of Lake Scugog. Though the dam was later lowered as a concession to the farmers in the area, Lake Scugog as it stands today can be considered as man-made.

The older inhabitants in the vicinity of this lake report that the mainland at Port Perry was originally connected to Scugog Island by a floating timber bridge constructed over the low marshy deposits in the lake. Between 1880 and 1885 this bridge was sunk in place by constructing a fill over it until equilibrium was reached. This embankment, 2600 feet long, then became known as the Western Causeway. It wasn't until about 1890 that the Eastern Causeway, 9700 feet long, was constructed from Scugog Island to the Mainland on the east side of the lake. A shallow fill was placed directly over brush and logs throughout the organic deposits in the lake.

Since both of the embankments were constructed over organic material in Lake Scugog, minor settlements occurred almost continually. This condition resulted in continued maintenance and when sections settled close to lake level, shallow granular lifts had to be placed over the causeway sections. In addition, during the spring months severe flooding occurred over the shallow fills in the lake, often isolating the residents of Scugog Island.

FIELD INVESTIGATIONS

In view of the low traffic volume (less than 600 A.A.D.T.) at the Causeway sections on Highway 7A, little was done to alleviate the flooding conditions and continuous minor settlements. In the summer of 1958 the portion of Highway 7A between Port Perry and the Caesarea Sideroad, which included the

Causeway sections, was programmed for reconstruction. A detailed subsoil investigation was carried out in the winter of 1959 to define the type and extent of the soil strata adjacent to and underlying the existing causeway embankments.

Subsoil Investigations at the Causeway Sections

Sections of the causeway investigated extended from Station 158+00 to Station 184+00 in Reach and Scugog Townships and Station 260+00, Scugog Township to Station 58+00, Cartwright Township. These sections are hereafter referred to as the Western and Eastern Causeways, respectively.

In addition to the drilling investigation through the shoulders of the existing embankments, detailed sampled borings, supplemented by a geophysical survey, were carried out along three parallel lines offset 75, 125 and 175 ft to the north on the Western Causeway and 125 ft on the Eastern section (see Figure 2). A few sampled borings were made at offsets of 125 ft to the south of the Western Causeway to confirm that similar conditions existed both north and south of the existing embankment.

Discussion of Soil Types Encountered

1. Western Causeway along existing embankment:

The combined depth of the existing fill, brush and old timbers varied from 6 to 20 ft. The depth of the underlying peat to firm sand bottom varied from 5 to 20 ft. The soil stratigraphy along the existing embankment is shown in Figure 3(a).

2. Western Causeway at offset lines:

The subsoil conditions encountered along all of the offset lines to the north were essentially the same. The thickness of each stratum as measured in the investigations were as follows:

a) The depth of water varied between 2 and 3 ft except at the equalizing stream channel at Station 171+50. At this location the water depth was approximately 9 ft.
b) In each of the borings the very soft dark brown fibrous peat was found to vary from 10 to 12 ft. The peat material is characterized by very short fibres in a highly decomposed state. The moisture content of this organic layer was approximately 400 per cent, and vane tests indicated a shear strength in the order of 150 psf.
c) The peat deposit was underlain by a stratum of very soft yellowish grey marl. The thickness of this layer averaged 10 to 12 ft. In situ shear strength values in the marl stratum were found to average 150 psf, and the moisture content approximately 250 per cent.
d) Underlying the marl stratum, a layer of medium dense silty sand was encountered. The thickness of this layer varied between 5 and 10 ft.
e) The fine-grained granular deposit of silty sand was found to be

underlain by a dense clayey till stratum. The thickness of this till deposit was not determined beyond 11 feet (i.e. elevation 787.5 ft).

3. Eastern Causeway along existing embankment:

The depth of fill material was found to vary from $4\frac{1}{2}$ to 9 ft. The thickness of the peat and muck below the fill varied from 9 to 19 ft to firm bottom, and there was little or no evidence of the brush or corduroy used in the construction of this causeway. The soil stratigraphy along the existing embankment is illustrated in Figures 4(a) and (c).

4. Eastern Causeway at offset lines:

The subsoil stratigraphy along the line offset at 125 feet north of the existing centerline was as follows:

a) The major part of the line investigated crosses a low-lying shrub vegetation. The only water portion occurs between chainages 273+00 and 277+00 and its depth here was not over 2 ft.
b) The depth of the peat layer was found to vary in thickness from 8 to 15 ft. This organic layer was found to be firmer and more fibrous than that encountered on the Western Causeway section. The average shear strength value measured was in the order of 200 psf
c) The very soft marl stratum, found to be continuous along the Western Causeway offset lines, was encountered only in localized areas along this Eastern section. The thickness of these marl pockets varied from 5 to 10 ft. The average shear strengths and moisture contents of this layer were in the order of 150 psf and 250 per cent respectively.
d) The soft compressible deposits of peat and marl were found to be underlain by a dense stratum of silty sand over a dense silty till layer.

Borrow Investigations

Since the borings along both causeway sections indicated that the compressible layers of peat and marl were moderately deep, it was apparent that large quantities of fill would be required if stable new embankments were to be constructed.

In September 1959 an investigation was carried out to determine the types of material that would be available for use as backfill in the organic deposits in Lake Scugog. As shown in Figure 5, the chief geological formations located in the vicinity of Lake Scugog are drumlins, kame moraines, clay, sand and till plains.

The soils in the clay, sand and till plains were found to be extremely variable since they were often overlain by layers of silts and clays. Because of the wet nature of the soils in these formations they were considered unsuitable as backfill across Lake Scugog.

Large quantities of sand and gravel, suitable as swamp backfill, were known to exist in the Oak Ridges kame moraine located approximately six miles south of Lake Scugog. However, due to the large quantities involved and the distances required to haul these materials, their use was considered uneconomical.

Glacial tills found in the drumlin formations, because they are well graded, are generally considered as good embankment construction materials (see Figure 6). Although the Department had little experience with these materials placed across open bodies of water, consultation with Ontario Hydro Commission indicated that similar materials had been used successfully in constructing cofferdams in Southern Ontario. Based on this information, it was decided to use the local drumlinized glacial till soils.

Adequate quantities of such suitable till materials were located both on Scugog Island and on the mainland just beyond the Eastern Causeway section.

SWAMP TREATMENT

The performance of the existing causeway embankments indicated that a fill 5 ft above lake level would be required to overcome the flooding conditions during the spring months. The additional fill on the existing roadway embankments would result in overstressing of the organic deposits with possible embankment failures and continual consolidation of the soft compressible strata.

In addition, reconstruction of the causeway embankments along the existing alignment could present other difficulties since through traffic had to be maintained. Closing this road was considered undesirable as it would result in a long detour around the southern tip of Lake Scugog.

On the basis of the boring data obtained in the field investigations it was realized that the soils stratigraphy at all of the offset lines were essentially of the same depths and condition. Constructing new stable embankments along any of these lines would certainly result in large swamp excavation quantities and an even greater volume of backfill.

After considering the various possible treatments and alternate alignments the following procedures (illustrated in Figures 7 and 8) were recommended:

1) The proposed centerline alignment should be shifted 25 ft to the north or south of the center of the existing embankments. This would mean that the existing causeways would be located under the shoulder of the future roadway and during construction could be used to maintain one-way through traffic.
2) The partial excavation and displacement method of treating these swamp sections was recommended. The maximum practical depth of excavation was assumed to be 10 ft and this depth should, for the most part, assure complete removal of the peat.

3) The excavation should extend from the edge of the existing pavement to a point where the 1:1 slope from the proposed embankment shoulder edge intersects the bottom of the peat. The width of excavation, therefore, would be approximately 40 feet. The wide excavation under the new fill would assure stability, while over the existing roadway stability would be assured by the present embankment.
4) Since the backfilling operation was expected to displace the underlying marl in the open excavation, this material would have to be considered to require excavation as well.
5) It would be necessary to excavate the peat and marl working from the existing embankment or from the backfill. In order to prevent instability and sloughing in of the organic wall of the open excavation, the material removed should be placed behind the dragline and against the new fill.
6) The backfill should be placed angle-wise to the direction of the centerline and in such a manner to ensure that the marl was displaced away from the new fill.

CAUSEWAY CONSTRUCTION

In August 1960 the grading, drainage, and granular base course contract was awarded on Highway 7A between Port Perry and the Caesarea Sideroad. By the end of that month the contractor had started his excavation and backfilling operations north of the existing Western Causeway, working in the direction away from source of borrow on Scugog Island.

As outlined in the recommended procedures, it was expected that the top 10 ft of peat would be excavated and the underlying marl would displace into the open excavation and also be excavated. However, it became evident shortly after construction commenced on the Western Causeway that because of the fluid state of the peat and marl it would readily displace laterally under the weight of the fill. Most of the displacement of the marl on this causeway section resulted from the very effective use of the D-8 bulldozer working off the existing causeway embankment or the newly placed fill.

The backfill material was trucked from the borrow pit at a rate of about 180 cubic yards per hour. By October 26 the new Western Causeway was completely constructed to its subgrade elevation. During this two month period 91,350 cubic yards of organic material were excavated or displaced and 137,762 cubic yards of backfill placed.

During the months of September and October, the contractor carried out his excavation and backfilling operations adjacent to the existing Eastern Causeway. Because of slight shifts in the alignment, the excavations on this causeway section were carried out both to the north and south of the existing embankment. The backfill material was obtained from two borrow sources requiring an average haul of 3600 ft to this causeway section.

The construction methods used here were similar to those employed in the construction of the Western Causeway. The major difference was that excavation only was necessary since the depth of peat to firm bottom over one third of its length was less than 12 ft. The sections where no lateral displacement of the peat and marl occurred are easily recognizable since the excavated organic material was piled high adjacent to the embankment. In the spring of 1961 it was necessary to trim up these piles of organic material and to cut ditches.

Because of a drainage channel lying approximately 75 feet north of and parallel to a 1500-ft section of the Eastern Causeway, the disposal area for the excavated organic material was restricted. It was specified in the contract that approximately 20,000 cubic yards of organic material at this location be hauled away and disposed of in a borrow area. On the Eastern Causeway section approximately 217,800 cubic yards of peat excavation was required and 325,000 cubic yards of backfill material placed.

In order to establish the effectiveness of this swamp treatment, a detailed drilling investigation was carried out through the new causeway embankments by the Materials and Research Division, in the spring of 1961. As illustrated in Figures 3(b), 4(b) and 4(d), all of the compressible organic materials have been excavated or displaced from beneath the new causeway fill.

It was considered that possibly some minor consolidation or settlement might occur in the new causeway fills, therefore it was decided to stage pave these embankments. In the summer of 1961, 2 in. of H.L.4 binder course was laid over the causeway sections. In the spring of 1962, the performance of the asphalt binder course indicated that no settlement had occurred.

SUMMARY

Many problems were envisaged in the reconstruction of these causeway embankments during its Pre-Engineering stage. These were due primarily to the fact that the causeway sections were located over long moderately deep organic deposits and also because traffic had to be maintained on the existing embankments.

The swamp treatment employed at these locations proved however, to be extremely effective. This can be attributed to the wise selection of the new alignment, the method of construction, the quality of the densely graded till material used as backfill, and to some extent on the ideal weather conditions during construction.

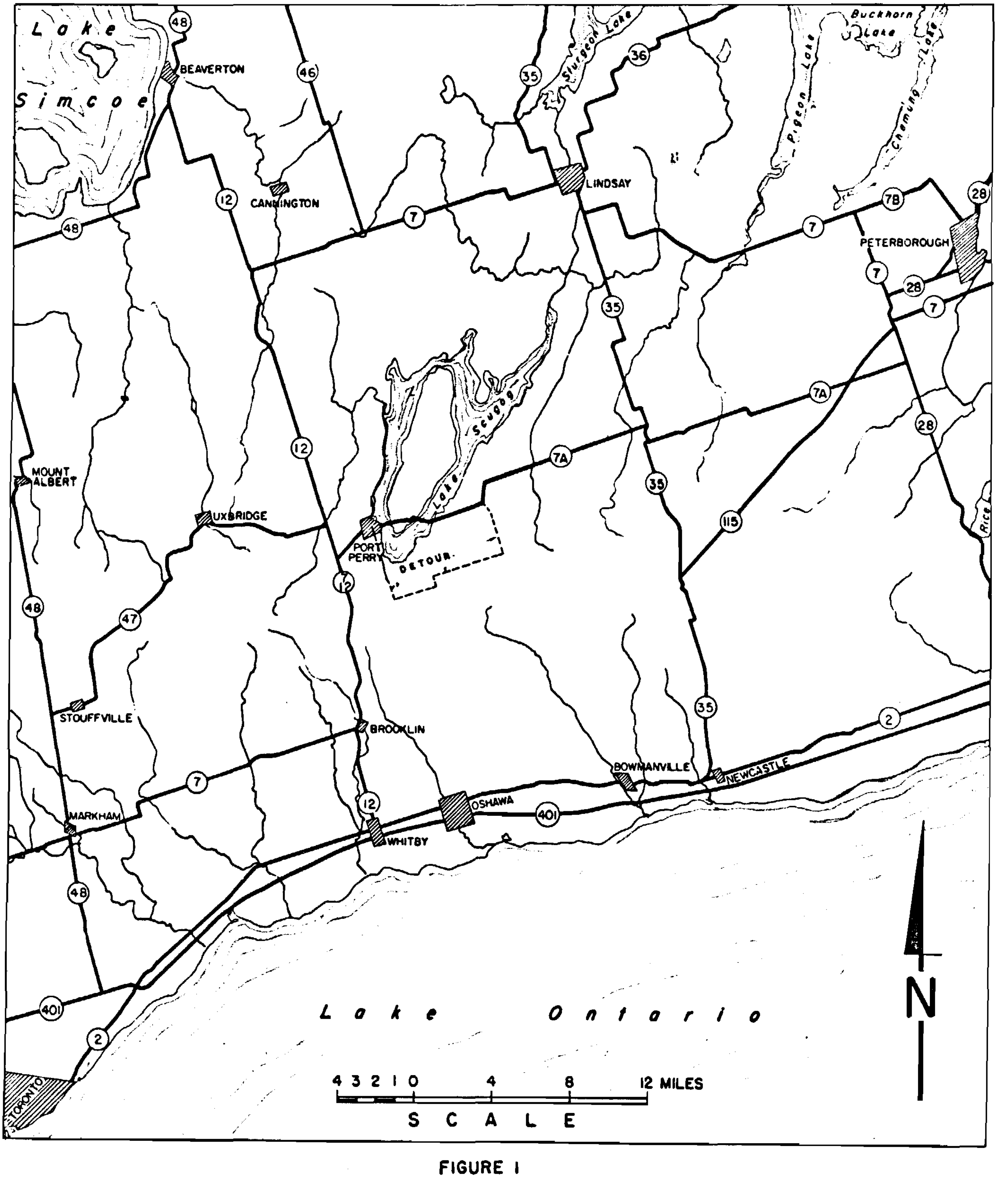
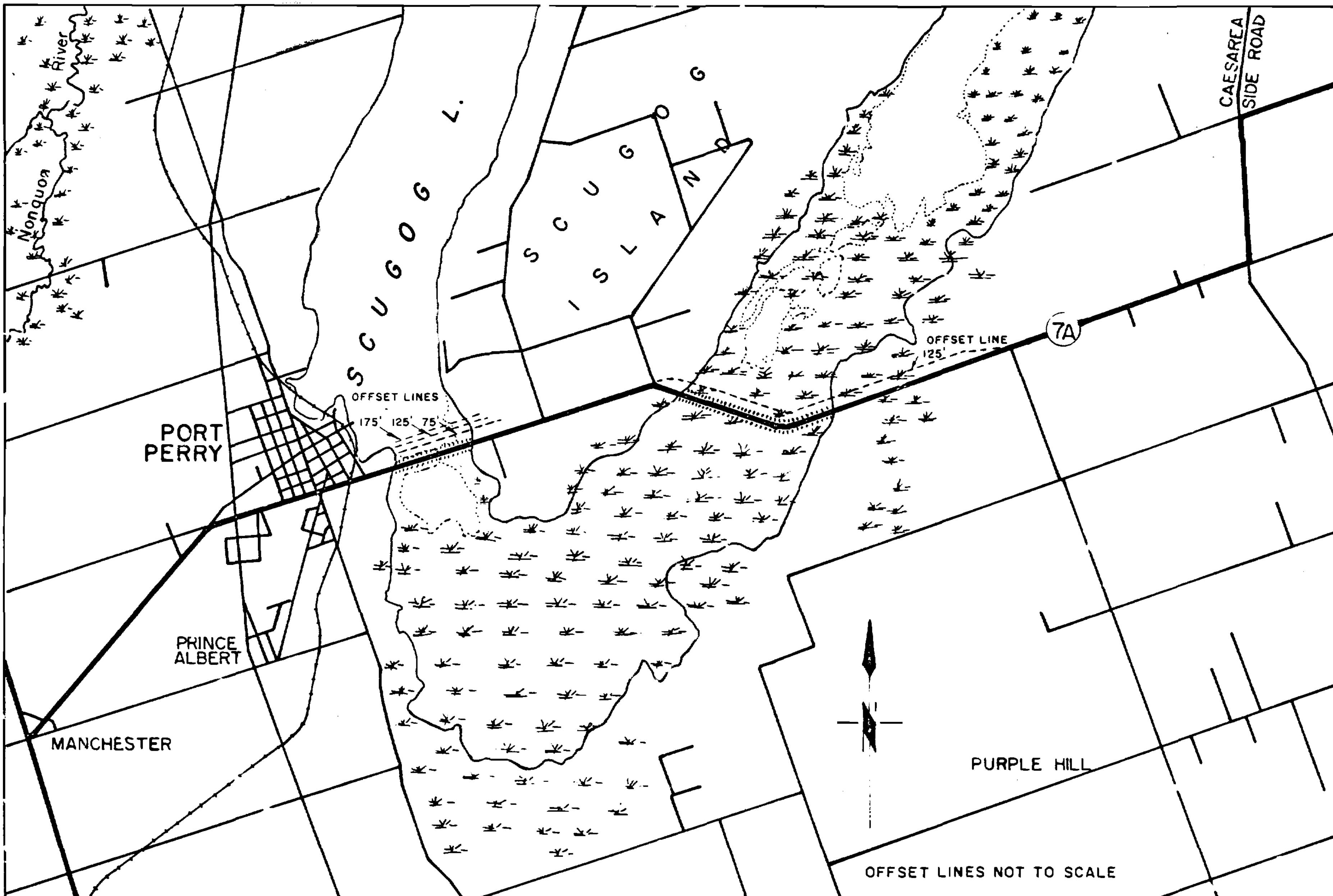

Lake Simcoe
BEAVERTON
CANNINGTON
Sturgeon Lake
Buckhorn Lake
Pigeon Lake
Chemung Lake
LINDSAY
PETERBOROUGH
Lake Scugog
MOUNT ALBERT
UXBRIDGE
PORT PERRY
DETOUR
STOUFFVILLE
BROOKLIN
MARKHAM
WHITBY
OSHAWA
BOWMANVILLE
NEWCASTLE
TORONTO
Rice
Lake Ontario
4 3 2 1 0 4 8 12 MILES
SCALE
N

FIGURE 1



PLAN SHOWING OFFSET LINES INVESTIGATED AT EASTERN AND WESTERN CAUSEWAYS.

FIGURE 2

SOIL STRATIGRAPHY WESTERN CAUSEWAY

TWP'S. OF REACH & SCUGOG

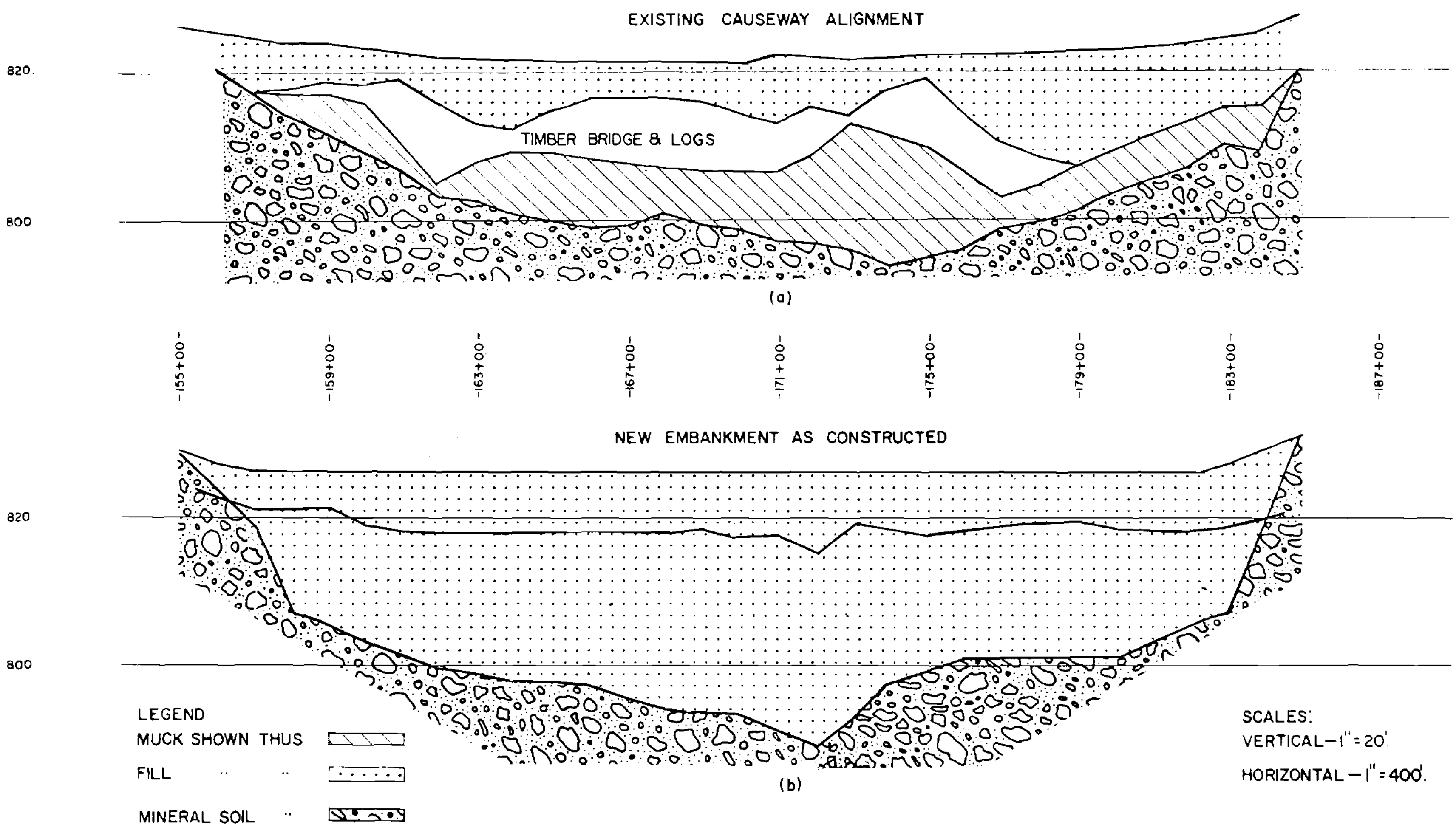
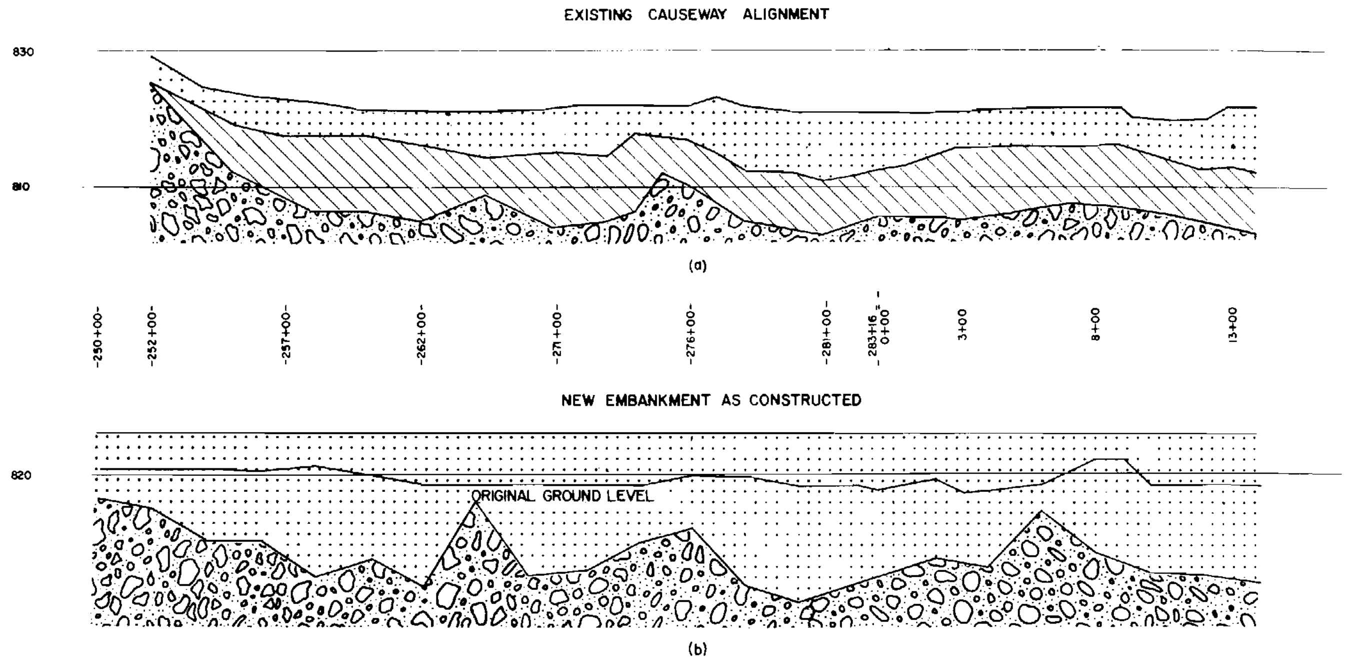


FIGURE 3

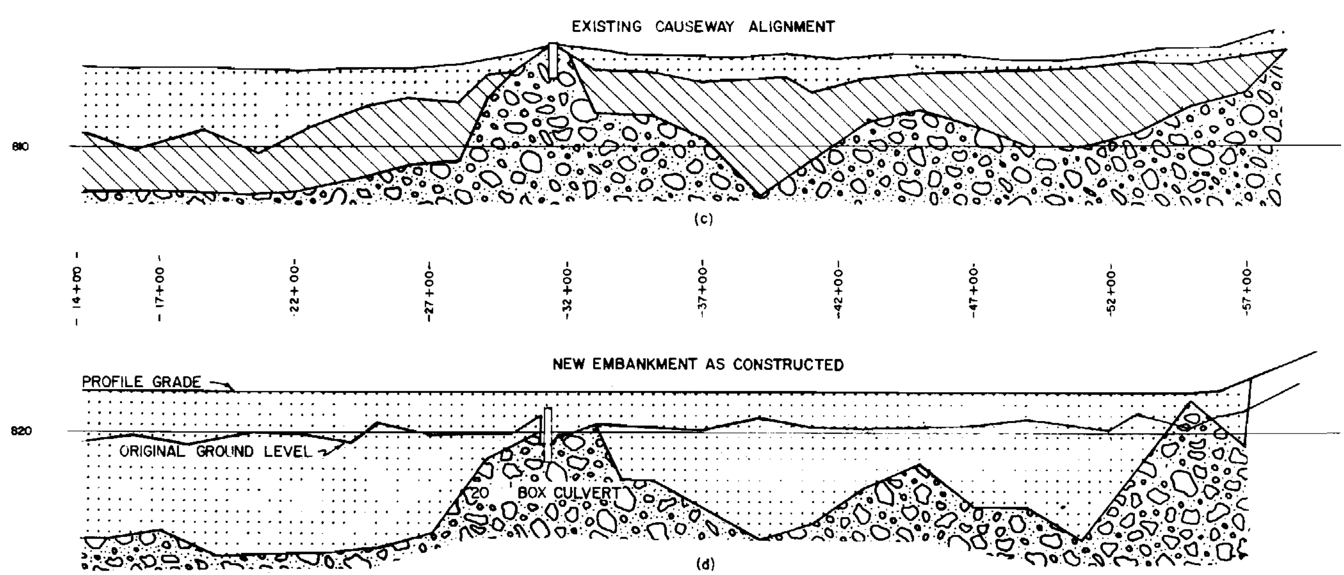
SOIL STRATIGRAPHY EASTERN CAUSEWAY

TWP'S OF REACH & SCUGOG | TWP. OF CARTWRIGHT



SOIL STRATIGRAPHY EASTERN CAUSEWAY

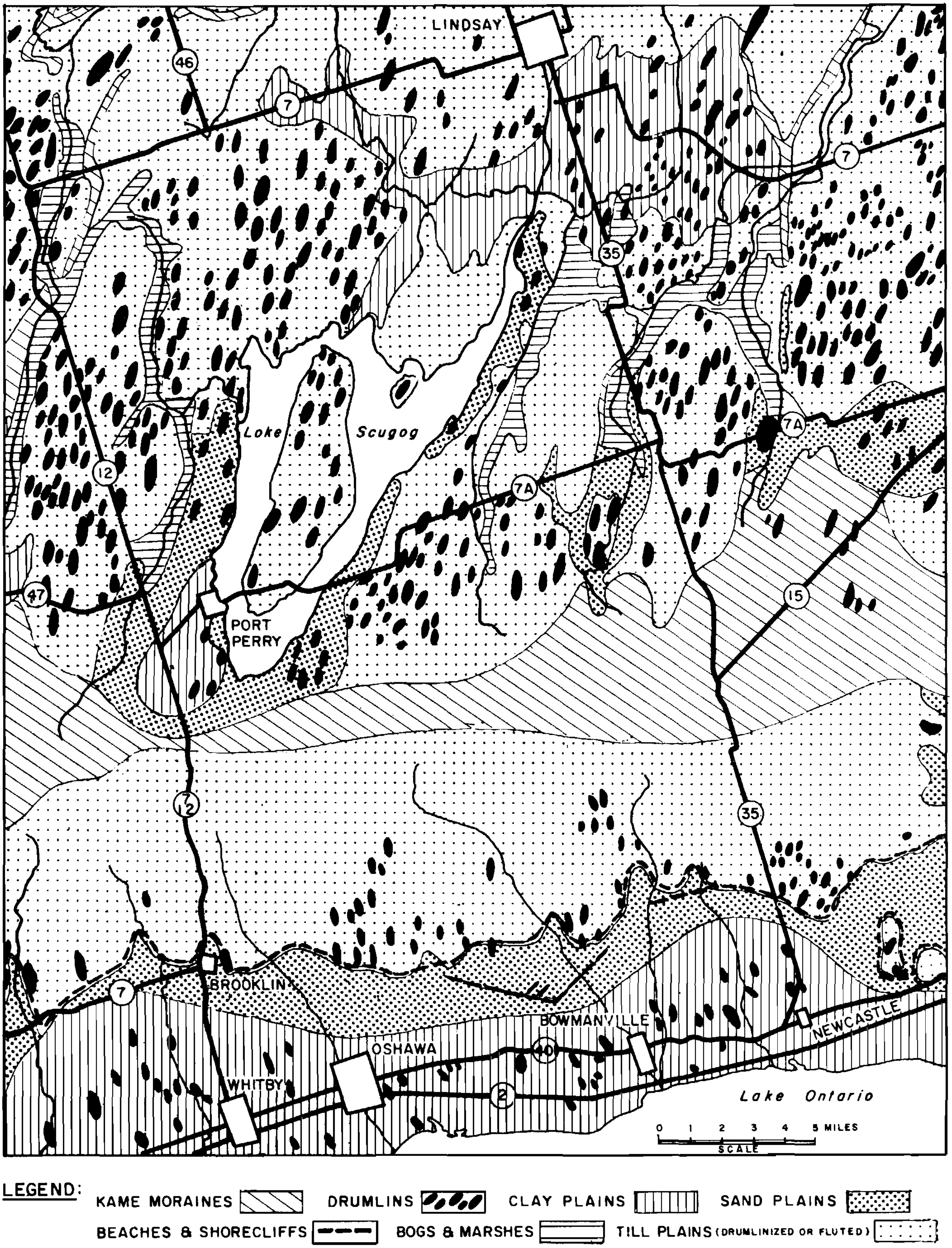
TWP. OF CARTWRIGHT



LEGEND
MUCK SHOWN THUS
FILL " "
MINERAL SOIL "

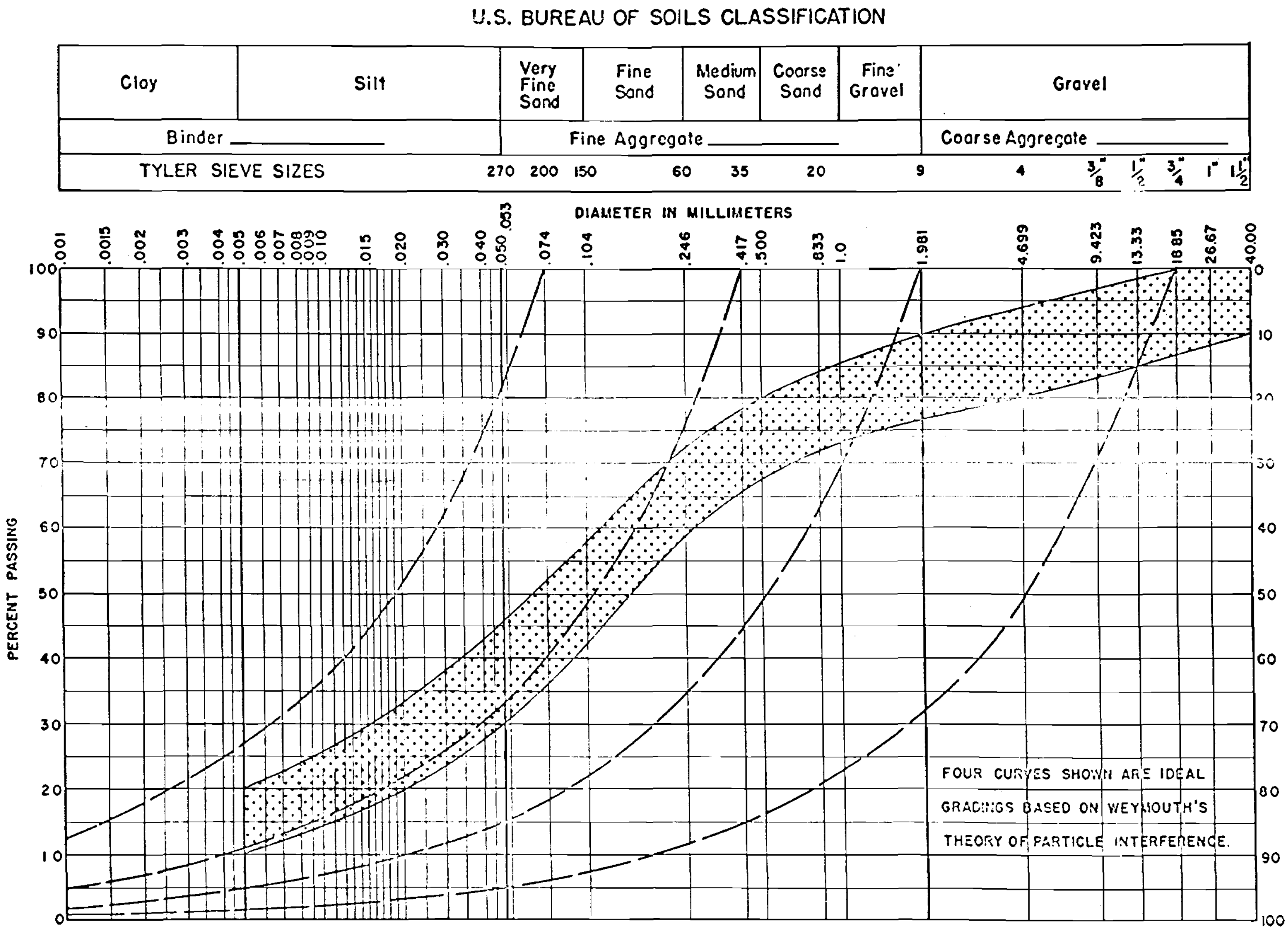
SCALES:
VERTICAL – 1" = 20'
HORIZONTAL – 1" = 500'

FIGURE 4

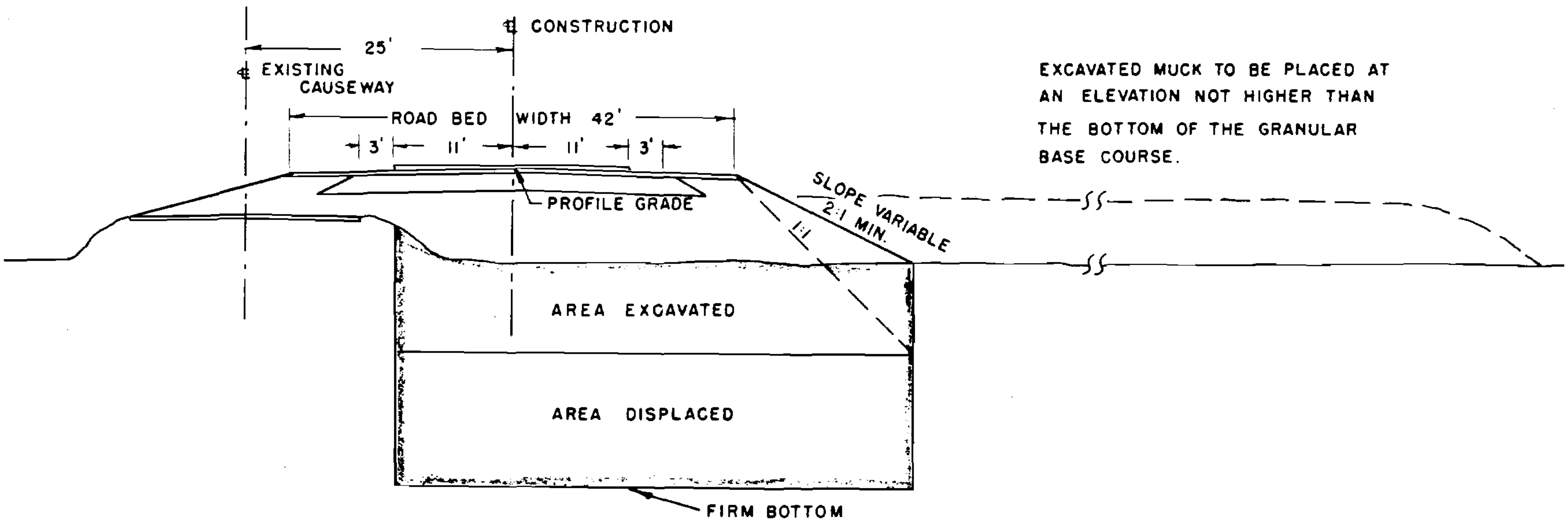


PHYSIOGRAPHY MAP - PORT PERRY AREA

FIGURE 5

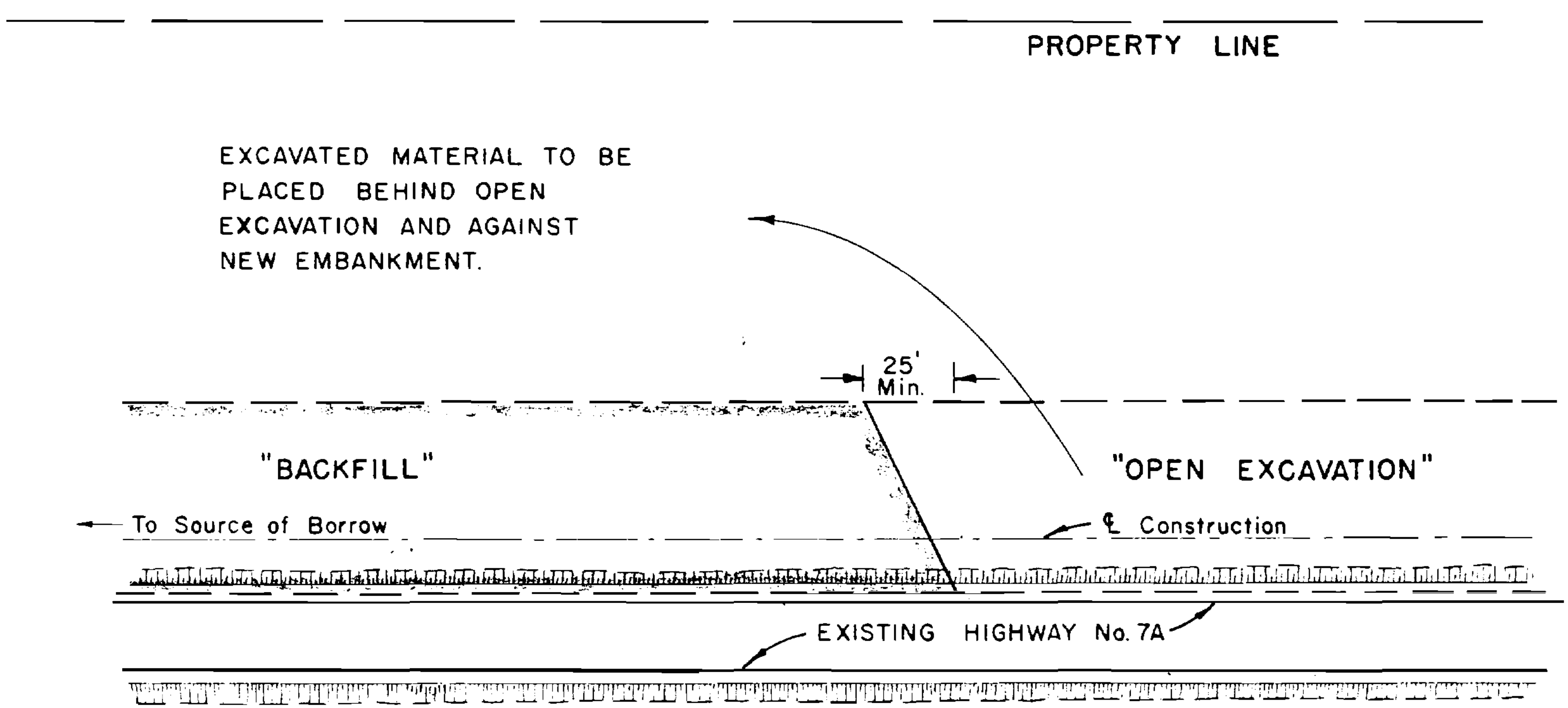


GRADATION RANGE OF GLACIAL TILL IN VICINITY OF PORT PERRY CAUSEWAY
FIGURE 6



TYPICAL CROSS SECTION

FIGURE 7



PLAN VIEW SHOWING
METHOD OF EXCAVATION AND BACKFILL
FIGURE 8

III.1

CULVERT SELECTION IN MUSKEG AREAS

by

C. O. Brawner and R. O. Darby

INTRODUCTION

The engineer is being faced more and more with the problem of constructing roads and railways in muskeg areas. The high cost of right-of-way in populated regions, the lure of forest and mineral wealth in isolated areas, the need for northern development and access for the oil industry account, to a large extent, for this situation.

Generally the initial program is directed toward the selection of a method of construction which will provide a stable grade. Numerous methods have been developed in recent years and found successful under various conditions (1, 2). The most important problems to resolve are undoubtedly an assessment of the rate and magnitude of settlement, foundation stability and grade thickness requirements. In addition, consideration is often required with respect to drainage. While some restriction of the normal drainage pattern in organic terrain will not normally be as critical as restriction in other areas, a disregard of drainage requirements can lead to serious consequences.

It is the purpose of this paper to discuss some factors considered significant in selecting culverts for use in a peat environment. Comments are presented on factors which affect culvert selection, the suitability of various types of culverts and installation and maintenance considerations.

FACTORS WHICH AFFECT CULVERT SELECTION

(a) Settlement - Peat is notorious for excessive settlement, even under very small loading. If construction is proposed on top of the peat without prior consolidation the culverts will settle with the fill. This settlement is usually differential in nature, being much greater under the center of the grade than at the toe of the fill. In this position the culvert acts essentially as a siphon and is very susceptible to plugging. In addition the inlet and outlet may settle below the free water surface. Not only does this reduce the capacity of the culvert to carry water but the bending induced by the differential settlement may cause the culvert to be pulled apart. If the peat is removed and replaced with granular material but is underlain by soft clay, as often occurs, moderate settlement may also occur.

No culvert will function satisfactorily if excessive settlement occurs. Consequently, efforts should be made to minimize this possibility. Some types of culverts will withstand some movements. The least affected are the metal culverts. On several installations in B.C. differential settlement of 2 feet per 25 feet has not caused structural failure. Wood stave culverts will also withstand moderate bending before failure. Concrete culverts on the other hand cannot withstand loading without fracturing the joints and should never be used where settlement will occur.

(b) Lateral Movement - Where the peat is not to be removed the placing of moderate heights of fill at a fairly rapid rate will often induce lateral movement in addition to settlement. This movement may be sufficient to cause a culvert to be pulled apart. Extra care should therefore be taken at culvert locations to control the rate of loading. Current practice in B.C. is to place movement stakes (3) about 15-25 feet from the toe of the fill and take daily readings during construction. If movement exceeds any prescribed limit (3 inches is suggested) further loading is discontinued until consolidation results in sufficient gain in strength for further loading.

Metal and wood stave pipes offer adequate resistance to lateral soil movement provided the movement does not exceed about 6 inches per 50-foot length of pipe. Concrete pipe tends to pull apart much more readily unless cast in a continuous cradle.

(c) Flow Conditions - The design of culverts for specific flow conditions is generally similar for muskeg areas as for normal drainage problems. The Armco Drainage Handbook (4) outlines methods of computing size and capacity which are satisfactory for most installations. Special emphasis is drawn to the increase in culvert efficiency by the incorporation of good inlet and outlet design. With metal and wood stave culverts, special flared metal entrances and exits can be obtained which will increase capacity. These are preferred to concrete headwalls because of light weight and ease of installation. Less expensive alternatives include sodding, sand-bagging or rip rapping.

Normally culverts in muskeg will be placed on relatively level grades. Hence culvert erosion is generally not a problem. In addition ditch scour is usually limited in muskeg. This is due mainly to flat gradients and the moderate resistance peat affords to scour, particularly the more fibrous peats.

(d) Location - The specific location relative to the source of supply and transportation facilities is of considerable importance in selecting the culvert type. Wood stave pipe was found advantageous on the northern Stewart-Cassiar Highway because of significantly lower transportation costs. Aluminum pipe may be economical to move if rates are based on weight. Normally several sizes are included in an order so the smaller culverts can be nested inside the larger culverts for shipment. Concrete culvert is the most expensive to move and becomes more expensive as the distance increases.

(e) Installation - Installation costs can vary greatly depending on the size and weight of the pipe, location, equipment available, etc. Aluminum culverts are generally the least expensive to install because of the ease of connecting and light weight. With the smaller diameters no heavy equipment is required.

Wood stave culvert pipe provides no difficulties for experienced crews and can be quite economical. Specific care must be taken with building the first ring, tensioning the bands and backfilling.

Small diameter galvanized pipe can be installed by hand but large diameter pipe usually requires the assistance of heavy equipment. Concrete pipe is the most difficult and expensive to place. This is due largely to the weight, the large number of sections involved and mortaring required at all joints.

(f) Design Life - The most economical selection of culverts must consider the proposed life of the road or railway. Minor culvert requirements such as untreated metal or wooden structures may be perfectly adequate for temporary routes. For secondary routes a more permanent type of culvert such as galvanized metal, creosoted wood stave, concrete, etc. is desirable. For primary and arterial highways it is suggested culverts should be selected which will have a service life of at least 20 years, preferably 30 to 35 years and for railways, 50 years.

(g) Corrosion and Deterioration - Careful consideration must be given to the type of culvert to be used in muskeg areas with respect to corrosion or deterioration since the materials of which they are constructed - metal, concrete or wood - are all subject to deterioration in such an environment. Various methods of protecting some of these materials are available and the effectiveness of such protection should also be considered.

Of all the factors considered significant in selecting culverts in muskeg, corrosion or deterioration are perhaps the most important. Hence this problem is discussed in some detail.

Many factors are involved in the deterioration process of the materials involved but in each case the process is dependent on water or moisture for its continuance. In muskeg areas it is probable that water will be present, either flowing through the culvert or in the soil surrounding the culvert for a high percentage of the year. Consequently the deterioration process will occur for a greater period each year than is the case where culverts are dry for extended periods in summer months. Investigation of corrosion of metal culverts in California, reported by J. L. Beaton and R. F. Stratfull (5) concluded "that the presence of continuous (or nearly so) water flow indicates a potentially corrosive area."

i) Metal Culverts - In the case of corrosion of steel the water provides the electrolyte necessary for the electrochemical process of corrosion to function. The nature of the electrolyte has a marked effect on the type and rate of corrosion. Low electrical resistivity and relatively high quantities of soluble salts generally indicate high corrosivity. Total acidity and pH are also factors affecting corrosion, but there is less direct correlation between them and corrosivity than there is with resistivity or soluble salts (6). Another important factor in corrosion is aeration; oxygen, either from the atmosphere or from oxidizing salts or compounds, tends to stimulate corrosion. Also local variations in the supply of oxygen to the soil surrounding the pipe can set up oxygen concentration cells; such cells often result in corrosion along the outside bottom of a culvert. Many other specific factors are also involved which are interrelated to each other and to the factors already mentioned. The relationship is complex and even if data on all factors are available in any one case it is still difficult to predict the nature and extent of corrosion that is likely to occur. However, it is normally possible to differentiate between mildly corrosive and severely corrosive conditions.

Since the chemical properties of organic soils are associated with relatively high quantitie of soluble salts, high total acidity and with pH values in the general order of 4.0 to 6.5, the water in contact with culverts in muskeg areas is likely to have the same general properties, and therefore to be highly corrosive. However, depending on the over-all drainage pattern the water in the ditch or stream may be "diluted" to a varying degree by surface water or water originating from soils of different characteristics.

There is general correlation between soil resistivity readings and the amount of soluble salts in soil moisture (6), consequently such readings taken in the soil in the vicinity of the culvert provide a relatively easy method of ascertaining whether the water is likely to be highly corrosive or not. This information coupled with assessment of the over-all drainage pattern may indicate that further information should be obtained on the stream water itself including resistivity, available oxygen, soluble salts, pH and total acidity, etc. It is appreciated that the cost of obtaining all this information may not be warranted in all circumstances, but to date no one test seems to be a reliable predictor of corrosion and therefore the more information that can be obtained the more reliable the prediction.

The most universal method of protecting steel culverts is by galvanizing. The zinc coating performs two functions: it insulates the steel from the electrolyte (water) and also, since it is anodic to steel, will protect small areas of exposed steel. The zinc itself corrodes at a much slower rate than steel in many environments, but in acid environments the rate is still relatively rapid. Galvanized metal in highly acid soils is likely to lose the zinc coating in from 2 to 4 years (Figure 1) and therefore if the water flowing through a culvert has high total acidity it is probable that the life of the culvert cannot be increased significantly by galvanizing alone.

Additional protection may be given to the galvanizing by coating the pipe with a relatively thick coating (50 mils) of asphalt. The asphalt is resistant to the degree of acidity involved and provided it is properly applied and the culvert carefully installed, it is reasonably impermeable to water. The disadvantage is that adhesion to the zinc coating is relatively poor and the life of the coating may therefore be reduced considerably. Adhesion may be improved by the use of asbestos bonded galvanized stock, whereby asbestos fibres are embedded in the molten zinc at the time of galvanizing to provide an anchor for the asphalt coating. Provided that the entire process is carefully controlled, that the asphalt has optimum properties, and that care is taken during installation it is reasonable to suppose that the service life in corrosive waters can be materially increased by this method of protection.

In the case of large installations, where the cost of replacement would be very high, and where it is not practical to provide a fully adequate coating in extremely corrosive waters, cathodic protection may be considered. There is little evidence, however, that this method of protecting culverts is used to any great extent. Sacrificial cathodic protection, referred to here, virtually involves creating a corrosion cell whereby magnesium or zinc anodes are installed in the electrolyte (soil or water) and connected by metal path to the culvert which then becomes the cathode, or protected electrode, of the system. Since two electrolytes are involved, soil outside the culvert and water inside, then two sets of anodes are required, one set buried in the soil and the other in the invert of the culvert. The latter presents a problem of obstructing flow and the size and shape of anode requires careful design. Cathodic protection is normally only considered in conjunction with a protective coating such as asphalt or coal tar enamel, etc. since the cost of protecting bare steel is likely to be about 10 times as high as that required in conjunction with even a mediocre coating. The better the coating the less the cost of cathodic protection. Owing to the difficulty of predicting the rate at which corrosion will occur in any given circumstance it is probably wise in suspected highly corrosive situations to use the best coating available and then if through the years deterioration of coating or corrosion is observed, cathodic protection may be added.

A relative newcomer to the metal culvert field is aluminum. While it is understood aluminum culverts have been installed, their time in service is insufficient to draw any conclusions as to their over-all performance. Aluminum is resistant to corrosion in many environments, but acid conditions are considered one of the exceptions. However, with the advent of new alloys and surface treatment methods there is reason to believe that improvements are being made. H. P. Goddard (7) reports considerable success with various aluminum alloys buried in a variety of soils, including highly acid soils, for periods of 5 and 10 years. It is pointed out however that his conclusions on expected life are apparently based on the metal being attacked from one side whereas culverts are likely to be attacked from both sides especially along the invert, and consequently the predictions may have to be reduced by 50 per cent. Advances in the aluminum field however seem worth watching as a potential solution to the corrosion of metal culverts.

ii) Concrete Culverts - Concrete is subject to quite severe attack from soluble salts especially sulphates and chlorides. Also the decomposition of organic matter produces carbonic acid which is detrimental to concrete. In fact the same general properties associated with soil water in muskeg areas which contribute to the corrosion of steel also contribute to the deterioration of concrete. If the concrete is very dense and a good air entraining agent is used the rate of attack will be relatively slow. However, most concrete culvert pipe is porous to a limited extent and therefore liable to some deterioration in highly corrosive areas (Figure 2).

iii) Wood Stave Culverts - Wood is subject to attack by fungi and insects. In both cases the wood must be moist (neither saturated nor dry) and a certain amount of air must be present. Such conditions prevail on at least some portions of culverts in virtually all installations. The fungi and insects are however dependent on the wood itself for sustenance, and the wood may be successfully "poisoned" by pressure treatment with creosote. The pressure process causes deep penetration of the creosote into the wood; this prevents the leaching out or wearing off associated with surface treatments, and reduces access to untreated wood through surface cracks. Neither the wood itself nor the creosote is subject to attack from the acid conditions likely to prevail in muskeg areas. The treatment will leach out but only very slowly and the life expectancy of pressure creosote treated wood stave culvert (Figure 3) is likely to exceed that of either metal or concrete in organic soils. A creosote content of 10 lb/cu ft of wood is recommended (8).

It is emphasized that life expectancy referred to in the foregoing applies to corrosion and deterioration of the materials only, and does not refer to failure due to the erosion or structural characteristics of the culverts.

COMPARISON OF CULVERT TYPES IN MUSKEG

The type of culvert selected must be based on the site conditions, desired design life, shipping costs, installation problems, etc. Since each site will differ it is impossible to make specific recommendations to cover all sites. General comparisons, however, can be made to serve as a general guide. Such a comparison is given in Table I.

INSTALLATION TECHNIQUES

Installation techniques will vary depending largely on the method of constructing the road or railway. If the peat is completely removed and replaced, or displaced with granular material, normal culvert installation procedures may be used.

Where preconsolidation is used several methods of installation can be employed. If flow across the grade must be maintained during construction the culvert may be placed temporarily, and allowed to settle with the grade. Following settlement it may then be dug up and placed at the desired grade. If the settlement is great however, it may be more economical to abandon it and place a new culvert. Occasionally it may be beneficial to temporarily divert the channel until the required crossing can be stabilized.

If there is little or no flow across the location it may be advantageous to construct the grade, leave it to settle and then install the culvert later. Field conditions will normally indicate where culverts are required and continued measurement of the settlement will allow a reasonable estimate to be made of future settlement so the culvert can be placed to allow for this.

Where settlement is not expected to exceed 3 to 4 feet the culvert may be placed above grade and with the center cambered so the culvert will settle to a relatively level position. To allow for error it is suggested a culvert size be selected which has about 50 per cent greater capacity than actually required.

Culverts should never be placed on piles. The adjacent fill will settle and may leave a substantial lump in the grade.

It is desirable that granular material be placed on the muskeg initially to a minimum depth of 12 inches and width of three culvert diameters with the culvert bedded in the sand or gravel. The backfill around the culvert should be well compacted. This is especially true with the wood stave pipe. Numerous failures of this type of pipe have been observed by the writer but the majority have occurred as a result of faulty installations.

Concerning alignment, it is suggested that culverts be placed perpendicular to the center line wherever possible. If more or less settlement in relation to the adjacent grade occurs at the culvert, an installation angled across the grade may cause a vehicle to lurch and roll, creating a more critical driving hazard than if the culvert is placed directly across the grade.

In order to provide maximum hydraulic efficiency it is desirable that an effective inlet and outlet be utilized. Special flared metal ends, concrete headwalls, sodding, rip rap or sand bags may find application.

Special precautions should be taken during installation of most culvert types. Where concrete culverts can be used the joint mortar should be made with Type 2 or high alumina cement. Bituminous coated metal pipe owes much of its service life to the asphalt coating. This coating is easily damaged during shipment and installation and must be handled with care. Also close control during the asphalt coating process is necessary since air and vapour bubbles can easily develop. These create small imperfections through which corrosion can start at an early date.

Wood stave culvert must be properly installed for continued success. Proper bedding, site fabrication, band tensioning and backfill compaction are essential.

MAINTENANCE

A general inspection and maintenance program for culverts, particularly those in muskeg is desirable. Not only will such a program provide greater assurance of proper drainage control, but it will also provide valuable data with respect to assessing the success and application of the various types of culverts for varied conditions.

Recently in B.C. a detailed field inspection of culvert installations was carried out. The final analysis is not yet complete but several trends were noticeable in muskeg. Galvanized pipe was subject to very early and severe corrosion where soil resistivity readings were low. Bituminous coated metal culverts were generally performing well but corrosion was developing at most points where imperfections occurred or where the asphalt had been removed from the pipe. Some deterioration of concrete culverts over about 10 years of age was observed but the most damage was associated with failure of the joint mortar. Creosoted wood stave culverts that had been properly installed were generally found to be in excellent condition, even after 15 to 20 years. This suggests that consideration of greater use of this type of culvert in future drainage installations is warranted.

In instances where cathodic protection is considered it is recommended that specialists in this field be consulted since very limited use of this technique has been used at culvert installations.

In the northern muskeg areas icing of culverts and culvert entrances is a frequent problem. Not only do the culverts frequently freeze but the ice may build up in layers once the culvert is plugged and creates nayleds or mounds of layered ice exceeding 3 to 6 feet in height. In some instances these may encroach on or cover the road.

In order to minimize the occurrence of nayleds several procedures may be used. Open trenches or ponding areas above the road for ice accumulation are sometimes effective. In other instances the construction of burlap, tar paper or wood fences along the road tends to limit the extension of ice build-up onto the roadway. When ice build-up reaches the top of the fence the height of the fence can be extended. The Northwest Highway Maintenance Establishment advises that 8 to 10 feet of ice has been built up behind these fences and the roadway remained clear. In other instances the use of two culverts, one higher than the other has proven successful. The bottom culvert carries summer flows with the upper designed to carry the spring run-off. Provided the upper culvert is above the winter ice level this procedure should be effective.

Occasionally fire pots made up of 45-gallon fuel drums or heaters have been employed. When freeze-up is associated with discontinuation of flow, heaters are only required until water flow ceases. If flow continues during the winter continual operation may be necessary. In other instances a bag of calcium chloride placed at the entrance may prove effective in reducing ice build-up.

If the problem is essentially one of de-icing culverts which freeze completely at the entrance several procedures are commonly employed to clear the culverts. Occasionally ½-inch pipes may be left in the culvert over the winter with a steam hose connected to them in the spring when flow starts. The heated pipe melts a hole through which flow can commence. Alternatively electrical heating cable or rod can be placed in the fall and connected to a high amperage direct current welder in the spring. A more recent procedure is to place an inflated expandable tube in the culvert in the fall and in the spring the tube can be deflated and removed to form a passage for water. Once a small hole is opened up the water will melt the remainder of the ice in the culvert quite rapidly.

In some instances it may be desirable to reduce the depth of frost penetration around culvert entrances following freeze-up. This can be done by piling snow over the entrance area to act as an insulator.

Generally it has been found that if freeze-up occurs suddenly followed by snow cover, icing is not nearly as critical as when it occurs slowly or there is little or no snow cover. Preliminary experience also indicates that all types of culverts are subject to icing but metal pipe appears to ice somewhat more readily than wood culverts.

Experience indicates that culvert icing and nayled growth are erratic at most locations. A culvert may present an icing problem one year then not ice up for several years following. This suggests it may be more economical to consider handling each problem as it occurs rather than to establish preventative measures at each culvert.

SUMMARY

Very limited published information is available which specifically deals with culverts in muskeg areas. Consequently an attempt has been made to provide a general assessment of the use of culverts in muskeg.

Numerous factors influence the selection of the type of culvert. They include settlement, lateral movement, flow conditions, location, installation factors, design life, corrosion or deterioration, etc. Each of the above factors is described briefly with specific reference to problems which may occur. Corrosion or deterioration is one of the most significant yet appreciated factors relative to culvert permanency. Therefore more discussion and detail is presented for this problem.

A comparison of various culvert types is included. Each type has application under specific conditions with no one type of culvert recommended for all muskeg areas. Generally, concrete culverts should not be used where settlement will occur or where concrete deterioration may be severe. Galvanized metal culverts are more applicable for temporary routes as they have low corrosive resistance in peat. Cathodic protection may offer economical longer life. Bituminous coated metal pipe will provide long service provided the asphalt coating remains intact. Creosoted wood stave pipe, properly installed, generally provides excellent service and would appear to warrant greater consideration for use than is now given.

Installation procedures are briefly commented upon and general maintenance considerations are discussed. One of the major maintenance problems in the north is culvert icing. Particular attention is focused on this problem.

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TABLE I - GENERAL COMPARISON OF CULVERTS IN MUSKEG AREAS

CULVERT TYPE / SELECTION CONSIDERATION	CONCRETE	GALVANIZED METAL	BITUMINOUS COATED METAL	ALUMINUM	UNTREATED WOOD	CREOSOTED WOOD STAVE
Settlement	Will not tolerate settlement in excess of a few inches. Do not use unless peat is completely excavated and replaced with granular soil	Can withstand differential settlements up to about 2 ft. per 25 ft. length structurally. Settlement in excess of about 1 ft. not desirable because of silting and reduced capacity. Can camber to allow for limited future settlement.	As for galvanized metal.	As for galvanized metal.	Can withstand some differential settlement. Magnitude dependent on type of structure. Can be cambered.	Can withstand moderate differential settlements. Suggest maximum of 1 ft. per 25 ft. length. Can be cambered.
Lateral Movement	Cannot withstand lateral fill or foundation movement unless constructed on reinforced cradle. Normally not recommended.	Experience in B.C. suggests up to about 6 inches of lateral movement in soil will not pull culvert apart. Close control over fill construction required.	As for galvanized metal.	As for galvanized metal.	Resistance to lateral movement depends on type of structure. Movements over about 3 inches undesirable.	Will withstand lateral movement of 6 inches or more, particularly if no major local vertical movement occurs.
Flow Conditions	High capacity. Emphasize entrance & exit design.	Medium capacity. Emphasize entrance & exit design.	As for galvanized metal.	As for galvanized metal.	High capacity. Emphasize entrance & exit design.	High capacity. Emphasize entrance and exit design.

TABLE I -(Cont'd) - GENERAL COMPARISON OF CULVERTS IN MUSKEG AREAS

CULVERT TYPE / SELECTION CONSIDERATIONS	CONCRETE	GALVANIZED METAL	BITUMINOUS COATED METAL	ALUMINUM	UNTREATED WOOD	CREOSOTED WOOD STAVE
Shipment	Shipping costs high Heavy equipment required to move. Subject to damage during shipping, particularly pipe ends.	Shipping costs medium if culverts of variable sizes as they can be nested. High cost if one size. Moderately heavy equipment required to move larger sizes.	Bituminous coating easily damaged. Costs comparable to galvanized metal.	Shipping costs medium for variable sizes. Costs may be less than other metal pipe if freight charges according to weight. Can normally be moved without heavy equipment.	Shipping costs very low. Normally use local material.	Shipping costs variable. Moderately easy to handle and move.
Installation	Installation costs higher than other types of culvert. Require heavy equipment. Must be placed on stable material.	Installation costs generally low. Require heavy equipment for larger sizes. Easy to install.	Installation costs generally low. Require heavy equipment for larger sizes. Moderately easy to install. Must not damage culvert coating.	Installation costs lower than all other types. Very easy to install.	Installation costs may be high depending on labor involved.	Installation costs greater than metal pipe less than concrete. Considerable care required in assembling, tensioning and backfilling. Most failures attributed to faulty installation
Materials Cost (Valid in Vancouver)	Least expensive, generally for small sizes. More expensive than galvanized or wood in larger sizes(over 48")	Nearly comparable to wood stave in smaller sizes.	About 20-35% more expensive than galvanized culvert. More expensive than others.	Generally comparable to bituminous coated pipe.	Least expensive generally Use local material.	Generally compares with galvanized metal pipe for small sizes. Least expensive over 48 inches.

TABLE I -(Cont'd)- GENERAL COMPARISON OF CULVERTS IN MUSKEG AREAS

SELECTION CONSIDERATION / CULVERT TYPE	CONCRETE	GALVANIZED METAL	BITUMINOUS COATED METAL	ALUMINUM	UNTREATED WOOD	CREOSOTED WOOD STAVE
Life Expectancy	Variable - Generally 10 to 30 years depending on rate of deterioration. Rate of attack can be estimated from pH and soil resistivity measurements.	Variable - Generally 5-10 years depending on corrosion conditions. Rate of corrosion can be estimated from pH and soil resistivity measurements. Cathodic protection may be economical for large culverts	Depends largely on integrity of asphalt coating. With a good coating life of 10 to 20 years should result. Cathodic protection may be economical.	Largely unknown but expected to be better than galvanized metal pipe. Requires further field evaluation.	Above water will deteriorate rapidly. Useful life about 5-10 years. Completely submerged will last indefinitely.	Expected life of at least 30 years if properly treated and installed. Some installations in very good condition after 20 years in B.C.
Corrosion or Deterioration	Subject to acid and sulphate attack. Rate of attack variable; specify high density concrete, air entrainment.	Subject to corrosion. Rate of attack variable but often rapid. Use of cathodic protection beneficial but not always economical.	Corrosion reduced by asphalt coating with rate of deterioration dependent on conditions of coating. Can consider cathodic protection.	Ability to withstand corrosion is generally unknown. Requires further evaluation. Field test installations suggested.	Deteriorates rapidly above water.	Very resistant to deterioration.

Figure 1

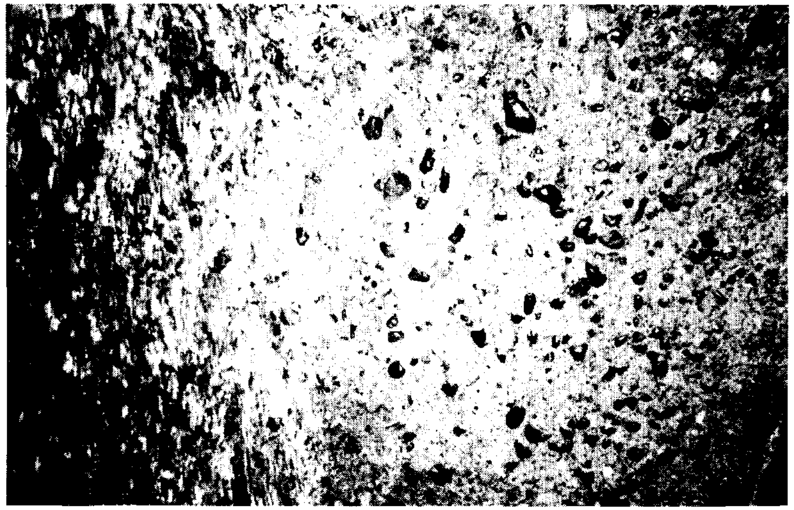

Figure 2

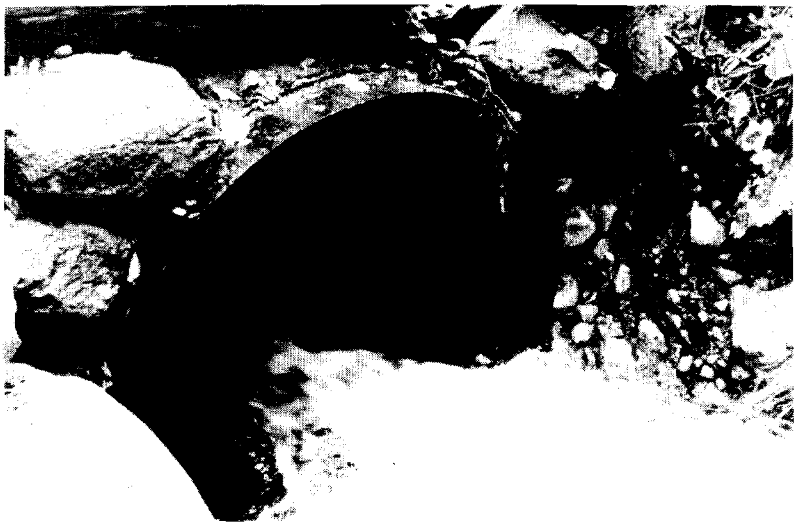

Figure 3

Figure 1 - Severely corroded galvanized metal culvert pipe after 5 years.

Figure 2 - Concrete culvert showing considerable surface deterioration after 8 years

Figure 3 - Creosoted wood stave culvert through granular fill after 10 years

DISCUSSION

Mr. Hughes commented that in the north muskeg waters may not be as corrosive as further south and corrosion may not take place as rapidly. He said, however, that in one stream at Fort St. John, the water has a pH of 5.5 and wondered if this was not rather bad. Mr. Brawner said that water with a pH of 5.5 is often very corrosive, but pH is not the only criterion for corrosivity. Other factors must be considered as well.

Mr. Savage suggested that the fact of not too much corrosion in the north may be due to the culverts having a flow for only a brief period in the summer. Mr. Hastings remarked that pH values of 3.4 and 3.5 are a common thing in sphagnum bogs in New Hampshire. Dr. Radforth pointed out that there is a pH gradient going from Fort Churchill south about 20 miles. It starts off very alkaline at Churchill and ends up very acidic to the south. There are muskegs which have developed almost entirely from brackish or saline water conditions. Dr. Radforth emphasized that, consequently, not only is it important to specify the nature of the peat, but also the pH of the peat at that particular latitude. Mr. Harwood commented that if corrosion is an electrolytic action surely acidity and alkalinity are relatively unimportant. Mr. Bergan wondered if anyone had tried fibreglass culverts. Mr. Brawner said that their use was investigated, but they were found to be extremely expensive.

Mr. Connelly, referring to de-icing of culverts, mentioned that this was often accomplished by the use of steam jets. Mr. Hansen remarked that in his experience, in an attempt to maintain stability of culverts from differential settlements, a pad was used on which was placed a granular layer of fill then the culvert.

III.2

CORROSION IN MUSKEG

by

G. Mainland

The ever increasing cost of replacing equipment which has been damaged by corrosion has led to an intensified interest in recent years in means of protection. It is only natural, therefore, that in Canada concern should be felt with the effect of muskeg on any structures which may be built. Unfortunately there is very little information on the severity of corrosion in muskeg, and no numerical test data are available which will improve the situation. The rather meagre information presently available does indicate, however, that the type and rate of attack follows a predictable pattern, considering the environment. A discussion of the mechanism of attack should therefore provide some aid in determining how much protection should be afforded in any particular case. Complete protection can always be provided, but as a Corrosion Engineer, the author is now considering only a level of protection which will show economic gain.

Prediction of corrosion behaviour in any particular environment requires a study of all factors which may have any influence, so reference should be made to the basic principles of corrosion in soil.

CORROSION PRINCIPLES

First of all, the process is largely an electro-chemical one. The three essential requirements for corrosion to occur are an electrically conducting structure or metal, an electrolyte and some mechanism whereby potential differences are self-generated on the metal.

The potentials are not difficult to achieve, since most metals immersed in an electrolyte will have a surface composed of many small areas of varying potential, due to its structure or treatment. At the anodic or corroding sites, metal ions will go into solution in the electrolyte, while electrons are liberated and travel to the cathodic sites. This mechanism may be written as $Fe \longrightarrow Fe^{++} + 2e^-$. At the cathode two main reactions normally take place. Positively charged hydrogen ions in the electrolyte migrate to the cathode where they accept a negatively charged electron and form a layer of hydrogen gas at the surface which is gradually evolved. $2H^+ + 2e^- \longrightarrow 2H \longrightarrow H_2\uparrow$. In the presence of oxygen the removal of the hydrogen is more rapid, as water is formed. $4H^+ + O_2 + 4e^- \longrightarrow 2H_2O$.

The rate at which all this takes place depends on the magnitude of the potential differences between the anode and the cathode and the internal resistance of the cell. The resistance initially is controlled by the resistivity of the electrolyte, but when corrosion has proceeded for a while other effects generally known as polarization begin to influence the rate. At the anode the increasing concentration of anodic products - such as Fe^{++} - gradually reduces the tendency for further action, while at the cathode the hydrogen layer has the same effect. There are several other polarization mechanisms. The removal of hydrogen from the system results in an increase in hydroxyl ions (OH) or alkalinity at the cathode, and this in turn may induce precipitation of a scale film on the cathode. In soils with rather low moisture content - not peat - electro-osmosis will tend to drive the electrolyte away from the anode and increase the circuit resistance.

It appears, therefore, that given time a corrosion cell will normally tend to stifle itself, and this is frequently true. However, the potentials and other conditions may be such that the damage rate is not economically tolerable, and in practice these polarization mechanisms may not be permitted to continue. For instance, in rapidly running water the hydrogen layer will be removed, and the electrolyte will be continuously replenished so that concentration polarization will not occur. If oxygen is present it has already been noted that the hydrogen was also fairly rapidly removed.

PREDICTION OF CORROSION IN MUSKEG

In studying the possible corrosion in muskeg, therefore, it is desirable to know:

1. Is the electrolyte of low resistivity?
2. Will there be large potential differences on the structure?
3. What polarization or depolarization mechanism can be expected?

In considering soil corrosion a common indicator of potential corrosivity is the soil resistivity, and variations of the following table are commonly employed:

0 - 1,000	ohm cm	- severely corrosive
1,000 - 10,000	ohm cm	- moderate to severe
10,000 - 100,000	ohm cm	- very moderate
100,000 and over	ohm cm	- non-corrosive

The resistivity depends on the moisture content of the soil and the dissolved salts in the moisture with the general relationships as shown in Figures 1 and 2.

Many soils in Western Canada have high salt contents and when moist exhibit resistivities of 500 ohm cm or less. In muskeg while there is a great deal of moisture the salt content is usually relatively low and generally the resistivity will be over 1000 ohm cm. The wetness of muskeg can be deceptive.

Potential differences on a structure, a pipeline for example, are of two types: those which give rise to micro cells all over the surface; and those much larger differences which result in one section of pipe being an anode and another a cathode. A varying environment or electrolyte has a strong tendency to create macro-cells since a potential difference is set up between portions of the metal immersed in different solutions. The variation in soil resistivity is therefore important. More damage can be expected if the soil varies successively from 1,000 ohm cm to 10,000 ohm cm than if the soil had a uniform resistivity of 1,000 ohm cm. This is partly because of the pitting type of attack which one might expect under these circumstances. Unfortunately soils frequently tend to generate pitting corrosion rather than a uniform metal loss. On such structures as pipelines this is very undesirable. On others it might not be so serious.

In large areas of muskeg the resistivity is not expected to vary very much, and the effect of the fairly low value can therefore be somewhat de-rated. However, if a pipeline is laid in an area where it runs through, for example, sand or clay hills and into muskeg then very severe attack can be expected in the muskeg. This is due to a combination of the variation in resistivities and the difference in oxygen concentration in the ground. Variations in oxygen concentration in the environment around a structure are one of the commonest causes of cell formation, and the area of lowest oxygen concentration becomes the anode. This accounts for frequent pitting found underneath a pipeline, where the environment is comparatively oxygen lean compared to the top.

The oxygen of course acts as a cathode depolarizer, increasing the activity of the corrosion cell, so that in many peats, which can be virtually oxygen free, polarization of the cathode should result in a lowering of corrosion rates. Unfortunately, another depolarizer may arrive on the scene in the form of several types of bacteria, the best known being sulphate reducing bacteria. These are anaerobic, thriving in the absence of oxygen and in environments which are fairly neutral or slightly acid, say pH 6-9. They are believed to have two functions, both of which result in depolarization of the corrosion cell. They utilize the cathodic hydrogen to reduce sulphates, depolarizing the cathode. The sulphide they form reacts with the dissolved iron near the anode to form iron sulphide and by reduction of the dissolved iron concentration depolarizes the anode. In general, therefore, they are undesirable. They can be detected by the presence of iron sulphide corrosion products, by culture of a water sample or by the use of a rather delicate instrument known as a 'Redox' probe with which the reduction - oxidation potentials in the soil can be measured. These bacteria are believed to be very commonly found in muskeg areas, and will assist in promoting pitting when varying resistivity ground is encountered, or uniform corrosion in large undisturbed muskeg environments.

Summarizing these effects on the rate of corrosion in muskeg it can therefore be predicted that:

1) If a structure is entirely immersed in a large body of muskeg, corrosion should be fairly slow and will be of a uniform nature.
2) If it is only partially immersed then some corrosion can be expected due to differential aeration cells.

3) If sulphate reducing or other types of depolarizing bacteria are present in the muskeg the corrosion rate in the metal will be appreciably increased.
4) If the structure is a long one, such as a pipeline, and is buried in muskeg and in adjacent inorganic soil then very serious corrosion can normally be anticipated near the edge of the muskeg.

PROTECTIVE MEASURES

Providing protection for metal installed in muskeg should present no particular problems. The two normal answers to underground corrosion: coating and/or cathodic protection can be applied.

There are a few points worth watching, however. One is in the manner of coating. Coating a cathodic area alone will frequently reduce the corrosion cell current, even if the coating is not defect free, and is a safe protective measure. However, on no account should a potential anodic area be the only portion of a structure to be coated, since the cell current is then concentrated at any small defects and very rapid, deep pitting can be expected. The practice of applying a coating to a pipeline in the areas where it passes through muskeg, and installing bare that portion of it which is in drier ground, is thus to be rigorously avoided, unless adequate supplementary cathodic protection is to be provided.

Coatings should generally be selected on the basis of their water resistance. Normal pipeline enamels and plastic tapes are satisfactory and coal tar epoxy paints should also be suitable. Some work has been done in recent years which has demonstrated that sulphate reducing bacteria are not allergic to, and are even rather partial to, some of the pipeline coatings, so in the presence of bacteria this is worth watching.

The normal criterion for the cathodic protection of buried steel is that the metal to soil potential should be maintained at a value of -.85 volts with respect to a copper-copper sulphate electrode. The output of the cathodic protection system is adjusted so that the entire structure will achieve this potential or be even more negative. Once again, the presence of sulphate reducing bacteria will affect the protection criterion and if they are present the potential should be increased to at least -.95 volts.

In designing the system protective current densities on the metal should also be higher if bacteria are present and about 6 - 7 ma/ft^2 on bare steel is probably required. In the absence of bacteria 2 - 3 ma/ft^2 should suffice. A coating will of course greatly decrease these requirements to a degree dependent on its quality. A good coating will reduce the current requirement by a factor of about 100.

Fortunately, in muskeg there should be little problem with current distribution, and the constant moisture is of assistance in maintaining a low resistance anode groundbed.

In summary:

1. Partial protection can be afforded by coating, provided that it is applied to the entire structure.
2. Cathodic protection alone can be employed, with the possibility that fairly high current densities will be required.
3. A combination of coating and cathodic protection will provide the optimum protection.

CONCLUSION

It is evident that in this discussion of corrosion in muskeg a large number of postulations have been made. This is due to the dearth of information and experience on the subject. Most pipelines installed in muskeg have been given full protection, since a long service life is required. The actual corrosivity of the ground has not been extensively determined.

Corrosion Engineers need to know considerably more about the environment. Such details as the resistivities, composition of the ground water, and the presence or absence of bacteria do not appear to be well documented. The recording of such information would be of assistance in designing optimum protective systems for structures.

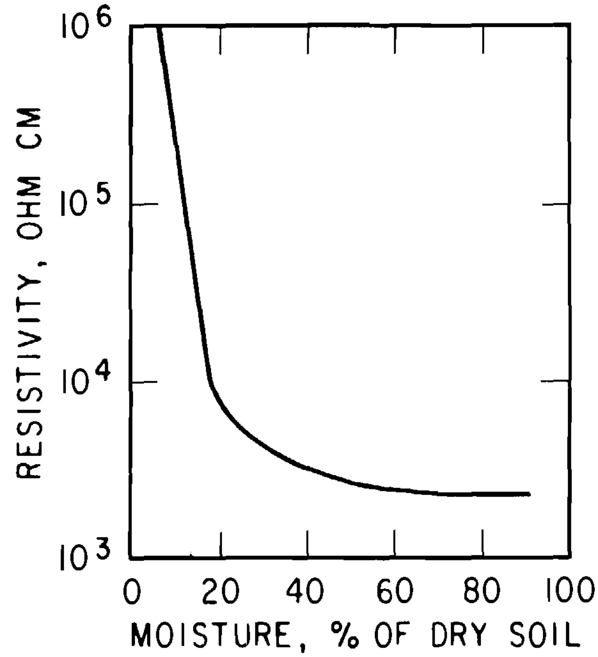

FIGURE 1

RELATIONSHIP OF SOIL RESISTIVITY
TO MOISTURE CONTENT

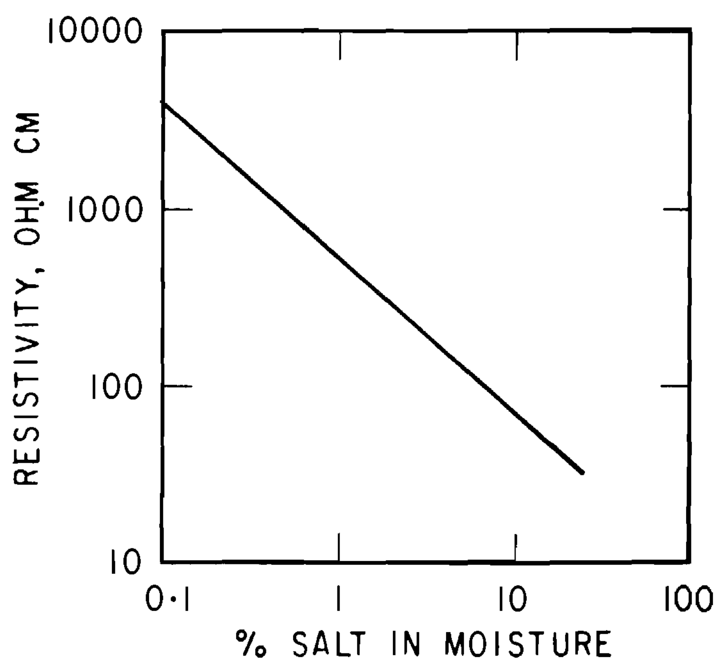

FIGURE 2

RELATIONSHIP OF SOIL RESISTIVITY
TO DISSOLVED SALTS

BR 2753

DISCUSSION

Mr. Hughes asked Mr. Mainland if he would normally expect that when a pipeline passes from a muskeg area to mineral soil area, the muskeg section is the cathode. Mr. Mainland replied that the muskeg section of the pipe is the anode and pitting will occur just before the line leaves the muskeg.

III. 3. LABORATORY AND FIELD DATA ON ENGINEERING CHARACTERISTICS OF SOME PEAT SOILS

by

Louis J. Goodman and Charles N. Lee *

SUMMARY

This paper describes a detailed field and laboratory program followed to determine the feasibility of preloading a site underlain with peat to permit the use of footing foundations and a floor slab on grade for a truck garage in Syracuse, New York. Engineering properties of this particular peat are discussed and compared with the field results of the preloading operation. Laboratory data on engineering characteristics of a peat from a site in Western New York are used as a basis for further discussion.

INTRODUCTION

When a building or an embankment is to be constructed over very compressible soil, such as peat, it is common practice to employ one of the following procedures to insure stability of the structure and to prevent intolerable differential settlements:

1. By-pass the peat by the use of piles
2. Remove the peat and replace with selected materials.

The above methods are widely used today and will continue to be used, particularly for heavy buildings and first class roads where settlements must be kept to a minimum. However, they are often expensive and many projects proposed for sites found to be underlain with peat have been abandoned because of the foundation costs involved. It is therefore apparent that some other type of foundation treatment such as site preloading might be desirable for projects where some settlement can be tolerated.

Unfortunately, very little work has been done in the past in studying the engineering properties of peat which must be considered in determining the feasibility of site preloading. MacFarlane (1) studied data from a number of references dealing with various properties of peat, reporting that many gaps exist in our knowledge of the engineering behaviour of this material.

This paper describes in detail the soil engineering investigation for the design and control of site preloading for a structure located on a site underlain with peat, including the results of the preloading operation. The engineering properties of this particular type of peat and peat from another site are presented and discussed. It is also the intent of this paper to stimulate more interest in furthering our knowledge of the behaviour of peaty soils under load.

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PRELIMINARY CONSIDERATIONS

Before discussing the results of the soil engineering investigation, some attention will be devoted to the general character of peat and a description of the soil conditions encountered at the site in question. This section will also include a description of the proposed facility and a discussion of the foundation possibilities.

General Character of Peat

Peat represents the first major stage in the decay process of vegetable matter that accumulated in the swamplands of the earth many years ago. It is generally defined by engineers as a fibrous, partially decomposed organic material. Taylor (2) very aptly describes peat as a "partially carbonized vegetable matter which has low shearing strength, is often permeable, is always extremely compressible, and is the poorest foundation material imaginable." It is easily recognized by its dark color, its fibrous nature, and its odor of decay.

The development of peat is a progressive process with the successive decomposition of parts of the plant matter. The plant fiber may disappear leaving only organic silt, or the organic matter may decompose into a gelatinous form (amorphous). Peatsoil mixtures are also quite common, resulting from peat being flooded with water containing soil in suspension. Thus peat may fall into one of several categories, depending upon the character of vegetation, decomposition factors, and place of formation.

It is apparent that there are wide varieties of peat material and therefore wide variations in degree and type of decomposition, amount of foreign mineral matter and water-holding capacity. MacFarlane (1) studied many references on the subject of peat and did a thorough job of assembling the material and presenting a summary of engineering properties of most peat types from the very fibrous to the highly decomposed or amorphous type. As expected, organic content, water-holding capacity, void ratio and deformation characteristics were found to be high while density and strength characteristics were low in all the references he studied. Unfortunately, as noted by MacFarlane, many of the peat samples discussed in the references studied were neither classified nor described.

Description of Site Investigated

The site discussed in this paper lies in a lowland area southeast of Onondaga Lake in the City of Syracuse, New York. It is located over sediments of considerable thickness which have been deposited in an eroded bedrock trough created by glacial action in the geologic past. Depths of more than 70 ft have been drilled in this particular vicinity without passing through the overburden; in fact, the deepest part of the trough contains overburden up to 400 ft in thickness.

The deeper deposits consist of glacial till deposited by the retreating glaciers and lake sediments which were deposited in the longstanding glacial Lake Iroquois. The upper sediments consist of loose sands and silts laid down in shallow waters as the glacial lake gradually subsided. Organic material represented by peat then accumulated in the bogs and marshes that remained. This particular peat contains both amorphous organic silt particles and some residual plant fibers remaining in a less decomposed state. Later deposition above the peat represents the work of man and consists primarily of recent granular fill.

The specific sequence of materials is shown in Fig. 1.

Description of Facility

It was planned to use an Armco Metal Building Type S-3 (40 x 60 ft) with a floor slab independently supported on grade. A maximum loading of 1000 lbs per sq ft was expected from this facility.

Foundation Possibilities

Three possibilities for providing adequate foundations for the proposed facility were considered. These foundation treatments were:

1 - End Bearing Piles

The majority of the fuel storage tanks and other structures in this area are supported on either short piles driven into a zone of medium silty sand and gravel, or long piles driven into the underlying glacial till. Representative lengths of the short piles vary from 25 to 35 feet. The long piles are used to by-pass the lake silts and clays which predominate in a depth range of 50± to 100± feet in this area and generally range from 90 to 110 feet in length.

Piles were not investigated in detail, however, since the Atlantic Refining Company decided to abandon the project if footing foundations and a floor slab on grade were not feasible.

2 - Removal of Peat

If the depth of compressible material is not excessive, it is frequently economical to remove the soil and replace it with properly compacted granular fill. This possibility was explored in some detail but was found to be fairly costly in view of the need to drive sheet piling and dewater because of the confined area and the subsoil conditions.

3 - Site Preloading

Preloading consists of applying a dead load or surcharge over the site, equal to or greater than the weight of the proposed structure, to develop the settlements prior to construction. After compression of the underlying soil has occurred under the preload, which is usually an earth fill, the preload is removed and replaced by the structure itself, using shallow foundations to support the loads.

When the preload is removed, the compressible stratum may expand, but generally only a small percentage of the compression resulting from the preload. Finally, when the building load is applied, recompression will occur but the amount should be small, usually only slightly larger than the expansion. An economical side advantage of this method is that the surcharge material can be utilized for fill purposes such as parking areas and driveways, if necessary.

A soil engineering investigation is necessary for the design and control of site preloading. These studies should include a determination of the general order of magnitude and rate of settlement under building and surcharge loads to define the time needed to develop settlements from the preloading operation. Also, monitoring of the settlements is necessary during the preloading stages to provide control on the actual progression of settlements.

This method can result in substantial savings in foundation costs but has not had much use in the United States, primarily due to the fact that any construction method which involves long delays is not popular in this country. Preloading, however, has been applied successfully to many projects including industrial buildings (3) and oil storage tanks (4) (5).

Field experience on peats in the Syracuse area by the writers indicated that a preloading operation was feasible for this project and should be investigated in detail. An additional consideration involved in this decision was the fact that the peat was in a remarkable state of preservation because of its existence below the water table. This decision was also reinforced by the fact that nearly one-half of the earth fill needed for the recommended 50 per cent overload was available on the site, which would result in additional economy.

PRELOADING PROGRAM

This section is devoted to the laboratory testing program, settlement studies and the site preloading program that were followed in this project.

Laboratory Testing

An estimate of the action of the site under loading was made by subjecting undisturbed samples of the critical peat layer to laboratory shear and consolidation tests. These undisturbed samples were obtained by standard methods using 5-inch Shelby tubes.

Past experience concerning testing procedures on samples of peat indicates the need to make some modifications to standard ASTM procedures in some cases.

Specific gravity determinations on fibrous organic material are subject to many errors. Loss of solid weight is one such error and occurs by charring of the organic portion of the sample while drying at relatively high temperatures. Determination of an accurate volume of the solid portion of the sample is another source of error and is caused by swelling and air retention when the material is immersed in water. Prevention of charring was accomplished by drying at a temperature of 85°C rather than at the ASTM standard of 105°C. Volume measurements were performed with a standard 500-ml volumetric flask using extra care in evacuating air from the soil sample in the flask.

Shear strength of the peat was determined from standard unconfined compression tests on specimens with a diameter of 3 in. and a length of 6 in. Several tests were found necessary before a run could be made on a sample free of internal flaws (pieces of wood or pebbles).

Relatively long-time strain characteristics were determined by means of consolidation tests performed on specimens with a diameter of $4\frac{1}{4}$ in. and a thickness of $1\frac{1}{4}$ in. Load increments were started at 1/8 ton per sq ft and doubled in succession until a maximum increment of 4 tons per sq ft was applied. Each load increment was allowed to remain in place until a minimum of 2 to 3 points on the secondary compression portion of the time curves were obtained. This occurred after a time interval of 24-48 hours for each increment. Again, more than one test was necessary to eliminate the effects of internal flaws. The laboratory time curve for the 1000 lb per sq ft increment from a consolidation test is shown on semi-logarithm paper as Fig. 2. In Fig. 3 compression and rebound characteristics of specimens free of foreign matter are shown in the usual manner as a function of the void ratio plotted against the log of the load increment.

The laboratory testing program included unconfined compression tests on undisturbed specimens of the peat which were preconsolidated under the expected maximum building load of 1000 lb per sq ft for 96 hours. This was done to afford an indication of the amount of increase in the shearing strength of the peat from the preload. Determinations of the organic content and pH of the peat were also made. Organic content was found by placing specimens initially oven-dried to a constant weight at 85°C in a kiln at 600°C for several hours.

The results of the laboratory tests are summarized in Table I.

Settlement Studies

An analysis of the laboratory data was made using the one-dimensional consolidation theory and time factors of Terzaghi to predict the time required to reach expected settlements under the super-imposed preload. Computations for stresses due to the building loads were made on the basis of the Boussinesq equations for stress distribution. The results of this analysis are shown in the time-settlement curves of Fig. 4.

Site Preloading

Theoretical analyses indicated a maximum settlement of 10.3 in. of primary consolidation from compression of the peat layer, due to a load of 1000 lb per sq ft applied at ground surface. Time studies indicated that 90 per cent of this settlement should occur in 7 months. The desired construction schedule allowed for a maximum delay of 3 to 4 months, so further analyses were made with a surcharge loading of 1500 lb per sq ft. Results of the study with the latter loading showed that $9\frac{1}{4}$ in. (90 per cent of maximum settlement expected from building loads) should occur at 5-1/3 months. However, experience of the writers and others (1) with past projects on peat, indicated that the actual time interval might be less than that predicted. Therefore, preload operations were commenced using a surcharge of 1500 lb per sq ft, anticipating a maximum loading time of 3± months to develop the expected settlements.

Surcharge loading was applied by placing a 12-ft embankment of general fill. The embankment material was primarily granular in nature with an average unit weight of 120 to 125 pounds per cubic foot in the in-place condition. It was end-dumped and spread by a bulldozer in essentially two 6-ft stages approximately 3 weeks apart. The plan area of the top of the surcharge was 42 by 62 ft to ensure adequate coverage of the 40- by 60-ft building area.

Provisions for monitoring the settlement under this load were provided by 5 settlement plates located as shown in Fig. 5a. Prior to placing these plates, all surface ash fill was removed from the site and the area dressed up for receiving the surcharge. The plates were then located and filling commenced. Details of the plate monitoring system are shown in Fig. 5b. Vertical movements of the plates were observed at regular intervals and recorded as shown in Fig. 6 to evaluate actual time-settlement conditions. Fig. 6 also includes rebound data during and after the removal of the preload. It can be noted that the actual settlement curves leveled off in a range varying from 5 to 11 in. after 3 months of loading in contrast to the theoretical predictions of $9\frac{1}{4}$ in. in 5-1/3 months. This range of field recorded settlements is explained in part by the fact that the front of the site (covered by plates 1 and 2) has a past history of some preloading. In view of the urgency of the construction schedule and the shape of the field settlement curves, it was decided to remove the preload 2 months ahead of the theoretical estimate.

CONSTRUCTION RECORDS

Arrangements were made with the Atlantic Refining Company to make settlement observations both during and after construction. This was accomplished by placing settlement pins in both the columns and in the floor.

As of this writing, no settlement due to a compression of the peat has been noted. Construction was started in November 1959. Maximum foundation loads have been in effect for over 2-1/3 years as of this date (April 1962).

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DISCUSSION

There are essentially two factors that must be considered in a study made for site preloading purposes - settlement magnitudes and time considerations. It is always of interest to compare the theoretical analyses against actual results on any such study for purposes of corroboration or revision of theory as necessary. Since little work of this nature has been done involving sites underlain with peat, the authors feel that if this paper adds to the meager information of the past and creates further interest in the subject, they will have served a worthwhile purpose.

Comparison between the one-dimensional consolidation theory of Terzaghi and actual field records on the project discussed herein show results that have been experienced with much of the work on peats in other areas. Much of the data available on peat (1) indicate that the classical consolidation theory predicts the magnitude of expected settlements rather well, but that the time considerations between theory and practice do not agree. Such was also the case in this study. Reference to Fig. 6 shows that the average field settlement value was approximately 8 in. with a range varying from 4.9 to 10.9 in. while the theoretical prediction was $9\frac{1}{4}$ in. (a deviation of + 16 per cent based on average field settlement). However, when considering field settlement values from plates 3, 4 and 5 (area not involved in any past preloading) the average field settlement agrees quite well with the estimated settlement. Time predictions, however, were not nearly as valid. Theoretical analysis indicated 90 per cent consolidation with a loading of 1500 lb per sq ft at 5-1/3 months whereas field records showed an equilibrium point at approximately 3 months.

Some of the authors discussed by MacFarlane (1) indicate a major settlement from long-term secondary consolidation while others feel that the secondary effects are minor. Although the secondary portion of the consolidation curves (see Fig. 2) indicated that secondary compression effects might be on the high side, the authors felt that relatively minor effects would result from secondary compression, based on previous field experience with similar peaty soils in the Syracuse area. Since the observation appears to be valid, it would seem that a proper classification as to the type of peat is imperative for any settlement considerations of this nature.

It is also interesting to note the comparison of rebound and recompression between laboratory and field data. The rebound estimated from Fig. 3, curve "a", showing the e vs. log p curve (before preload) amounted to 25 per cent of the total void ratio change occurring under a load of 8000 lb per sq ft while the rebound from the field settlements shown in Fig. 6 amounted to approximately 10 - 12 per cent of the total settlement recorded in the area not involved in any past preloading. Projection of laboratory rebound on the basis of the field loading of 1500 lb per sq ft indicates a change of void ratio on rebound of approximately 50 per cent of that resulting from compression. It is noted here that the stress increase at the midpoint of the peat layer from a surface loading is actually slightly under the value of the surface preloading of 1500 lb per sq ft.

A second consolidation test was run on an undisturbed sample of the peat obtained from the central portion of the preload area after removal of the surcharge. The e vs. log p curve from this sample is also shown in Fig. 3 as curve "b". Reference to curve "a" of this figure shows that a void ratio change of 0.86 should be expected from a stress increment of 1500 lb per sq ft. Reference to curve "b" of the figure indicates that a void ratio change of 0.80 might be expected when applying a stress increment of 1000 lb per sq ft (from the building loads) to the pre-compressed peat. This would indicate that a considerable amount of recompression should be expected from the building loads. Observations at this writing are not in accord with this speculation, however, since no recompression has taken place under construction and building loading to date. It is felt that the foregoing can be explained in part by the fact that the partial stress relief which occurs when the preload is removed causes little disturbance to the structure of the peat in contrast to the relatively severe disturbance from the sampling operations. Settlements are to be periodically checked in the future to observe the effects, if any, of long term recompression.

The laboratory test results on natural water content, specific gravity of solids, void ratio, dry unit weight and organic content agree fairly well with the past data compiled on peaty soils in the Syracuse area. The relatively low organic content of Shelby sample No. 1 is explained by the fact that the portion of this specimen used for this determination was oven-dried at 105°C for a long time period, resulting in some charring. Oven drying of organic soils at lower temperatures such as 85°C appears to be desirable, based on the experience of the writers.

No particular difficulty was experienced in performing the consolidation and unconfined compression tests on the undisturbed specimens of peat. The only source of trouble noted was due to the occurrence of small pieces of wood and some isolated pebbles, resulting in the need to perform additional tests. It is felt that larger diameter samples of peaty soils should be used for strength and deformation determinations to obtain more representative values in view of the heterogeneous nature of many peaty soils.

ENGINEERING PROPERTIES OF A PEAT FROM EAST AURORA, N. Y.

The authors have recently completed a similar detailed subsurface investigation for the proposed construction of a warehouse on a site underlain with peat in East Aurora, New York. The warehouse is a one story building covering an area of approximately 120 by 262 ft, with maximum column loads of 60 kips and a contemplated floor loading of 500± lb per sq ft. The floor will consist of a slab independently supported on grade and will be some 4 ft above existing grades, which will involve the emplacement of compacted granular fill.

Typical profiles of the materials encountered on this site are shown in Fig. 7, along with a plan showing the location of the test borings. Undisturbed samples of the peat were obtained from both above and below the water table by means of 5-in. Shelby tubes. The submerged peat still retains a fibrous texture in some constituents and wood, though decayed, is recognizable. The peat from above the water table has largely decomposed to an organic silt in which little of the fibrous structure remains. Recent growth of shrubs and trees over the swamp has added late-generation roots and probably some fallen logs to the upper part of the peat.

The laboratory testing program followed for this project is similar to the one reported for the Syracuse site, but unfortunately no field results are available at this writing. The results of the laboratory tests on both undisturbed and spoon samples of the peat are summarized in Table II. Figure 8 shows the results of consolidation tests on the samples of peat from above and below the water table. Each load increment was allowed to act until the secondary compression portion of the time curve was well defined, taking as long as 48+ hours in some cases. The laboratory time curves for the 1000 lb per sq ft increment for the above samples are shown in Fig. 9.

Time-settlement curves for three representative stress points within the eastern half of the building area are shown in Fig. 10. These curves show the predicted rate and amount of estimated settlements due to primary consolidation for loadings due to both the emplaced fill and the fill plus a surcharge of 1000 lb per sq ft. The Boussinesq Equations for uniformly loaded areas were used to compute the stresses in the peat layer from the imposed loads and, as in the case for the Syracuse peat, the one dimensional theory and coefficients proposed by Terzaghi were used for the analysis.

CONCLUSIONS

The major conclusion made from these investigations is that more data on the engineering properties of peat and other highly organic soils are needed to afford a better understanding of these materials under load. For example, it has been generally agreed in the past that the secondary time effect of organic soils is so great that estimates of the time-rate of compression cannot be made by the classical theory of consolidation, and a danger exists that long term settlement may prove detrimental to any structures located over such deposits. Yet, the experience of the authors and others (1) indicates that secondary time effects of certain types of peat may not be as great as expected. The authors have had field experience with other structures built over as much as 8 to 10 ft of peat occurring at shallow depths where most of the settlement was noted to take place shortly after construction.

In view of the above, it appears that site preloading should prove to be practical and economical in controlling foundation settlements both overall and differential, for many projects being considered for sites underlain with peat.

However, it is thought that a critical evaluation of the effects of sample disturbance from both "undisturbed sampling in thin wall samplers and from subsequent sample removal on the strength and deformation characteristics of compressible soils must be made.

Other areas requiring more data are those concerned with the amount and rate of decomposition of peaty soils subjected to alternate wetting and drying, and the very important consideration of classification type and origin, since this will have much influence on the interpretation of some of the laboratory data.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the splendid cooperation from Mr. Dale E. Thompson, Regional Engineer for the Atlantic Refining Company in Syracuse, N. Y. during the entire investigation.

Recognition is also given to Empire Soils Investigations, Inc. of Groton, New York for their cooperation in the site investigations. This included the securing of additional undisturbed samples beyond the scope of the project and making additional consolidation and shear tests. The geologic history of the Syracuse site was obtained from Dr. Earl T. Apfel, formerly Chairman of the Department of Geology at Syracuse University.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS
SYRACUSE, NEW YORK PEAT

Sample No.	Dry Unit Weight pcf	Natural Water Content %	Initial Void Ratio	Compression Index C_c	Unconfined Comp. Strength tsf	Specific Gravity G_s	Organic Content %	pH
Shelby 1a	22.7	345	-	-	0.12	1.62	42.5	4.7
Shelby 1b	14.7	403 (221)*	5.48	2.84	-	1.51	-	-
Shelby 2a	25.0	282	-	-	0.15	-	74.6	5.9
Shelby 2b	17.4**	(318)*	-	-	0.22**	-	-	-
Shelby 3***	15.7	319 (286)*	5.23	2.58	-	1.57	75.2	8.6

*(221) = water content after consolidation

** This specimen preconsolidated under 1000 lbs. per sq. ft. for 96 hours prior to testing

*** Obtained after surcharge removed

TABLE II

SUMMARY OF LABORATORY TEST RESULTS
EAST AURORA, NEW YORK PEAT

Hole No. & Sample	Depth (ft)	Dry Unit Weight (lbs/cu.ft.)	Natural Water Content %	Initial Void Ratio	Compression Index C_c	Unconfined Comp. Str. (Tons/sq.ft.)	Specific Gravity G_s	Organic Content (%)	pH
B-12 Shelby #1	6 to 7-1/2	16.3	304.0(96)**	4.85	2.30*	-	1.53	42	-
B-12* Shelby #2	10 to 11-1/2	14.7	340.0(165)**	5.50	2.15*	-	1.53	44	-
B-9* 3	10 to 11-1/2	-	577.3	-	-	-	-	75	7.1
B-3 2	5 to 6-1/2	-	242.2	-	-	-	-	65	6.1
B-11 Shelby #1	6 to 7-1/2	-	298.0	-	-	0.22	-	-	-
B-11* Shelby #2	10 to 11-1/2	-	352.0	-	-	0.17	-	-	-

*Samples taken from below water table
Remaining samples from above water table

**(165) = water content after consolidation

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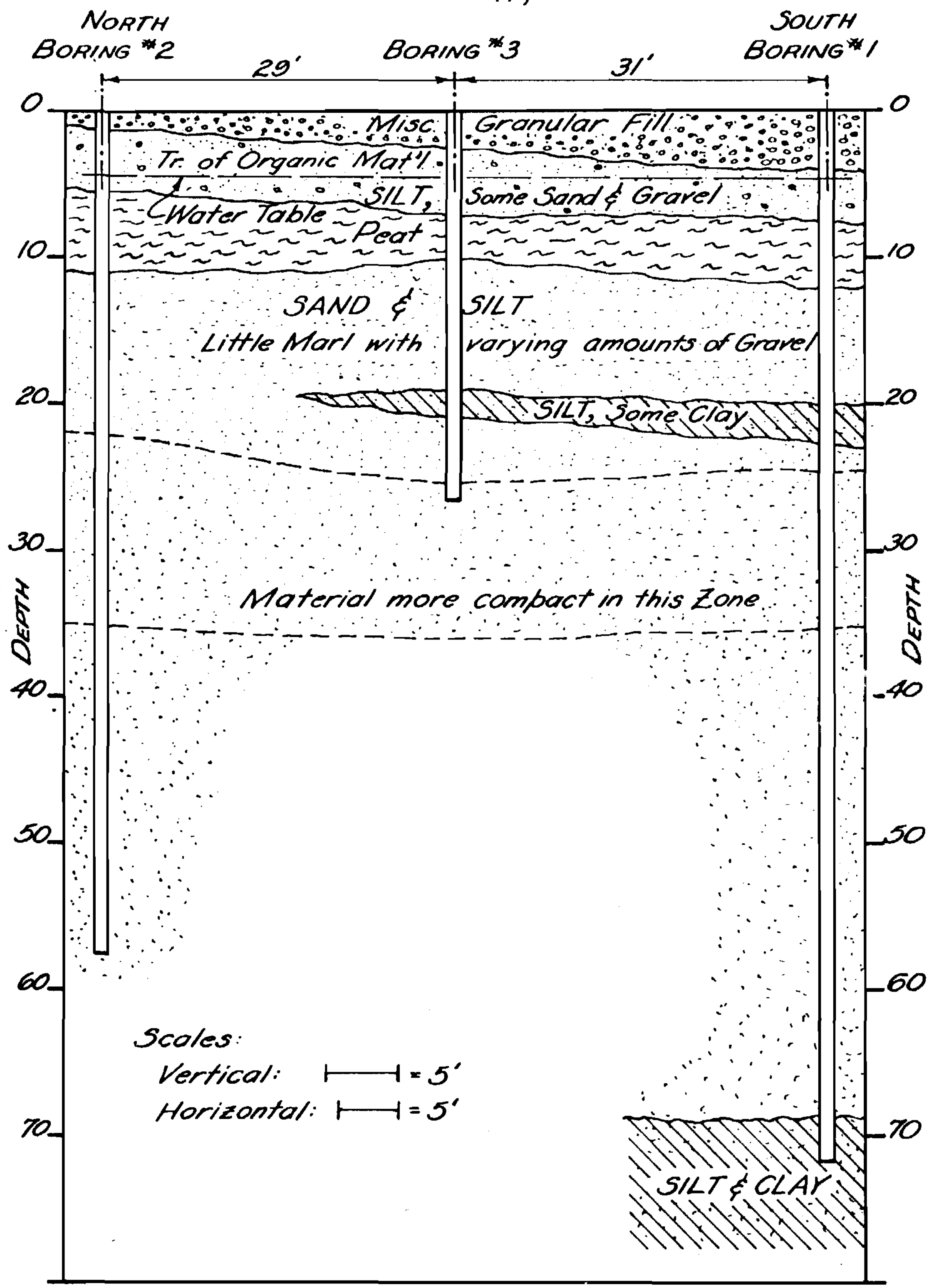


FIG. 1
SOIL PROFILE
SYRACUSE SITE

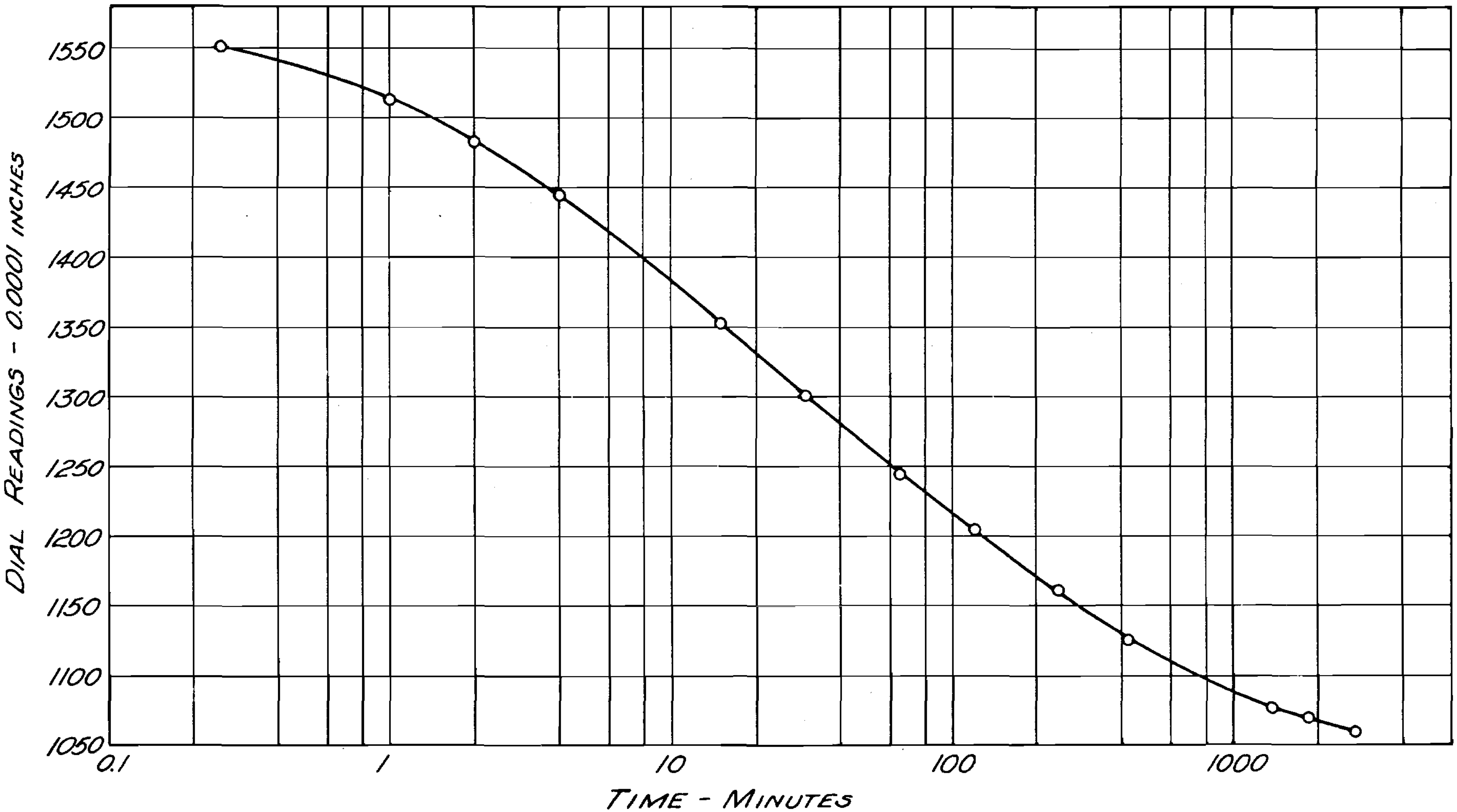


FIG. 2
TYPICAL LABORATORY TIME-CURVE
(LOAD INCREMENT 1/4 TO 1/2 TON PER SQUARE FOOT)

SYRACUSE SITE

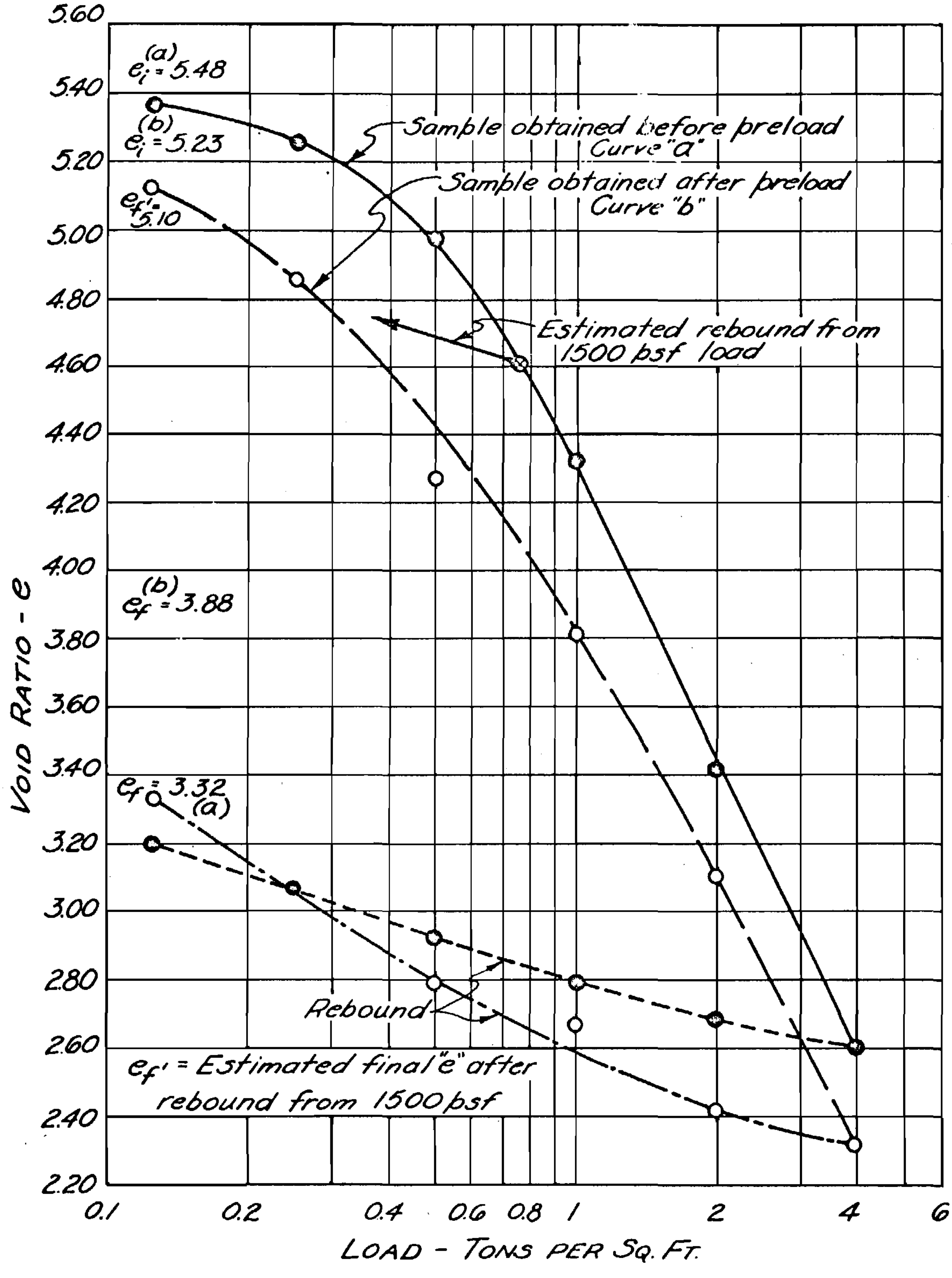


FIG. 3

LABORATORY COMPRESSION CHARACTERISTICS OF PEAT
(e vs Log p)
SYRACUSE SITE

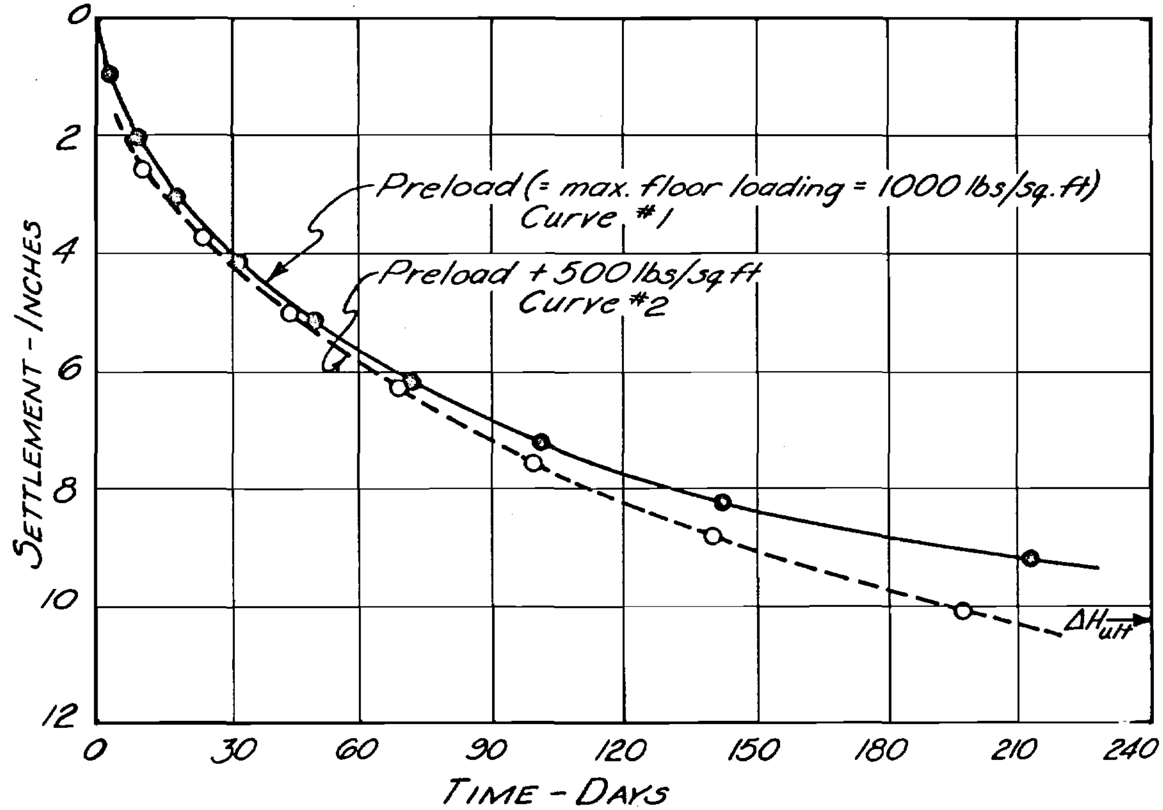
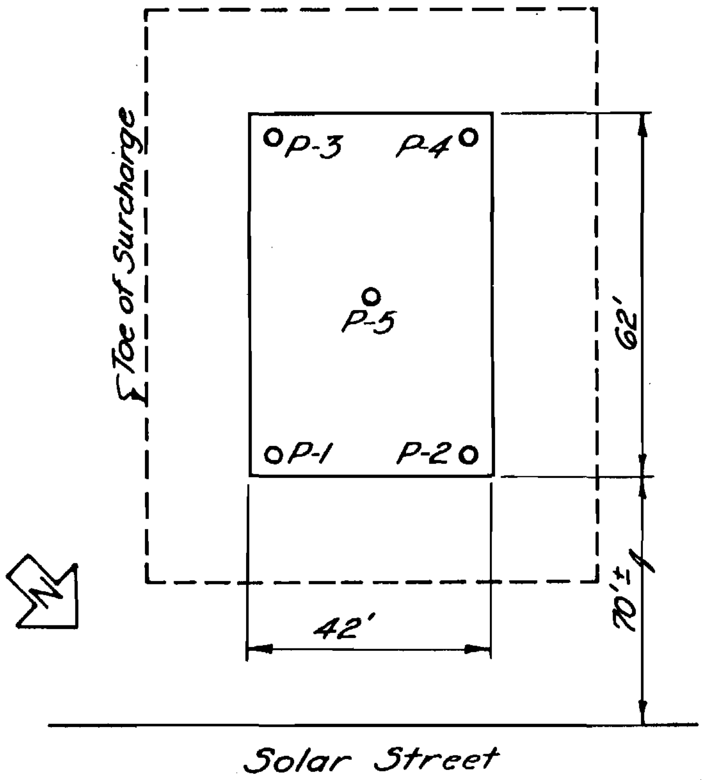
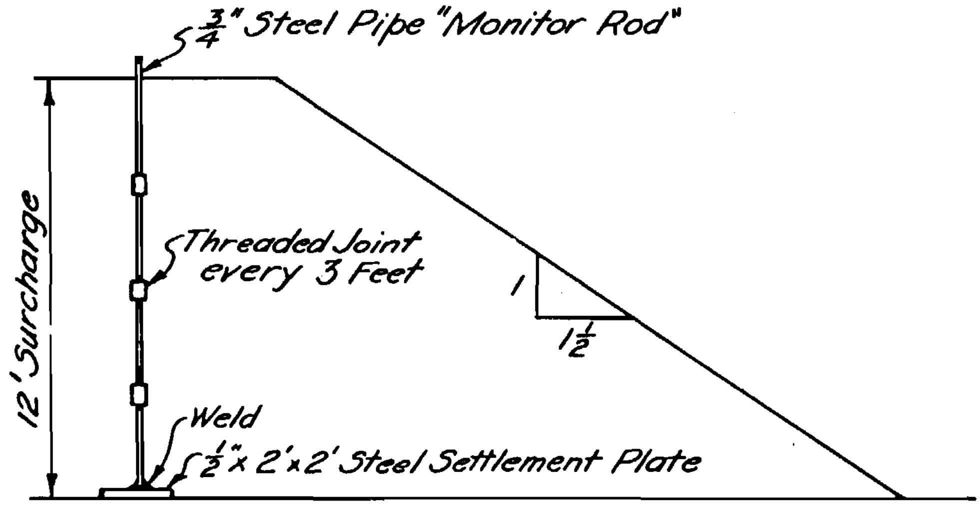


FIG. 4
LABORATORY TIME-SETTLEMENT CURVES
SYRACUSE SITE



A - PLAN



B - PROFILE

FIG. 5
LOCATION & DETAILS OF SETTLEMENT PLATES
SYRACUSE SITE

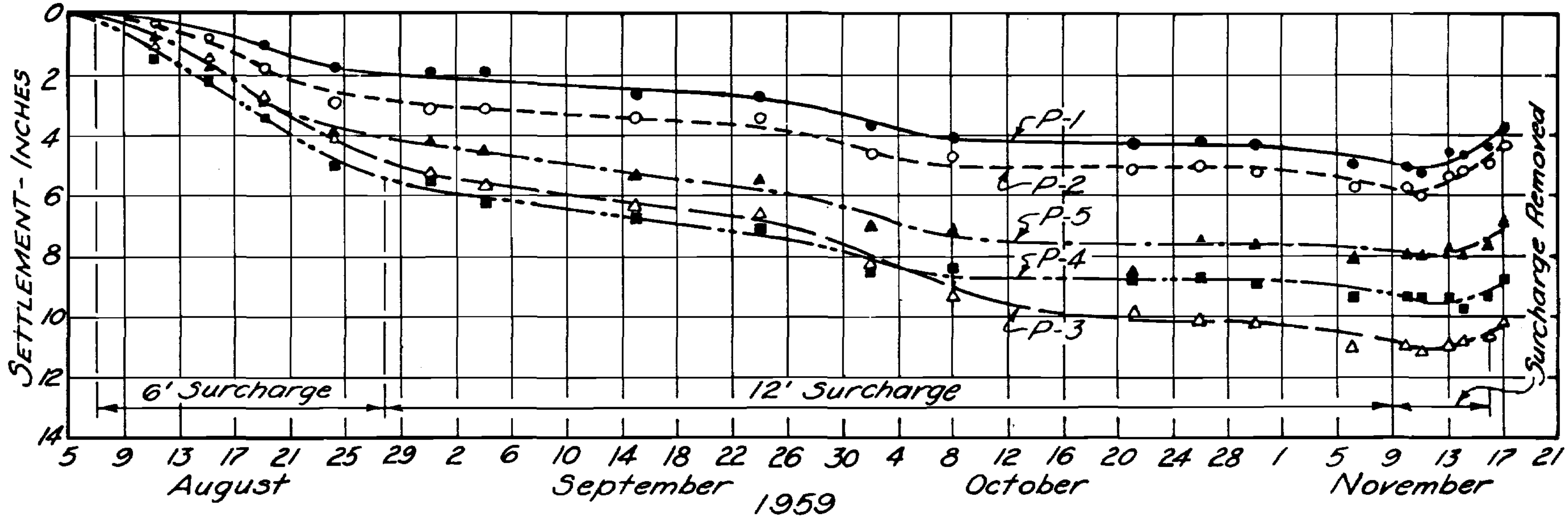


FIG. 6

FIELD TIME–SETTLEMENT CURVES

SYRACUSE SITE

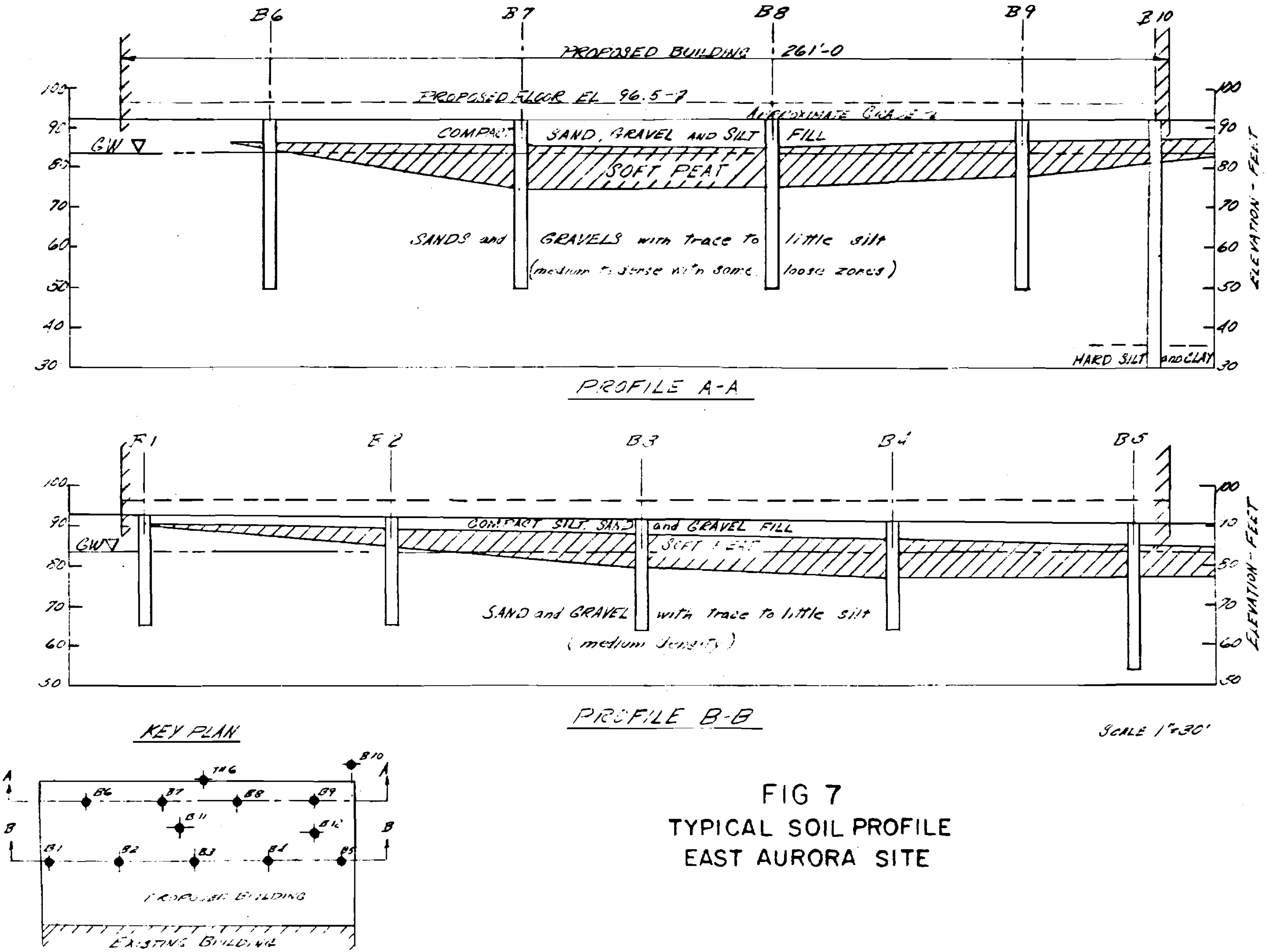


FIG 7
TYPICAL SOIL PROFILE
EAST AURORA SITE

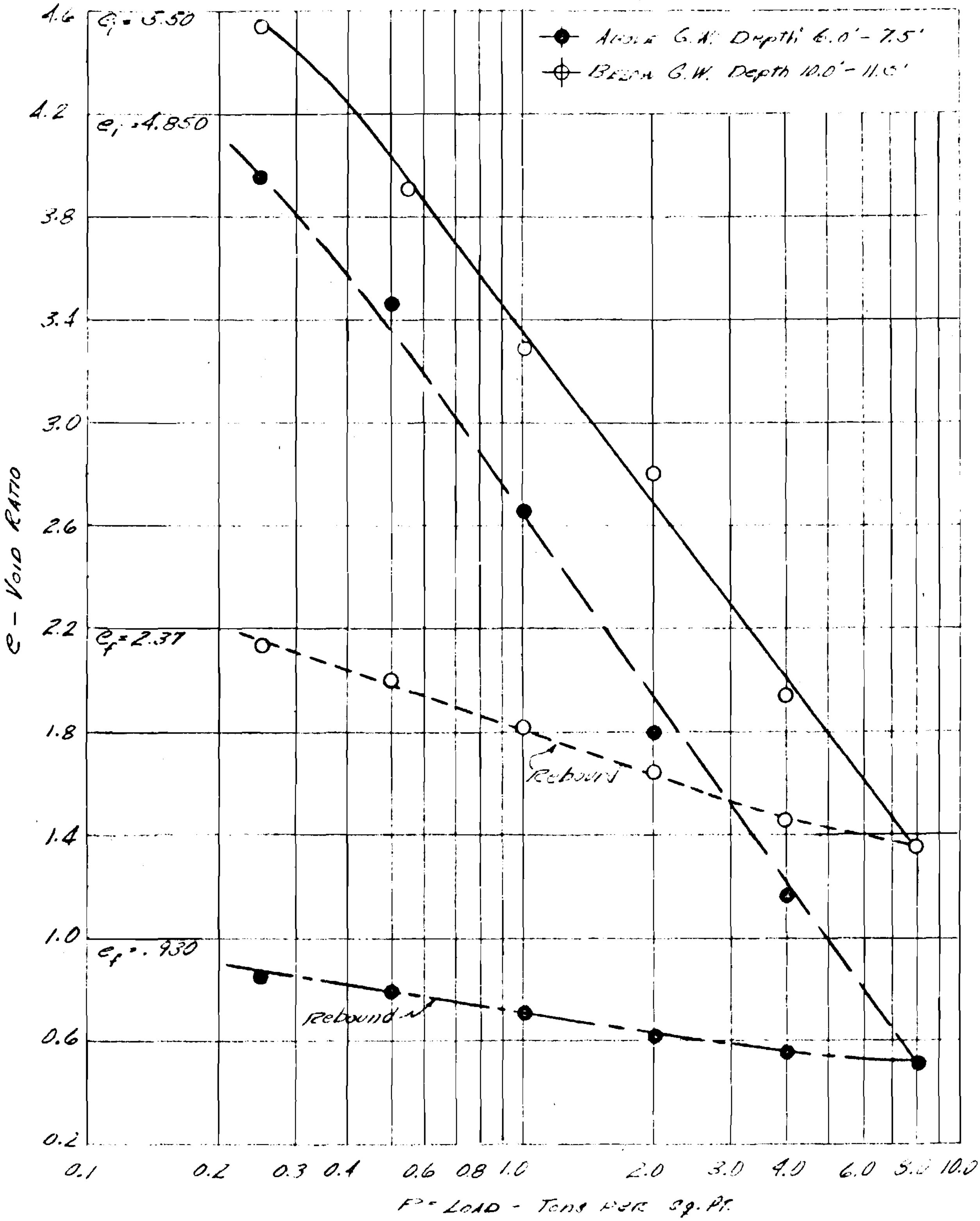


FIG 8

LABORATORY COMPRESSION
CHARACTERISTICS OF PEAT
(e vs log p)
EAST AURORA SITE

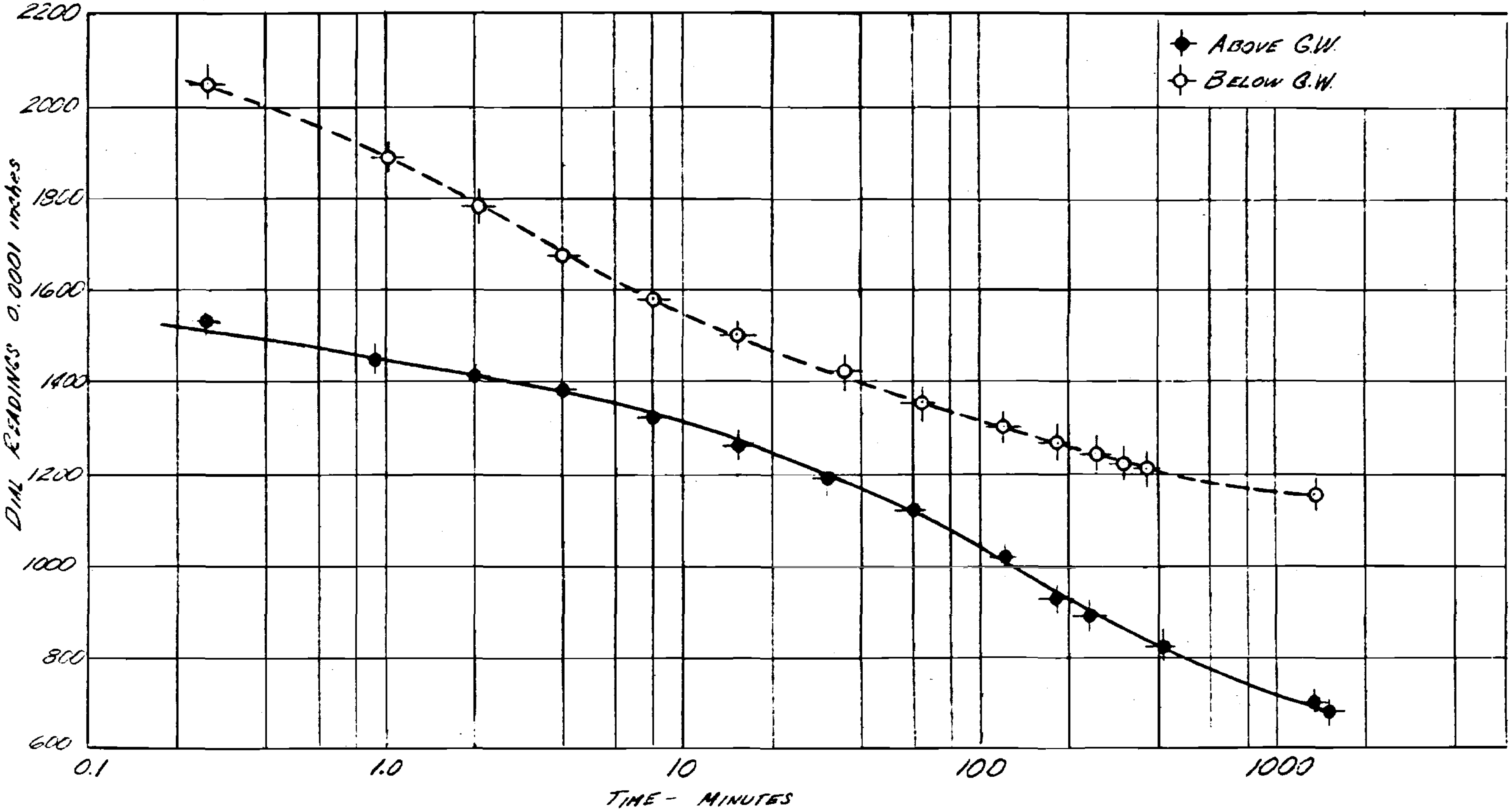


FIG 9

TYPICAL LABORATORY TIME CURVES
LOAD INCREMENT 1/4 TO 1/2 TON PER SQUARE FT.
EAST AURORA SITE

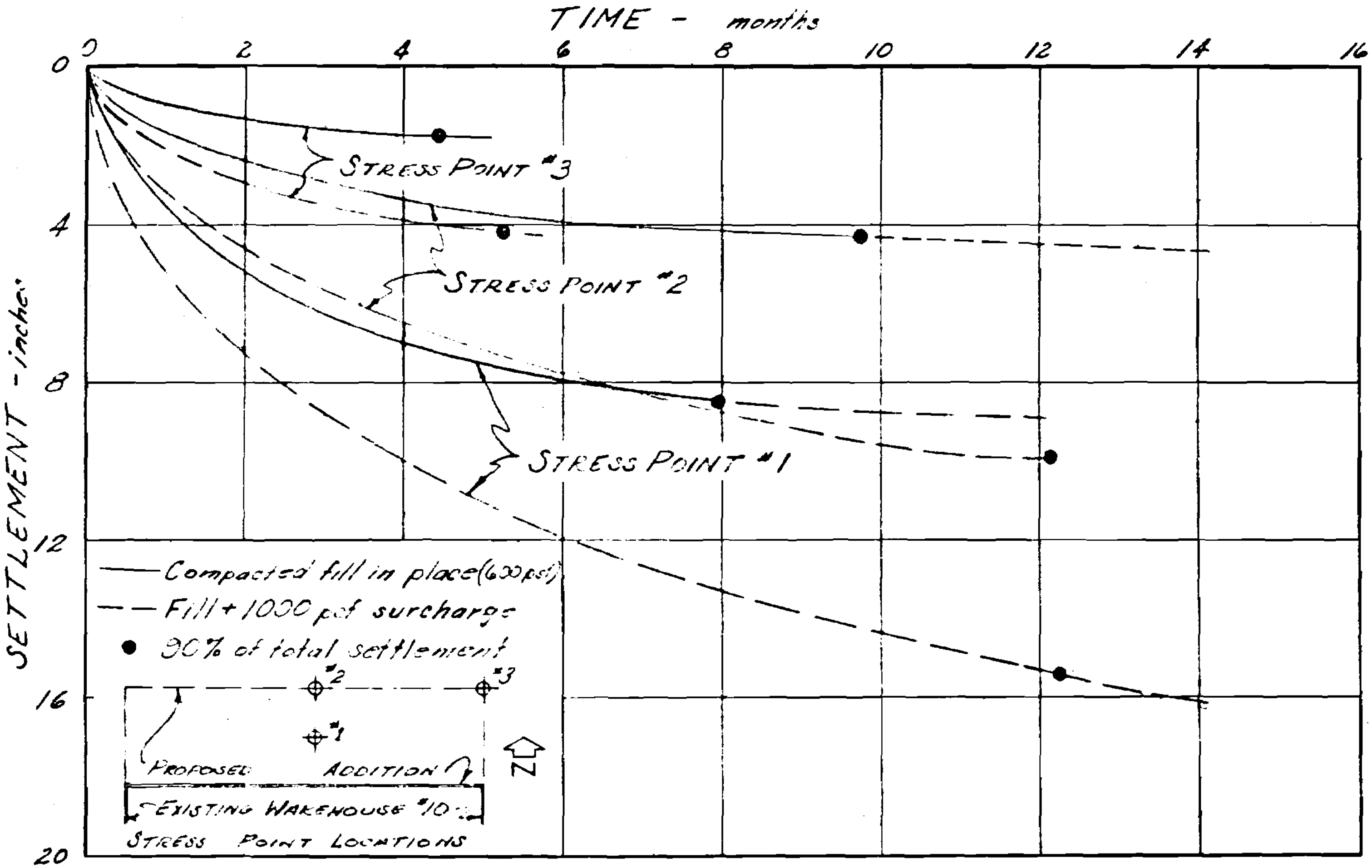


FIG 10
LABORATORY TIME SETTLEMENT CURVES
EAST AURORA SITE

DISCUSSION

Professor Wilson queried Professor Goodman on the type of loading used in his early tests, i.e., the load increment ratio. The reply was that the conventional procedure was followed. Loading was commenced at 1/8 ton/sq ft and doubled until it reached 4 ton/sq ft.

Mr. Lea raised two questions:

1. With regard to the relationship between predicted settlements before and after preloading, it was observed that one curve was shown before preloading and one after. Were these a result of one test, or the summary of a group of tests?
2. Has Professor Goodman looked into the statistical aspect of predicting settlement from a series of tests?

Professor Goodman replied as follows:

1. The "before" curve is based on the summary of a series of tests, whereas the "after" curve is of a single test.
2. No, not with respect to a direct statistical study in connection with the number of consolidation tests necessary to predict settlements with a good degree of accuracy. However, some work has been done with respect to estimating compression characteristics of a site based on a limited number of consolidation tests and correlation of index properties of these samples with other samples of the same material.

III.4 THE ANALYSIS OF SECONDARY CONSOLIDATION OF PEAT

by

J. Schroeder and N. E. Wilson

SUMMARY

This research on peat was carried out in order to reach a better understanding of the basic consolidation characteristics of the material. Peat samples, from muskeg with a cover classification of EFI (1), were taken from Copetown Bog, near Hamilton, Ontario. Ten partly disturbed specimens were consolidated under loads of 0.5 to 20 pounds per square inch; these samples had initial void ratios varying from 40 to 10 and initial heights varying from $\frac{1}{2}$ in. to 25 in. The test data, when plotted on an e-log p (void ratio versus logarithm of vertical stress) graph, indicated a considerable scattering of the results and confirmed the variability of peat from adjacent parts of the bog.

An additional series of samples all taken from one large disturbed specimen was tested to investigate the fundamental behaviour of the material. When plotted, these test results showed little scattering on the graph. A rheological diagram (an axial-stress / axial-strain-rate relationship) for Copetown peat was constructed and this made it possible to investigate whether the material behaved in a plastic or viscous manner. This rheological diagram indicated that peat is a pseudo-plastic material (the term "thixotropic" has been avoided since it is usually used in soil mechanics for age hardening of clays). At low stresses, the strain rate was low indicating plastic behaviour. At higher stresses, however, the strain rate increased considerably indicating viscous flow.

From the pseudo-plastic (thixotropic) characteristic of peat it was possible to draw conclusions about "primary" and "secondary" consolidation; in addition some typical features of the material could be explained.

Any consideration of consolidation found little acknowledgement among engineers until Terzaghi developed a theory for one-dimensional consolidation which enabled him to predict settlement and time relationships during the process of soil compression. His famous theory can be summarized in the following way:

Consolidation of soil yields a stress-strain-time relationship; the strains leading to vertical displacements where there is no possibility of the stresses being sufficiently large to overcome the shear strength of the material. The straining of the material causes a slow escape of the pore water which is accompanied by rolling and sliding of the individual grains into a more dense position. Terzaghi was the first to realize that in the case of clay the plastic resistance to deformation caused by the sliding of individual particles was so small for 80 per cent of the consolidation process that a considerable time lag was not involved. This simplifying assumption led to the development of a time-settlement relationship for clays based only on the escape of pore water.

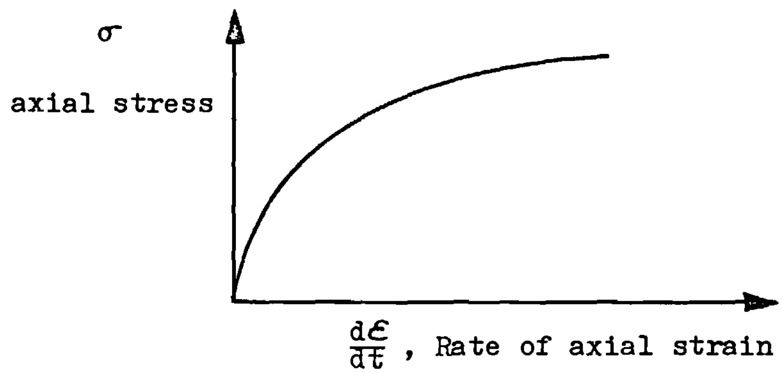
The solution of Terzaghi's equation yields the result that the time taken to reach a certain percentage of consolidation varies as the square of the height of a stratum, as long as drainage is the governing factor. The part of the consolidation process which can be predicted by Terzaghi's equation is called "primary" consolidation; the soil is said to be 100 per cent consolidated when the pore pressure throughout the soil is zero. This instant also marks the beginning of the "secondary" compression wherein the inter-granular stresses cause a creeping deformation of the material and the drainage effects are negligible. "Primary" and "secondary" consolidation are empirical terms since it is obvious that deformation of the soil skeleton has to take place in order to make drainage possible. It can be said that rate of drainage governs the primary part, whereas plastic or viscous flow governs the secondary part of consolidation. The dividing line between the phenomena is purely empirical, but justified in the case of clay.

From the beginning of the research project on peat it was realized that Terzaghi's Theory for Consolidation of Clays could not be applied to peat for the following reasons:

1. Considerable variation in permeability exists during one load increment.
2. The idealized pressure versus void ratio relationship was not considered to be valid for peat as the behaviour of the material indicates a highly plastic and viscous nature; considering $de = a_v \cdot dp$ (a_v is the coefficient of compressibility) neglects the dependence of the void ratio on plastic or viscous characteristics. The assumption that the coefficient of compressibility (a_v) for peat is a constant was questioned but it is considered to be as valid as in the case of clay.
3. Appreciable structural rearrangements take place during the consolidation of peat due to large deformations involved.
4. Boundary conditions applied to Terzaghi's equation are not considered to be valid for peat as large axial strains indicate a moving boundary.

The research work consisted of two series of consolidation tests; all compression tests of the first series being carried out with pore pressure measurements. The time-settlement curves for peat plotted on semi-log paper had characteristics similar to clay consolidation curves for the first load increment only, (Figure 1A). For the following load increments the time graphs (Figure 1B) were either straight lines or slightly curved downwards at the start making the logarithmic time fitting method inapplicable. Points of zero pore pressure occurred at various positions on the straight part of the time settlement curves due to differences in drainage paths but had no effect on the curves themselves. These observations would appear to indicate that secondary consolidation and, therefore, plastic or viscous time relationships are more important for peat than for clay, at least for a load increment ratio of unity (2). These facts made it necessary to investigate the basic character of the material.

The phenomena of viscous and plastic flow can be represented by a Rheological Diagram of the type shown below:



In order to establish a Rheological Diagram for peat, the correlation of all the data from the first test series was necessary. By plotting all test data of the first series on an e-log p graph (Figure 2) a considerable scatter was observed which would yield a poor correlation when analysed by statistical means. As the points plotted for individual samples were on straight lines, it was realized that the scatter was caused by a number of variables (3). First, the variability of the peat can be substantial, even for samples from adjacent parts of the bog and, second, the samples had different lengths of drainage paths.

In order to eliminate these two variables a new series of tests was conducted, with samples taken from one large disturbed specimen and all samples having the same length of drainage path.

During the first test series, it was realized that time relationships and settlements of different peat samples could be compared only if they occurred at the same void ratio, as the drainage conditions and plastic or viscous resistance changed considerably with the void ratio. A test specimen with an initial void ratio of 20 would show different deformation characteristics than one with an initial void ratio of 15, even though both were exposed to the same axial stress. In the second test series, therefore, settlements were always plotted in terms of void ratio changes; the strains and strain rates being referred to the void ratios at which they occurred. Because of the large deflections encountered in peat, strain was defined as the change in height divided by the instantaneous height; in elasticity problems, where deformations are small, strain is defined as deformation per original height.

The importance of void ratio can be shown by tests conducted under identical loading conditions on samples having identical initial void ratios but different initial heights. If identical load increments are applied to all samples for the same time interval, the higher samples will have a greater final void ratio (i.e., less strain) than the shallow samples, as the drainage of water prevents the establishment of higher strain rates (4). However, for the following load increments, the retarding effect of the longer drainage path for the higher sample will be partially cancelled by its lower resistance to viscous deformation, as the sample still has a higher void ratio. This trend will be present during each load increment and eventually samples of different heights may deform at equal rates.

All samples of the second test series showed very consistent results on the e-log p graph (Figure 2) and it was possible to represent the data by a smooth curve. Because of this consistency on the graph it was possible to correlate the test data from the samples and construct a Rheological Diagram for this specimen of peat.

The diagram was drawn by plotting strain rates $\left(\frac{d\varepsilon}{dt} = \frac{1}{h} \cdot \frac{dh}{dt}\right)$ versus void ratios for constant applied stresses (Figure 4). Strain rates at constant void ratios were then replotted as functions of applied stresses. This procedure made it possible to construct an axial-stress axial-strain rate diagram for this specimen of peat, at constant void ratios (Figure 5). The diagram is based on the assumption that the viscosity of the material will vary for different void ratios. The Rheological Diagram for the peat indicates a basic pseudo-plastic (thixotropic) character (5). For example, in Figure 5, if the material has reached a void ratio of 9.5 under a stress of 2 psi its axial strain rate would be approximately 0.0001 $\frac{\text{in.}}{\text{in.}}$/min. If the load is now increased to 3.5 psi, the axial strain rate increases to 0.5 $\frac{\text{in.}}{\text{in.}}$/min, into the viscous flow range. Similar behaviour for clays was anticipated by D. W. Taylor in 1942 (6).

Because of the sensitivity of peat to loads it is reasonable to anticipate the hysteresis effect usually found in the case of pseudo-plastic materials, i.e., that the viscosity of the material is dependent upon immediately prior strain rates. In other words, if the material used in the previous example had an axial-strain rate in the viscous flow range of 0.5 $\frac{\text{in.}}{\text{in.}}$/min at 3.5 psi, then immediately after reduction of the stress to 2 psi, the material would deform under the influence of the previous higher stress; this would introduce a hysteresis effect upon loading and unloading.

There are some indications that this hysteresis effect is present during the consolidation of peat. Figures 3 and 4 indicate that all samples initially exposed to high strain rates (i.e., caused by large load increments), reached a lower void ratio in a shorter time than samples that were consolidated under moderate strain rates.

To illustrate this effect, the behaviour of Sample No. 23 can be compared to the behaviour of Sample No. 32 (Figure 4). Sample No. 23 had a maximum strain rate of 0.5 $\frac{\text{in.}}{\text{in.}}$/min, when the axial stress was increased from 2.5 psi to 4.0 psi, and reached a final void ratio of 8 after 80 minutes; Sample No. 32, when stressed at 4 psi initially, had a maximum strain rate of 5 $\frac{\text{in.}}{\text{in.}}$/min and reached a void ratio of 6.7 after 20 minutes. The final void ratio is not affected by the rate of change of momentum as, for example, Sample No. 32 where the rate of change of momentum accounted for a pressure increase of 20 per cent in the early stages. After a strain rate of 1 $\frac{\text{in.}}{\text{in.}}$/min and a void ratio of 15 were reached, however, the effect of momentum change was only 0.01 per cent.

The effect of drainage is indicated, qualitatively, on the stress-strain rate graph (Figure 5). The sharp increase in curvature on the e = 16 curve, at a strain rate of 1.8 $\frac{\text{in.}}{\text{in.}}$/min, shows a departure from the normal viscous flow curve (dotted line) and indicates that the drainage rate has become the limiting factor.

The effect of drainage can be interpreted from the Rheological Diagram (Figure 5) by use of the following assumptions.

1) The total applied stress is equal to the effective (intergranular) stress plus the pore water stress.

$$\sigma = \bar{\sigma} + u$$

2) Sample deformation can be expressed in terms of change in void volume which is equal to the change in water volume.

$$\text{Strain, } \mathcal{E} = \frac{\Delta H}{H} = \frac{\Delta V}{\text{Area}} \bullet \frac{\text{Area}}{V} = \frac{\Delta V}{V}$$

This assumes that the sample is fully saturated and neglects the gas content of the peat. The pore water and fibre water phase are treated as one unit and the compression of the organic particles forming the fibres are considered to be insignificant.

It follows that

$$\frac{d\mathcal{E}}{dt} = \frac{1}{V} \bullet \frac{dV}{dt} = \frac{1}{H} \bullet \frac{dH}{dt} ,$$

which means that the rate of structural deformation is the same as the drainage rate.

In order to clarify the effects of drainage, a schematic Rheological Diagram (Figure 6), was constructed using the general shape of the viscous flow lines from Figure 5. Drainage flow lines for constant void ratios are introduced; the actual shape of these lines is unknown because the solution to Terzaghi's equation for varying boundary conditions is not available. The shapes and positions of these drainage flow lines could be determined, however, from the drainage characteristics of the samples, if the viscous flow lines are known.

When a sample deforms at a particular strain rate, the sum of the effective stresses causing viscous flow, and pore water stresses causing drainage, must be equal to the total applied stresses. Consequently, according to Figure 6, for a given total applied stress the sample deforms at a strain rate which is uniquely related to the void ratio. It can be shown that the drainage conditions govern the strain rate (7) while viscous resistance provides a correction which is independent of pore pressure.

CONCLUSIONS

The research has confirmed the influence of the great number of variables involved during the consolidation process. It has been found that the following factors govern the consolidation characteristics:

1) Variation in material.
2) Drainage conditions.
3) Load increment ratios.
4) Range of void ratios.
5) Stress and strain history.
6) Viscous or plastic resistance to flow.

This preliminary report has given a better understanding of the fundamental behaviour of the material and further research work is currently under way to expand these concepts.

ACKNOWLEDGEMENTS

The authors would like to thank Dr. Radforth for his encouragement and assistance during this research work. The project was conducted with the assistance of a grant from the Defence Research Board (Grant No. 9768-04).

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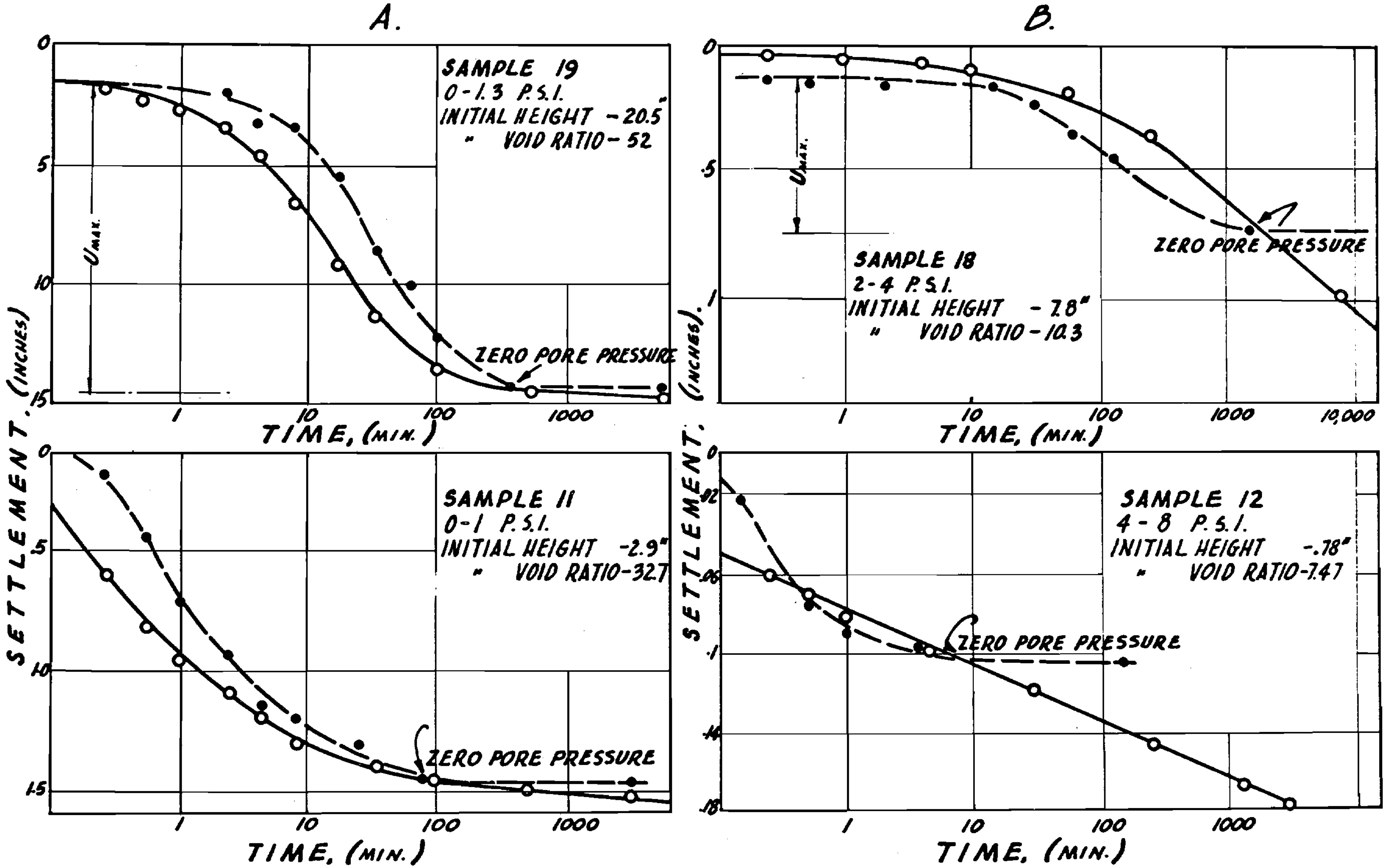
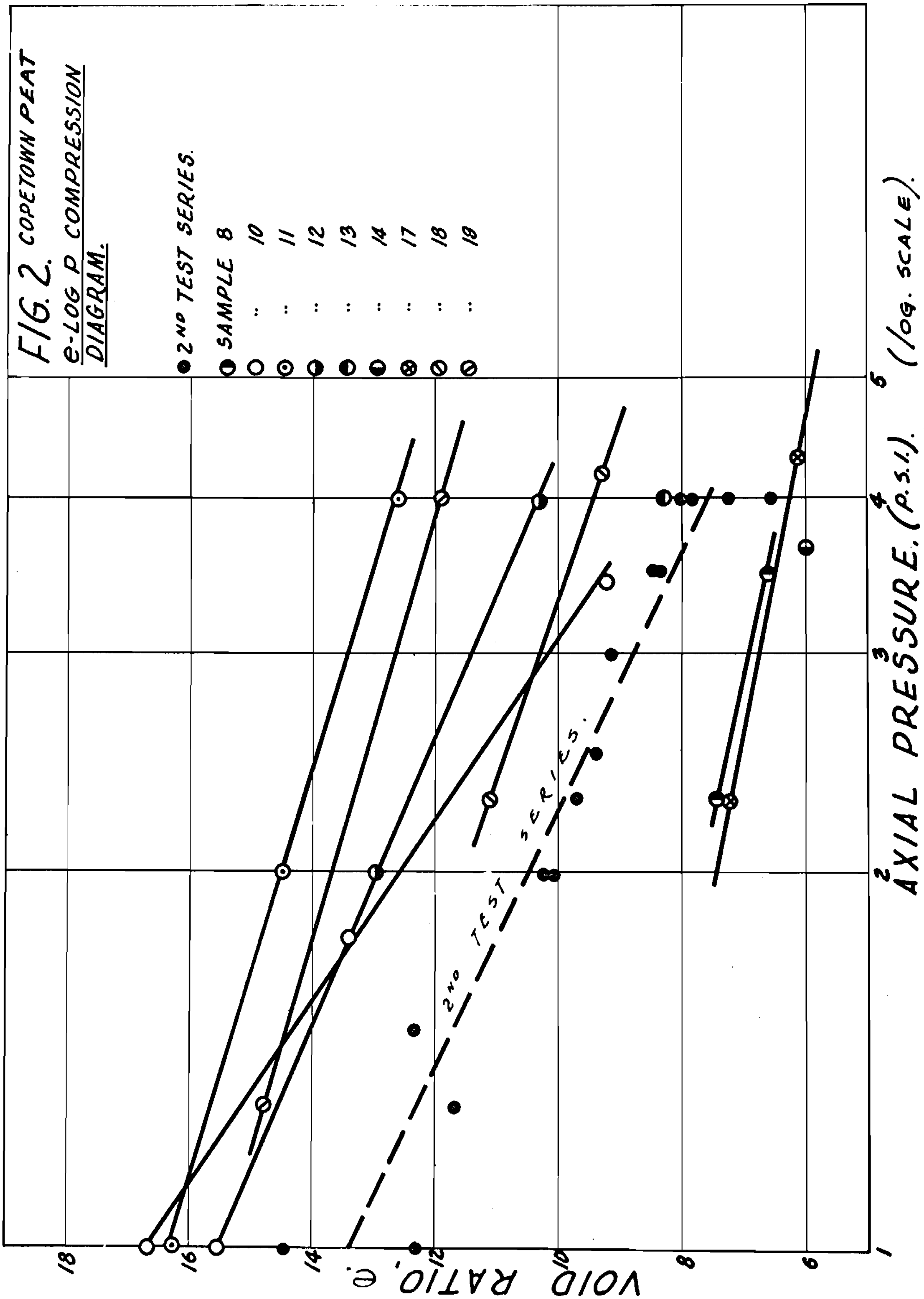


FIG. 1. COPETOWN PEAT
1ST TEST SERIES

TYPICAL e-LOG t PORE PRESSURE DISSIPATION CURVES

○ TIME-SETTLEMENT CURVES.
• PORE PRESSURE DISSIPATION CURVES.

FIG. 2. COPETOWN PEAT
e-LOG P COMPRESSION DIAGRAM.



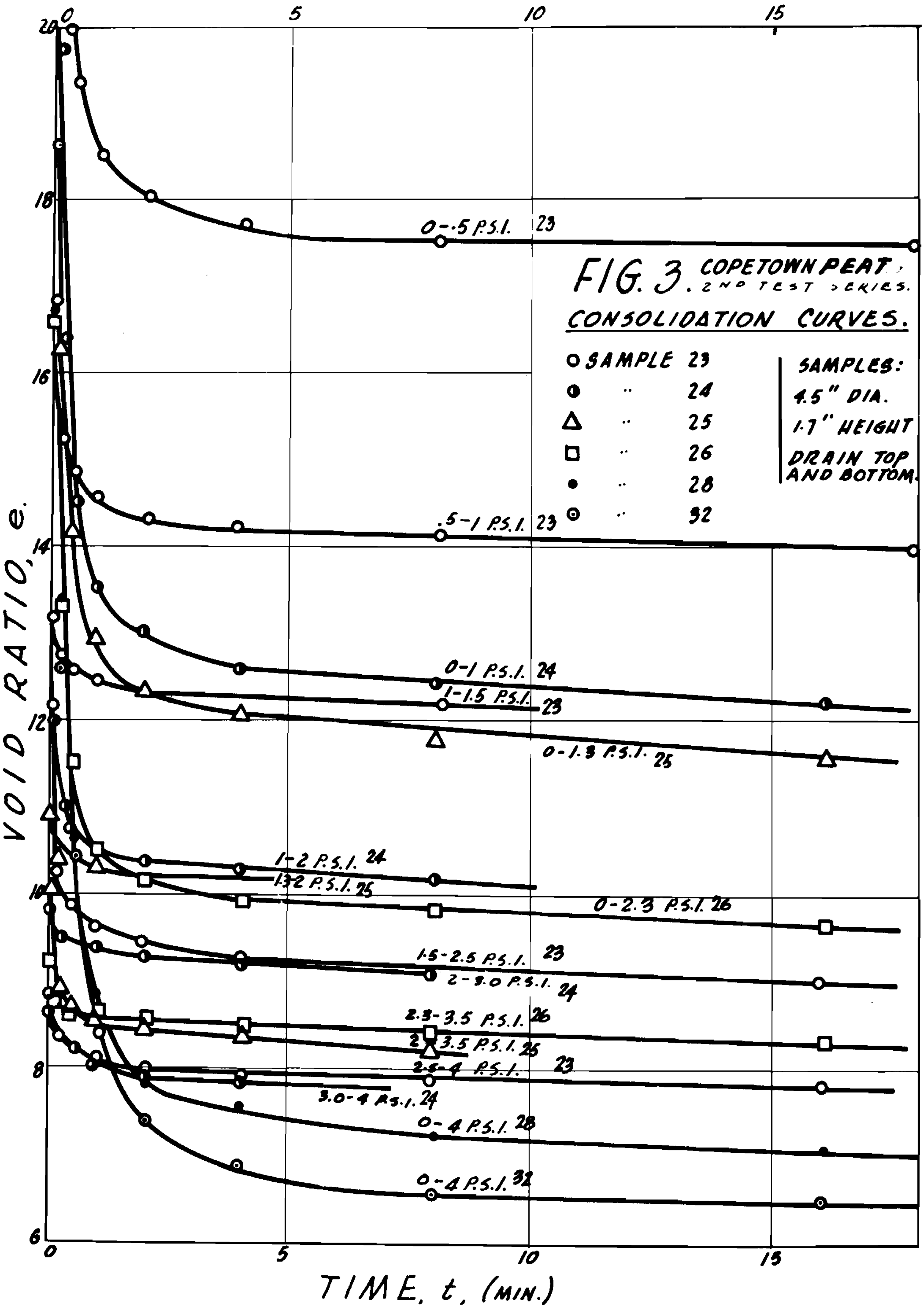


FIG. 3. COPETOWN PEAT, 2ND TEST SERIES.
CONSOLIDATION CURVES.
O SAMPLE 23
◐ .. 24
△ .. 25
□ .. 26
• .. 28
⊙ .. 32
SAMPLES:
4.5" DIA.
1.7" HEIGHT
DRAIN TOP AND BOTTOM.
0-.5 P.S.I. 23
.5-1 P.S.I. 23
0-1 P.S.I. 24
1-1.5 P.S.I. 23
0-1.3 P.S.I. 25
1-2 P.S.I. 24
1.3-2 P.S.I. 25
0-2.3 P.S.I. 26
1.5-2.5 P.S.I. 23
2-3.0 P.S.I. 24
2.3-3.5 P.S.I. 26
2-3.5 P.S.I. 25
2.5-4 P.S.I. 23
3.0-4 P.S.I. 24
0-4 P.S.I. 28
0-4 P.S.I. 32
VOID RATIO, e.
TIME, t, (MIN.)
6
8
10
12
14
16
18
20
0
5
10
15

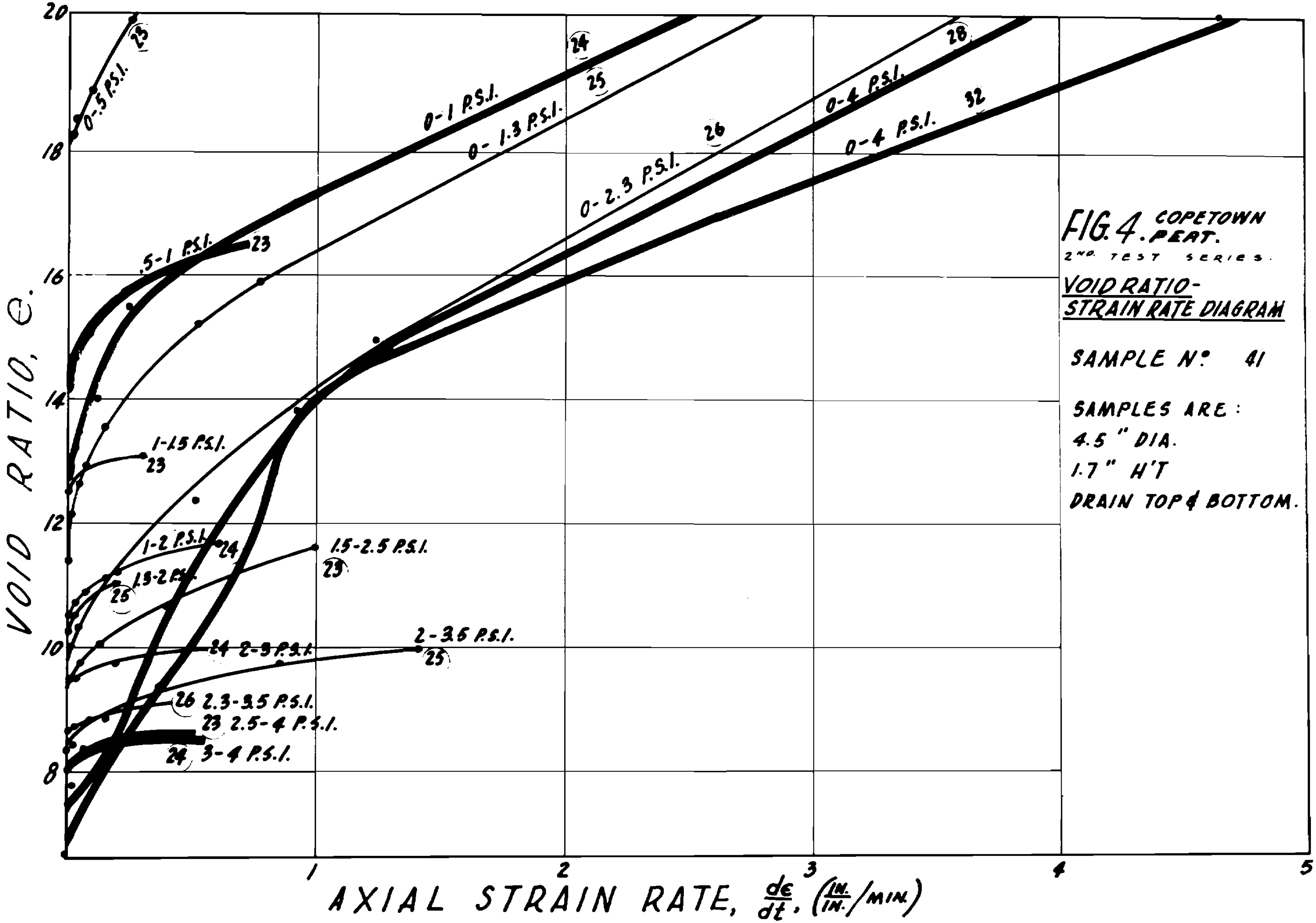


FIG. 4. COPETOWN PEAT.
2ND TEST SERIES.
VOID RATIO-STRAIN RATE DIAGRAM
SAMPLE Nº 41
SAMPLES ARE:
4.5" DIA.
1.7" H'T
DRAIN TOP & BOTTOM.
VOID RATIO, e.
AXIAL STRAIN RATE, $\frac{de}{dt}$, (IN./IN./MIN)
20
18
16
14
12
10
8
1
2
3
4
5
0-.5 P.S.I. 23
0-1 P.S.I. 24
0-1.3 P.S.I. 25
0-2.3 P.S.I. 26
0-4 P.S.I. 28
0-4 P.S.I. 32
.5-1 P.S.I. 23
1-1.5 P.S.I. 23
1-2 P.S.I. 24
1.5-2 P.S.I. 25
1.5-2.5 P.S.I. 25
24 2-3 P.S.I.
2-3.5 P.S.I. 25
26 2.3-3.5 P.S.I.
23 2.5-4 P.S.I.
24 3-4 P.S.I.

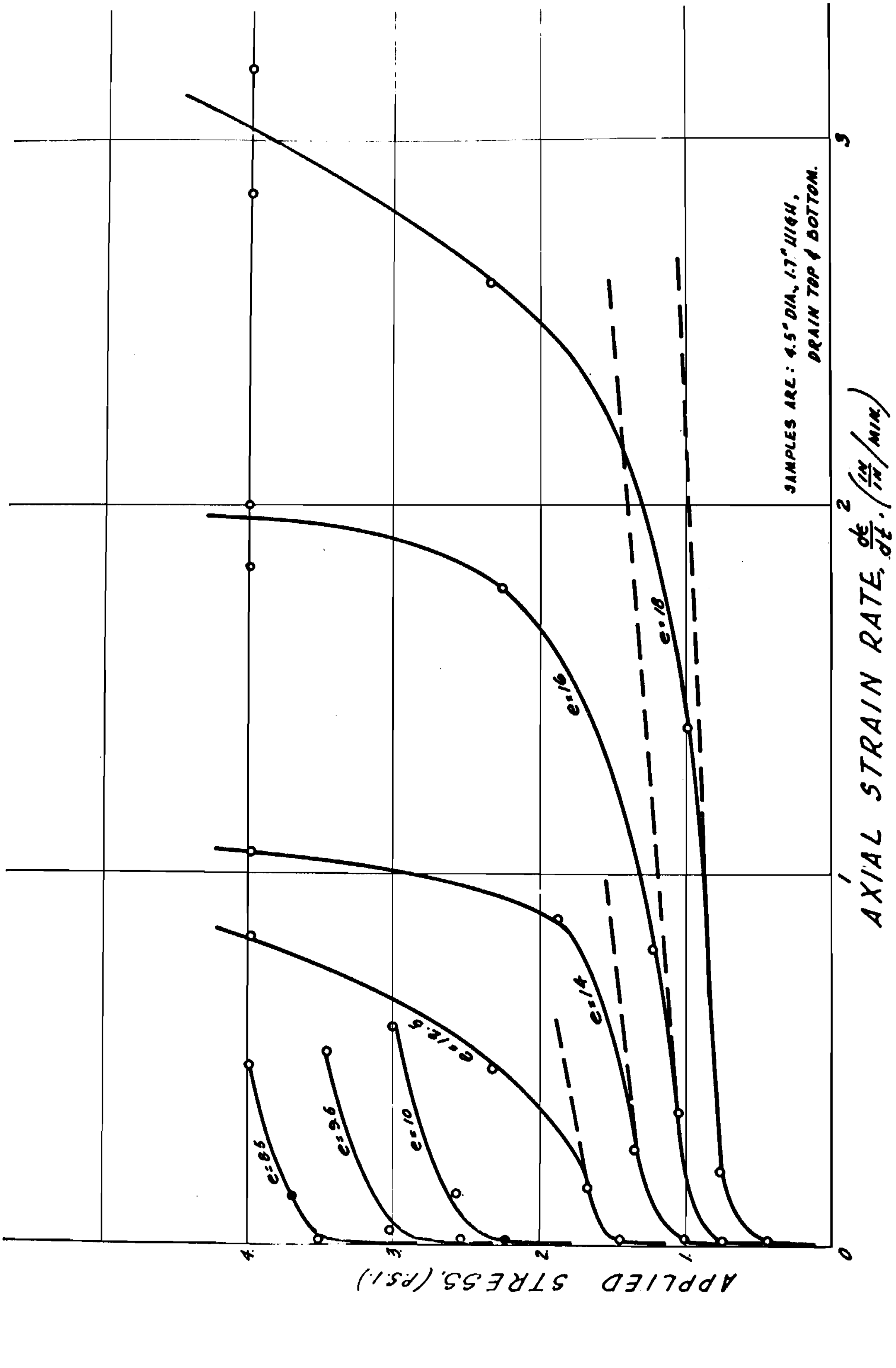


FIG. 5 COPETOWN PEAT. 2ND TEST SERIES.
RHEOLOGICAL DIAGRAM.

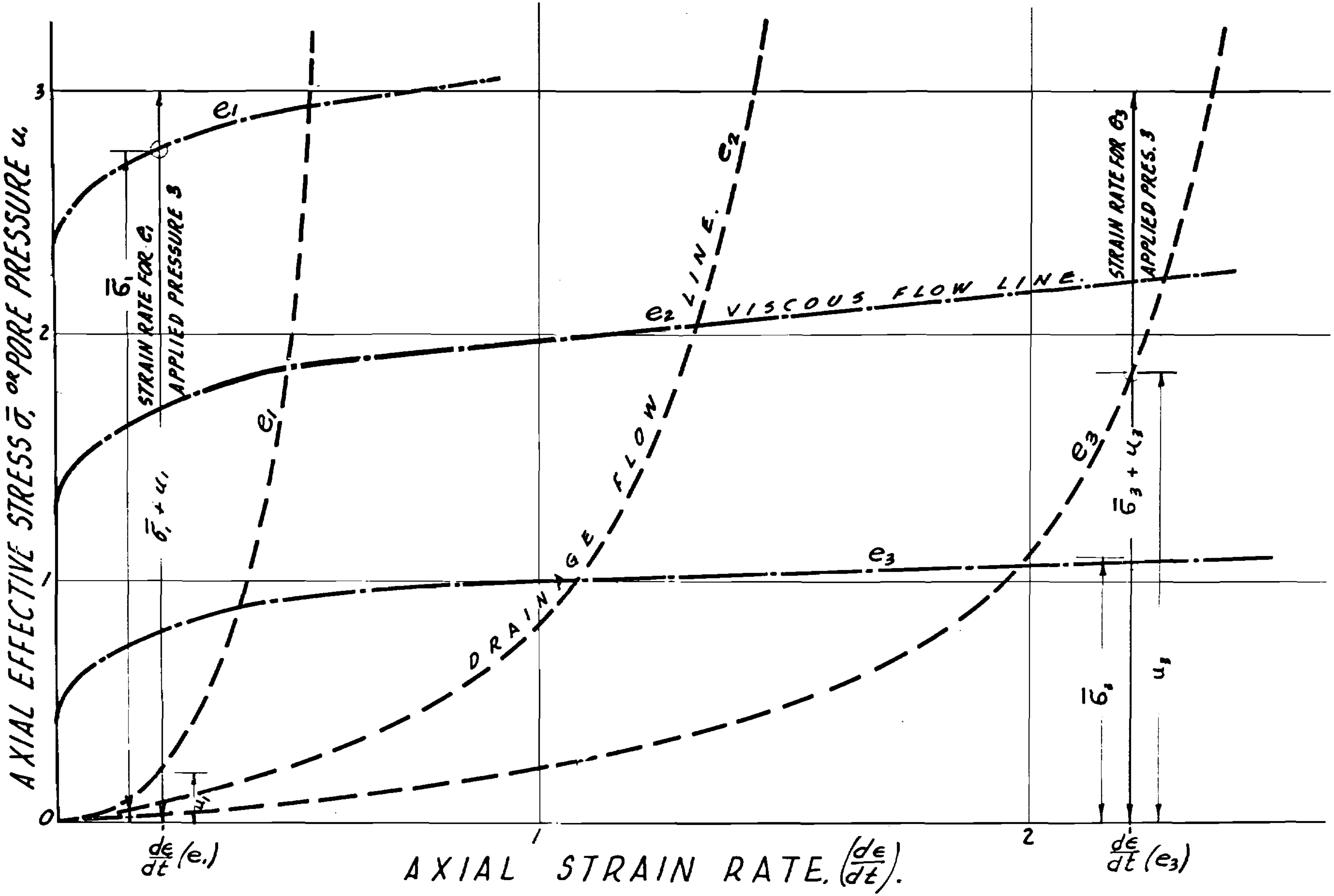


FIG. 6. COPETOWN PEAT. RHEOLOGICAL DIAGRAM.
2ND TEST SERIES.
SCHEMATIC DIAGRAM FOR AXIAL STRAIN RATE Vs EFFECTIVE AXIAL AND POREWATER STRESSES FOR SAMPLES OF CONSTANT HEIGHT.

DISCUSSION

Professor Goodman asked a series of questions summarized as follows:

1. Figure 3 (void ratio vs time) would indicate that equilibrium is reached in a very few minutes. Is this so?
2. Has Professor Wilson gone beyond these loads?
3. Were any loads left on for a longer time than shown on this plot?
4. Does Professor Wilson have any field data to show whether or not there is any secondary consolidation?

Professor Wilson replied as follows:

1. Yes, for this particular thin sample of this particular peat. It must be remembered, however, that the void ratio started very high.
2. Yes, and the same trend is evident although not to as marked a degree.
3. Yes, and the curves remained reasonably straight.
4. No, only laboratory test data are available.

   Also, no long-term data are available as yet.

Professor Goodman remarked that more information is required on time effects. He mentioned that in Syracuse there are several buildings (constructed on peat) with no long-term settlements observed.

Mr. Harwood asked how the void ratios were determined. Professor Wilson said that they were determined by calculation. The specific gravity of each sample was determined and 100 per cent saturation assumed.

Mr. Bergan wondered if change in permeability was determined with the change in void ratio. Professor Wilson replied that in the first series of tests, permeability was determined. It was found to change considerably from one increment to another. Also, gas was produced during the consolidation test, which affected the permeability. Professor Wilson said that when consolidation in a particular load increment was complete, a permeability test was run. The sample was then flushed out and another permeability test run which yielded completely different results. Mr. Hughes inquired as to what affected the change in permeability. Professor Wilson said that permeability is proportional to void ratio.

Mr. Lea referring to Figure 6, commented that the strain rates are exceedingly large, much larger than one would get in the field. Professor Wilson agreed with this, but stated that these rates of strain commonly occur in the laboratory. These high rates of strain cannot last very long, however. He acknowledged that there is a problem here in relating laboratory tests and field conditions. Mr. Lea suggested that primary consolidation is obtained only when these high rates of strain are obtained. Professor Wilson agreed with this interpretation and admitted that the laboratory tests do have their limitations. Mr. Lea observed that laboratory tests are excellent to increase fundamental understanding of peat. However, he thought that it would be difficult, if not impossible, to get the required experience in the time effect of peat compression except in the field. The primary and secondary processes occur together in the field. In secondary effects in the field, one is dealing with extremely slow rates of strain. Mr. Lea stressed that this is where information is most needed. Professor Anderson referred to work carried out by him about four years ago (reported in Proceedings of 5th Muskeg Research Conference, N.R.C., A.C.S.S.M. Technical Memorandum No. 61, 1959). An FI muskeg was loaded to about 0.2 tons/sq ft. Excess pore pressures dissipated within a few days, after which settlement continued at a rate proportional to the log of time. He pointed out that this would be at a much smaller rate of strain than shown by Professor Wilson, i.e. in the order of 8×10^{-6} in./in./min for a 10-ft depth or 0.01 in./in./min for a 1-in. thick sample. Professor Goodman observed that for New York peats he would not like to preload without some prior laboratory information. Theoretical time estimates have been observed to be higher than actual field values. He opined that it was necessary to have laboratory test data checked out by field experience.

Mr. Harwood considered that Professor Wilson's test results pointed up the divergence between classical soil mechanics and vehicle soil mechanics which have entirely different rates of loading. He suggested that for a proper interpretation of the results of these tests it is necessary to go into some field between dynamic and static loading. He suggested further that the main application of Professor Wilson's work might be in the design of track grousers rather than the design of roads, etc.

Professor Eydt wondered what size of particles were removed from the samples before consolidation. Professor Wilson said that this depended upon the size of the consolidation rings and was a matter of judgement. He pointed out that the purpose was to eliminate some element of variability.

Professor Noble commented that Professor Wilson has stated that time rate of consolidation varies with height and depends upon void ratio. He wondered how this would affect Professor Goodman's results. Professor Goodman pointed out that he had been working in an entirely different range of void ratios. Professor Noble asked if the remoulded samples could be referred back to the original peat. Professor Wilson replied that there is a difference between the two materials, but in his work he was interested in a fundamental approach, which is why he used a remoulded material. Professor Goodman wondered how Professor Wilson would define "remoulded." Professor Wilson explained that this simply means he had removed the large chunks of wood.

IV. 1

WHY AND HOW DOES MUSKEG OCCUR?

by

N. W. Radforth

It would be unreasonable if at this time no allusion was made to the circumstances that have led up to the question expressed in the title of this paper. Since the beginning of the author's work on muskeg in 1945, there has been little deviation from a bilateral approach which was designed on the one hand to express and on the other to explain that muskeg is classifiable. The special emphasis on subsurface and aerial interpretation of the terrain, muskeg as it is usually termed, is an embellishment of the main approach dealing with classification.

The story of how the interpretation procedure has developed will not be recounted. This is no longer necessary, because it is known - reference to the proceedings of the Muskeg Conference shows that the system of classification and subsequent interpretation has been applied by several people and for different purposes. On the other hand, there may be some who do not recall, or who have never known, that in 1945 the muskeg work began as a result of an expediency. This refers particularly to the unhappy state of ignorance in which scientists, engineers, foresters and agriculturalists were held. They were powerless to design, reassuringly and effectively, to overcome problems that this kind of terrain posed. There was no other country in the world, however, with so much muskeg (500, 000 square miles minimum), with so little knowledge of what muskeg was, and yet with so great a need for access to the north, to which muskeg was a formidable barrier one way or another.

The expediency is still present and the work of improving the classification and interpretive devices will need to continue. Also, as this Conference has demonstrated, much attention will need to be placed on assessment of physical and mechanical properties of vegetal cover and the peat beneath it to give adequate engineering and scientific meaning to the botanical phenomena which indirectly govern the natural and ordered basis of muskeg.

On the other hand, what little is now known about what muskeg is provides a stimulus and good reason to inquire into why and how it arose. Metaphorically, muskeg to the engineer and scientist is like a threatening disease is to an ignorant patient. He knows he is stricken; he can name his malady and even classify his symptoms; but he will help himself to do a better and braver job to recovery if he understands and not merely recognizes his problem. The better the phenomenon of muskeg is understood by the scientist, the less will he and his associate, the engineer, rely upon empiricism. To recognize by organized signs, therefore, is not enough.

Understanding begins with basic concept. Because muskeg, top to bottom, was all living at one time or another, it is not too unrealistic to liken it to an organism. Indeed, a seed or a spore started it on its way. The peat that eventually followed literally grew into place as the legacy of the living cover. Then, like an organism, it differentiated according to an ordered sequence. Differentiation was followed by development and the building of a complex massive structure which proceeds to maturity.

The convenience of this concept is augmented by a secondary idea which also pertains for the organism - the idea of a dynamic constitution. It must be accepted that muskeg in a given area, at a given time, is never quite the same as it is at another time.

Also, there is young muskeg and old in the sense that it does not all start to grow at exactly the same time. Some of it is originating on mineral terrain at this very moment. It may have arisen spontaneously, or it may have proliferated from the edge of a parent mass of organic terrain already in existence. It has periods of active growth or slow growth and in all dimensions. Occasionally, it deteriorates, or it can be killed or injured. It can regenerate.

On the basis of this primary "organismic" concept, and the dynamic implication, let us now turn to consideration of the controls that govern muskeg formation and growth. There are two sets of controls - one belongs to the living world of plants, the other to the non-living. The former will be referred to as "botanical," the other as "extrabiotic" or, literally, outside life. Now to know how these controls operate at all times and in all circumstances would require a set of volumes bigger than the Biological Abstracts and the Engineering Abstracts put together. One cannot even begin to account for the origin, differentiation, development and aging of any single organism, much less the masses that have contributed to the condition of muskeg. On the other hand, there are certain things that are known and this will help to generalize sufficiently to provide a basis of understanding on which to work.

THE EXTRABIOTIC EFFECT

The primary reason for muskeg is water. Without it, muskeg would not arise. If one considers plants growing on a normal, well-drained mineral soil, it is readily acknowledged that they mature, die, deteriorate and, through oxygenation, eventually decompose and, - as it were - disappear. If by rainfall or high water table persisting in the mineral soil, the organs of the plant following death are kept wet then they will not disappear though they may change chemically. For mechanical reasons these plant parts will accumulate until in a generation or two no mineral terrain will be exposed. Seeds and spores of some plants will germinate on such a mat or sometimes new cover develops from simple vegetative expansion of the old cover. Whatever the source, this also adds to the fossilized layer where it will remain indefinitely so long as the water, the preservative, persists. The vegetation contributing to this layer need not be sub-microscopic or even microscopic; it could be a massive tree trunk. Whatever the source of the fossilized layer and no matter how deep it may be it is known as peat. This, together with the living cover, is the material of muskeg. In the region of Prince Rupert, British Columbia the author has encountered peat between 70 and 90 ft deep, and it continues to deepen according to the process described.

Once the peat is established, the water factor may not necessarily be present in terms of free water as it was in the beginning. It may be water of capillarity or of adsorption, or it may even be water in all these three aspects. If it is free water it may be derived from a fundamental water table or from a perched water table. It may have a complex array of solutes and may vary in pH from strong acidity to strong alkalinity. It may be present as liquid or as ice. If it is removed by either evaporation - which may at times exceed precipitation - or by drainage, deterioration of peat immediately ensues and decomposition can be expected. Analyses have shown that this in fact does happen on occasion. Eydt's work (4), which concentrates on examination of tissues, shows that older peat may deteriorate to the point where it can no longer be recognized as tissue. It also shows that deterioration may go on evenly or unevenly. In other words, there are differentials with respect to both deterioration and deterioration rates.

Another reason for muskeg is temperature. Sub-Arctic and Antarctic temperatures are most favorable for the encouragement of peat formation. On the other hand, as long as temperatures do not increase high enough or continue long enough to disturb the appropriate water relations, organic terrain can exist and increase in amount in the sub-Tropics and Tropics. The temperature, therefore, is apparently secondary and is a controlling factor only when water is limiting and controls in the sense that it contributes to water becoming a limiting factor when temperatures are high.

On a similar basis, wind must be regarded as a significant agent of control. When wind effects high rates of evaporation, peat accumulation will not take place. If the wind factor becomes limiting after the peat has originated, accumulation is arrested and recession may follow.

Mineral sub-layer is also important in effecting peat accumulation. Like the other factors, its significance lies partly in its connection with the water factor. If the type of soil and the form in which it exists are such that a depression is formed no matter how shallow - the accumulation of peat will be inevitable if the water factor is appropriate. It does not matter how shallow the depression is so long as it will hold free water long enough to enable the accumulation process to ensue.

It is commonly thought that fossilization will begin at the outer rim of a depression. This will be the case only if water is consistently present at that position. Also, it must be kept in mind that wind will encourage wave action, which in turn will promote aeration and mechanical disturbance not immediately conducive to fossilization.

Finally, it should be indicated that especially in the early stages of peat formation, fossilized debris gets washed towards the centre of the depression and accumulates secondarily in that position. Where free water is in consistent supply either from rainfall, or release from a reservoir of ice or from normal ground water table, nuclei of peat generated in depressions will overgrow and coalesce to form a continuous sheet of organic terrain. The rate at which the terrain becomes continuous depends in part upon the amplitude of change of the local topography and upon the soil constitution in the land adjoining the depressions. If ice or water recession is rapid, peat accumulation beyond the depression will be small or postponed indefinitely.

Major orders of topographic difference lack ultimate significance. Thus, peat may form on hillsides or on mountainsides as well as on flats. Similarly, in the last analysis, the kind of mineral soil makes little difference. Peat may accumulate on any kind of mineral aggregate, including solid rock.

Where drainage is good and between depressions, the amplitude of microtopography is high, peat is usually confined only to the depressions even though these are at different elevations. This situation is common in Precambrian country as, for example, in the Quebec Peninsula. Organic terrain is then said to occur in the confined rather that in the unconfined sense. In such situations, the depressions are often very deep. Unless climatic or geomorphic events intervene,however, peat accumulation will be relentless and all the water will be displaced by peat, a situation which has alréady arisen in many "kettle holes" or similar depressions that have been left as the result of Precambrian folding and glacial scouring. Turbulence either vertical or horizontal will, of course, delay the process.

Chemical constitution of a mineral terrain will also have a controlling effect, but chiefly through control of growth and selection of the plant species which normally associate in consistently wet areas to produce peat.

Finally, with reference to land-form, instability may arise and persist as manifested by primary and secondary sedimentation phenomena. Thus, formation of organic terrain may be delayed or interrupted. The latter results in buried peat beds which incidentally are common in the Pleistocene as well as recent times, and occur as far north as the Arctic islands. In these conditions, swamp arises, not muskeg. Peat will eventually begin to accumulate if stability returns. The end result of this will be organic terrain with at least one mineral soil parting, a situation commonly found in confined muskeg.

If a depression occurs in which water is inconsistently present, the wet vegetation will give rise to marsh, but this is not organic terrain because peat formation does not occur.

Among the extrabiotic factors, sun and high humidity undoubtedly play a part. The former associates with temperature and constitution of the living cover. The latter behaves likewise but in a different sense and is undoubtedly linked with the water factor.

THE BOTANICAL EFFECT

The question of why muskeg forms is partly answered by reference to qualities intrinsic to the plant constituents.

Often the tissues comprising peat can absorb many times their weight in water. This is a characteristic endowed by the plant constituents of the living cover. Sphagnum, if present, is the chief contributor and agent responsible for this feature. Plants of this genus can survive recession of the water table and maintain a positive water regime by retaining water of adsorption and capillarity until the water table is recharged. Once these plants become established, they grow upon their own fossil remains and literally take the water table with them - at least on a local scale. When the deposit becomes several feet deep, the main water table gets left behind and a perched water table exists above it in the top zone of living and fossil material. When the deposit is very deep very often there is only the one zone of free water, which is at the top and no free water is found at the base of the peat.

Eydt's work (4) has demonstrated that there are combinations of tissues constituting distinctly different kinds of peat. This accounts for the differences experienced on qualitative inspection (2) and for some of the differences in water relations. The main point is that organic terrain, once established, can maintain the water requirement for fossilization independent of the extrabiotic factors that would normally prevail for the area.

Some plants contributing to peat produce a loose, open mesh in their growth habit. When these die, no peat will form if free water does not persist. If there is free water in appropriate supply, these plants will fossilize to an open mesh structure. This will persist except where there is incipient water loss and excessive ice action and then the structure will become granular with negligible mesh and with an increased ability to adsorb water. This encourages slow increase in depth, poor insulation, percolation and rapid recession of winter ice.

Other plants endow the peat with a small mesh of enormous internal area which - with the help of woody fibrous constituents - resists natural compression. This provides for fast growing muskeg with high insulation value. It favours limited percolation but encourages ice retention.

Where wood is absent in the small mesh, mechanical compaction ensues and free-water retention is favoured.

Plants characteristic for peat cover (1) form structural associations. This also is conducive to peat formation. Frequently lichens are associated with mosses, the latter growing directly beneath the former. The lichens can withstand incipient drought and often do so even when associated with organic terrain. The drought is not critical, however, and peat formation continues because presence of lichen prevents inordinate water loss from the moss. Thus, without the lichens the high water table or the water factor generally would limit further development. The moss, on the other hand, supplies the water to rejuvenate or maintain the lichenaceous mat. Thus, the living moss beneath the lichen continues to grow, contributes to the peat and wears its competitor as a crown to symbolize its dominance and ultimate control. Perhaps this single example will suffice to show why plant associations may generate the conditions conducive to muskeg origin.

The principal reasons why enable us to understand how peat arises but a complete answer defies simplicity. To explain how, one approach would be to emphasize origin, another to gain understanding of development on a comparative basis, and a third would be to make an investigation of present states in which peat is found aside from development. The last mentioned is perhaps the least fruitful because a complete understanding must have reference not only to horizontal axes, but also to vertical axes.

Deep peat has had an estimated ten to twenty-five thousand years to form. One might wonder why some peat is just beginning to grow and as yet is only one to three inches deep. Remembering that both the earth and the muskeg behave dynamically, to claim that in 15,000 years shifting circumstances will provide ample opportunity for muskeg to arise needs no defence. Yet, once peat commences to form in the unconfined condition, it stays remarkably constant in structure throughout depth. For confined organic terrain, change in structure is on the other hand much more likely to be encountered.

This suggests that peat composition must differ in different areas. Even from the outset of deposition the records show this to be the case, though the number of differences is few - no more than sixteen categories of composition as revealed by inspection of gross structure. Aerial observations of muskeg suggest that the culminating structure reflected in the living cover varies from place to place as did the original cover. The end products of the depositional process, in terms of composition, are also few in number, and these recur in the south as well as in the far north. Except for the trees, the young muskeg nearest the ice-cap shows patterns of organization similar to those of the old, and those structural combinations arising on newly exposed mineral surfaces are markedly similar to those now arising on the old eroded surfaces of the south.

The best explanation of how the terrain arises comes from a threefold approach to the comparative anatomy of peat, especially if this is accompanied by information on physical and mechanical properties. Eydt (4) is now exploring structural difference on the basis of comparative tissue analyses. Stewart (4) derives explanation from cuticular remnants. Suguitan (3) reads history of development from the pollen and spore sequence. Dr. J. Terasmae in Canada, and Dr. H. Godwin in England are differentiating on the age basis with the aid of Carbon 14. MacFarlane, Wilson, Ashdown and others are explaining how as well as contributing to the original question which was asked seventeen years ago - "What is Muskeg?" Though the answers to the new questions of "Why" and "How" will take years to unfold, it is hoped that the job will be done before the next ice sheet descends and puts an end to the problem. In the meantime, there are reasons to be encouraged!

ACKNOWLEDGMENTS

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DISCUSSION

Following his formal presentation, Dr. Radforth answered several prepared questions which had been submitted to him relevant to the topic of "Why and How Does Muskeg Occur?"

1. "What relationship is there between the dense large "A" type tree cover and the subsurface conditions such as drainage, peat structure, shear strength, peat depth range, etc. as compared to the sparse tree covered "D" or "B" type muskeg?"

Dr. Radforth pointed out that ADE muskeg is always shallow. DFI is also shallow, but is deeper than the order of depth of ADE. Dr. Radforth stressed that ranges of values are just as significant as absolute values. With regard to drainage, ADE is gently sloping, has a low amplitude of knolling, etc. DFI muskeg has "traps" between the root structure, filled with amorphous-granular peat. Usually it is associated with ADE muskeg, which can be drained to the DFI.

2. "What relationship is there between the subsurface structure shear strength in the main body of an FI or EH type of muskeg and the same characteristics in the drainage features that flow through this common type of muskeg?"

Dr. Radforth said that one cannot really measure shear strength in this type of muskeg as yet, so it is difficult to answer the question. He thought that perhaps this was not very important since what is wanted is to evaluate the material so that one can design for settlement, for example. However, the instruments have not yet been designed which can measure the complexity of the terrain. He pointed out that the values obtained with the vane and the cone may not in fact be what the observer thinks he is getting. Dr. Radforth explained that the peat under an FI cover is always amorphous-granular. Under EH cover it is fibrous, with some woody fine fibrous and is much stronger than the amorphous-granular.

3. "To what extent and how serious is the change in subsurface peat material beneath mats where horizontal water movement is going on? Is there a significant change or deterioration or decomposition due to this water movement?"

Dr. Radforth said that when water is moving horizontally in peat, it is simply a sure sign that drainage exists, and it is possible to determine the amount and extent of drainage. He doubted that there was any significant change in the peat structure due to this water movement.

Mr. Harwood referred to the albedo of soil, which has not yet been mentioned. He pointed out that the albedo of lichens in the evapotranspiration cycle is a very important factor. Dr. Radforth agreed and commented that it is interesting that lichens supply a very thin cover. However, their presence is quite marked because of their colour and structure. He pointed out that the structure of lichens has no significance to the structure of peat. Lichens are an agent to help peat growth to continue, in that it prevents the evaporation of water.

Mr. Harwood wondered if the strings in string bogs have a preferred orientation depending upon prevailing wind direction. Dr. Radforth replied that one side of the string slopes gradually towards the water; on the other side it is a shear drop. This and certain other factors would point to wind as a primary cause of string orientation. Dr. Radforth stated, however, that this is not so, as the strings can and do occur in all directions as evidenced in the air photo reticuloid pattern. Mr. Keeling observed that string bogs are rarely found in level, flat muskeg areas; usually they are found in muskegs with a flat grade, and the strings are perpendicular to the grade. Dr. Radforth said this may be so at the beginning but it does not necessarily obtain later on in the development of the string bog. He emphasized that there are four sources contributing to the growth of a bog:

1. mineral terrain
2. ice
3. water
4. inherent factors in the organic material.

Dr. Radforth stated that there will come a time in the development of a muskeg area, when a gradient in the mineral soil terrain will be overcome by the peat factor. Major Liston requested amplification regarding the effect of temperature in peat formation. Dr. Radforth explained that if there is a geomorphological condition such that the temperature will cause depletion of water, peat cannot form. The temperature then becomes a primary factor. When the geomorphology is such that the water table will always be high, regardless of the temperature, then there will be peat accumulation. Mr. Hemstock wondered about the effect of temperature on peat decomposition. Dr. Radforth said that the rate of decomposition is higher for higher temperatures. Professor Goodman asked if it were possible to tell if a peat bog area was growing or if it was in a state of deterioration. Dr. Radforth said that it is not possible to do this (except by peat analysis) apart from setting up a controlled experiment. He suggested that if a controlled water level were to be established in a bog, careful records kept and the temperature controlled with the use of infra-red heat, then it would be possible to measure the rate of growth or of decay.

Professor Goodman inquired if peat would decompose very rapidly if it were buried under a fill, above the water table. Dr. Radforth answered that it would if dry, but not if wet. Exposure and aeration cause deterioration of peat. Mr. Fox commented that this is recognized by "muck" farmers who have to maintain a high water table to keep their farmland from deteriorating. Mr. Harwood remarked that he had heard of an unusual use for peat - feeding it to pigs. Dr. Radforth said he had never heard of this but since it is an organic substance, could see no reason why the living cover at least could not be milled and fed to livestock.

Mr. Schlosser asked for an explanation of the subsurface constitution of peat with relation to the life cycle of the surface vegetation. Dr. Radforth stated that in deposition of peat, one commonly thinks of one generation after another of vegetation being added. In northern areas this occurs very slowly but in time a layer of peat is built up. Dr. Radforth stressed, however, that there is no question of a succession of vegetal species moving toward a climax, a theory which has long since been discarded. He emphasized that there is a succession of one kind of vegetation throughout the whole depth of a muskeg and regretted the wrong impression sometimes given of a progressive series of vegetal types. There is, for instance, often "E" class at the very beginning. The deposition rate may be differential, however, so there will be an uneven surface as one travels over a muskeg from one type to another. With regard to the effects of a fire, Dr. Radforth acknowledged that the organization is destroyed and for the time being the species is different. Instead of a common height of vegetation as before, height of the new vegetation is very erratic. Dr. Radforth maintained, however, that in due course the original cover will grow back in. There will be a layer in the peat reflecting this change in the vegetation, but he thought that through the depth of peat it will not be unduly significant. The only time it may be important is when drainage is being considered. Mr. Hughes observed that he has come across a tamarack forest with trees 40 ft high and peat 9 ft deep. This seems to be an exception to the general rule. He also inquired about the effect of the water holding capacity of the moss. Dr. Radforth replied that if there is a high water regime and there is an "A" cover, then the chances are that it will become and remain tamarack and that the peat will be fairly deep and wet. With regard to the moss factor, the kind of moss governs the amount and length of time water can be held during a dry period. Concerning exceptions to the general rule, Dr. Radforth contended that there are exceptions to everything natural; there are always anomalies. He said, however, that in developing a classification system, certain fundamentals that recur must be used. Also, it is important to remember that discrete patterns of unconfined muskegs are telescoped in a confined muskeg.

Mr. Enright referred to the rapid deterioration of peat on side slopes of highway fills and wondered if this could not be arrested by immediate seeding with grass. Dr. Radforth did not think so, for there would actually be an increase in the drying-out factor for in the grass there is an increased facility for evapotranspiration. Mr. Enright asked if after a few years, once the grass roots had penetrated into the peat and perhaps to the water table, would this not rejuvenate the muskeg, or at least stabilize the road shoulder. Dr. Radforth doubted that the peat would be rejuvenated in any way, but thought that the grass roots would certainly strengthen the peat and thus contribute to the stability of the shoulder.

Mr. Harwood asked for Dr. Radforth's comments regarding the vegetative change in the construction of the Kapuskasing airstrip. Dr. Radforth pointed out that an airstrip was built on muskeg. The living cover was stripped off and FI cover established itself which is the normal condition. By now, however, the strip is reverting to its original cover, which will eventually become BEI over that area and the present tamarack will disappear. Mr. Harwood said that it is also interesting to know that the water table was at the surface of the muskeg adjacent to the airstrip, but was 14 to 20 in. below the surface of the FI cover of the strip.

Professor Goodman asked if a vehicle got bogged down and was left alone for a while, would there be a regain of strength in the peat to enable the vehicle to get out. Dr. Radforth replied that there would be some regain of strength, but certainly neither as much nor as quickly as for clay. Mr. Schlosser, speaking on the basis of his experience on seismic surveys, did not think there would be any regain of strength. He has used trails for several passes until vehicles have bogged down. These trails still could not be used two weeks later. Mr. Keeling remarked that after two years one can re-use old seismic lines which had an FI cover and a lowered water table. Also, a greater number of passes can be made than originally. However, he said that a long time must elapse before "chewed up" FI can be re-used. Mr. J. G. Thomson observed that there are new vehicles available which can travel over muskeg and not damage the mat, so there is really no need to get bogged down. Mr. Schlosser raised the question of articulated vehicles and wondered what Mr. Thomson's opinion was regarding them. Mr. Thomson said that the advantage of the articulated vehicle is simply a matter of geometry. A long vehicle has a much better chance of negotiating small inconsistencies in the muskeg. An articulated vehicle can be made to act as a single long vehicle, quite apart from the steering advantage. In the lock track type of steering, it is adding to the difficulties to speed up one track and to stop the other. He pointed out that the standard steering ratio is 1.4/1 or $\frac{2c}{2\ell} \leqq \frac{1.4}{1}$ where $2c$ = distance between tracks and 2ℓ = track length. Mr. Schlosser agreed that in his experience there is certainly not the tearing of the mat with an articulated vehicle.

CLOSING REMARKS

Dr. Radforth expressed the appreciation of the Muskeg Subcommittee for the hospitality of the University of Saskatchewan. He was grateful for the co-operation of the University authorities, particularly Dr. J. B. Mawdsley, Dean of Engineering. Dr. Radforth announced that the 1963 conference would, in all likelihood, be held in the Province of Quebec, either in Montreal or Quebec City. He also mentioned that a special regional conference is being planned to be held in the Atlantic provinces in September 1962.

APPENDIX A

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