DESIGN METHODS FOR MUSKEG AREA ROADS

FINAL REPORT

by

Dr. Arvind Phukan, P. E.
Associate Professor

Civil Engineering Department
University of Alaska
Fairbanks, Alaska 99701

February 1982

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES
DIVISION OF PLANNING AND PROGRAMMING
RESEARCH SECTION
2301 Peger Road
Fairbanks, Alaska 99701

in cooperation with

U. S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No. FHWA-AK-RD-83-14	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Design Methods for Muskeg Area Roads		5. Report Date February, 1982
		6. Performing Organization Code
7. Author (s) Dr. Arvind Phukan, P. E.		8. Performing Organization Report No.
9. Performing Organization Name and Address Associate Professor Civil Engineering Department University of Alaska Fairbanks, Alaska 99701		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. HPR F26462
12. Sponsoring Agency Name and Address Alaska Department of Transportation and Public Facilities Pouch Z Juneau, Alaska 99811		13. Type of Report and Period Covered Final
		14. Sponsoring Agency Code

15. Supplementary Notes

Conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration

16. Abstract

The objective of this report was to produce a design guide related to roadway design and construction in organic terrain referred as "muskeg". The report covers the latest design and construction principles so that the designers, planners, builders and operators can systematically analyze the available information, assemble required data and design roadways. General aspects of foundation stability, common types of problems which may develop and investigations required to evaluate problems are also outlined.

17. Key Words		18. Distribution Statement No Restrictions	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 106	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

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1. INTRODUCTION

1.1 Scope

Design, construction and maintenance of roadway on highly compressible muskeg is unique to various parameters, and careful design procedures are required for the long term performance to avoid costly maintenance as well as foundation stability of the roadway section. In Alaska, where approximately 80% of the land mass consists of permafrost, the frozen ground conditions may further complicate the design considerations.

Recently, numerous design conceptions and construction methods are used to build roadway on muskeg, and a wide range of data has been published on the performance of roadway on muskeg. A design manual to include all aspects of design and construction of roadway on muskeg is not readily available and such a manual may bridge the gap between the latest works and the design principles used previously. It is expected, therefore, the design manual will play an important role to minimize the cost of construction and maintenance of roadways on muskeg. This manual is primarily concerned with the design of roadway sections, consisting of base and subbase layers with fill materials below the surface course of pavement.

Roadway and muskeg terminology to be used is detailed in the glossary of terms presented at the back of this manual.

The objective of this manual is to produce a design guide related to roadway design and construction in organic terrain referred as "muskeg". The manual covers the latest design and construction principles so that the designers, planners, builders and operators can systematically analyze the available information, assemble required data and design roadways. General aspects of foundation stability, common types of problems which may develop and investigations required to evaluate problems are also outlined. The essential topics included are discussed under the following headings:

(i) Exploration of Data
(ii) Evaluations of Various Design Methods and Their Relative Usefulness
(iii) Construction Techniques, Problems and Control
(iv) Cost Evaluations
(v) Recommendations.

Particular emphasis has been placed on designing the supporting fill thickness against the differential settlement and consolidation of highly compressible muskeg and on construction techniques to place "fill" materials in the organic terrain. New construction techniques are also outlined and the future performance studies on these new construction methods will dictate the required improvements of design procedures.

1.2 Summary

In North America where the predominance of muskeg is in Alaska and subarctic Canada, the development of potential resources and the economic development have led to an awareness of the problems encountered in the construction of roads on muskeg or peat deposits. The term "muskeg" is derived from the North American Indian language and it is a distinct terrain type characterized by living and decomposed plants.

The properties of peat are considerably variable, differing markedly from one deposit to another. These variations in properties together with variations in the peat thickness make design and construction in peat particularly difficult. The classification of muskeg, the basic physical, mechanical, chemical and thermal properties of peat with their laboratory and insitu determination, which are essential to design, are discussed in Section 2.

Three basic approaches may be taken in the design of roadways on muskeg. Engineering design that avoids the muskeg areas by judicious site locations, will reduce muskeg construction problems. Most construction projects are primarily involved with either removal of the organic material and replacing it with suitable fill or building on it, relying on improved strength to support the structure. Design techniques include adequate shear strength and bearing capacity, knowledge of consolidation characteristics and stability analysis. These design considerations are described in Section 3.

Factors that influence the construction methods over organic terrain are the characteristics of peat and the underlying soil, peat thickness, availability of construction materials, equipment required, available time, location of structures and drainage requirements. Section 4 includes the various aspects of construction of roadways on muskeg.

Construction costs including design and engineering, construction and maintenance are discussed in Section 5.

Recommendations regarding the future needs are given in Section 6. These include the determination of peat characteristics, its insitu strength and consolidation characteristics, methods of improved construction and stability of fill sections.

2.0 EXPLORATION OF DATA

2.1 General

This section discusses the muskeg classification system, the general influence of physical and mechanical properties, thermal and chemical properties. Engineering laboratory investigations and field techniques which are significant to design are also described. Typical ranges of muskeg properties can be seen from the included data that characteristic values can vary over wide ranges depending on various parameters such as muskeg type, water content, mineral soil content, compressibility, shear strength and so on. It is important to require data representative of the actual subsurface conditions for the final roadway design on muskeg.

2.2 Muskeg Classification System

Von Post and Radforth classification systems are most commonly used to classify peat. The Von Post Classification System is based on the degree of composition or humification, indicated by the letter "H", and are divided into 10 categories (Ref. Table 2-1).

A modified Von Post classification system was suggested by Korpyaakko and Woolnough (1) and the following items are to be recorded in conjunction with Table 2.1:

1. The peat type, utilizing the recognizable features of the original plant constituents that formed the peat. The most common plants forming peat the the symbol used for them in the peat formula are: Sphagnum (S), Carex (C), Eriophorum (Er), Equisetum (Eq), Phragmites (Ph), Scheuchzeria (Sch), Shrubs (N), wood (L) and mosses (other than Sphagna, i.e., Bryales),(B).

DEGREE OF DECOMPOSITION von POST'S SCALE	INFORMATION FOR IDENTIFICATION
H_1	Completely unconverted and mud-free peat which, when pressed in the hand, only gives off clear water.
H_2	Practically completely unconverted and mud-free peat which, when pressed in the hand, gives off almost clear colourless water.
H_3	Little converted or very slightly muddy peat which, when pressed in the hand, gives off marked muddy water, but no peat substance passes through the fingers. The pressed residue is not thick.
H_4	Badly converted or somewhat muddy peat which, when pressed in the hand, gives off marked muddy water. The pressed residue is somewhat thick.
H_5	Fairly converted or rather muddy peat. Growth structure quite evident but somewhat obliterated. Some peat substance passes through the fingers when pressed but mostly muddy water. The pressed residue is very thick.
H_6	Fairly converted or rather muddy peat with indistinct growth structure. When pressed at most 1/3 of the peat substance passes through the fingers. The remainder extremely thick but with more obvious growth structure than in the case of unpressed peat.
H_7	Fairly well converted or marked muddy peat but the growth structure can still be seen. When pressed, about half the peat substance passes through the fingers. If water is also given off, this has the nature of porridge.
H_8	Well converted or very muddy peat with very indistinct growth structure. When pressed, about 2/3 of the peat substance passes through the fingers and at times a somewhat porridgy liquid. The remainder consists mainly of more resistant fibres and roots.
H_9	Practically completely converted or almost mudlike peat in which almost no growth structure is evident. Almost all the peat substance passes through the fingers as a homogeneous porridge when pressed.
H_{10}	Completely converted or absolutely muddy peat where no growth structure can be seen. The entire peat substance passes through the fingers when pressed.

Table 2-1 Degree of decomposition after von Post (from Fl'aate, 1965)

2. The moisture regime of peat sample which is estimated by utilizing a scale of 1-5 and the symbol "B"

B_1 = dry peat,
B_2 = low moisture content,
B_3 = moderate moisture content
B_4 = high moisture content,
B_5 = high moisture content.

3. The fire fibre content, estimated and expressed by utilizing a scale of 0-3 and the symbol "F". The fibres are, in this case, mainly derived from Eriophorum (Er)

F_0 = nil, F_1 = low content, F_2 = moderate content, F_3 = high content.

The coarse fibres are also estimated by using a scale of 0-3 and the symbol "R" in the manner described above.

4. The presence of woody remnants in the peat, using a scale of 0-3 and the symbol "W" are used

W_0 = nil, W_1 = low content, W_2 = moderate content, W_3 = high content.

5. The depth from which the sample has been extracted and this information is generally given at the beginning of the formula.

6. Other pertinent information is added to the end of the formula using suitable symbols and abbreviations.

According to Radford Muskeg Classification System, surface vegetation, topographic features and subsurface characteristics are described. The description of the surface vegetation is based on qualities of vegetation such as stature, degree of woodiness, external texture and certain easily recognized growth habits. Nine pure vegetation classes are recognized (Ref. Table 2.2). If one coverage class is not present to

Table 2-2. Summary of properties designating nine pure coverage classes. (Radforth classification).

Coverage type (class)	Woodiness vs non-woodiness	Stature (approx. height)	Texture (where reg'd)	Growth habit	Example
A	Woody	15 ft or over	-	Tree form	Spruce Larch
B	Woody	5 to 15 ft	-	Young or dwarfed tree or bush	Spruce Larch Willow Birch
C	Non-woody	2 to 5 ft	-	Tall, grass-like	Grasses
D	Woody	2 to 5 ft	-	Tall shrub or very dwarfed tree	Willow Birch Labrador tea
E	Woody	up to 2 ft	-	Low shrub	Blueberry Laurel
F	Non-woody	up to 2 ft	-	Mats, clumps or patches, sometimes touching	Sedges Grasses
G	Non-woody	up to 2 ft	-	Singly or in loose association	Orchid Pitcher plant
H	Non-woody	up to 4 in	Leathery to crisp	Mostly continuous mats	Lichens
I	Non-woody	up to 4 in	Soft or velvety	Often continuous mats, sometimes in hummocks	Mosses

(After Radforth, 1969)

TABLE 2.3 TOPOGRAPHIC FEATURES
(Radforth classification)

Contour type	Feature	Description
a	Hummock	Includes "tussock" and "nigger-head". Has tufted top, usually vertical sides. Occurs in patches, several to numerous.
b	Mound	Rounded top. Often elliptic or crescent-shaped in plan view.
c	Ridge	Similar to Mound but extended. Often irregular and numerous. Vegetation often coarser on one side.
d	Rock gravel plain	Extensive exposed areas.
e	Gravel bar	Eskers and old beaches (elevated).
f	Rock enclosure	Grouped boulders overgrown with organic deposit.
g	Exposed boulder	Visible boulder interrupting organic deposit.
h	Hidden boulder	Single boulder overgrown with organic deposit.
i	Peat plateau (even)	Usually extensive and involving sudden elevation.
j	Peat plateau (irregular)	Often wooded, localized and much contorted.
k	Closed pond	Filled with organic debris, often with living coverage.
l	Opened pond	Water rises above organic debris.
m	Pond or lake margin	(Abrupt).
n	Pond or lake margin	(Sloped).
o	Free polygon	Forms a rimmed depression.
p	Joined polygon	Formed by a system of banked clefts in the organic deposit.

Table 2-4 Subsurface constitution.

Predominant characteristic	Category	Name
Amorphous-granular	1.	Amorphous-granular peat
	2.	Non-woody, fine-fibrous peat
	3.	Amorphous-granular peat containing non-woody fine fibers
	4.	Amorphous-granular peat containing woody fine fibers
	5.	Peat, predominantly amorphous-granular, containing non-woody fine fibers, held in a woody, fine-fibrous framework
	6.	Peat, predominantly amorphous-granular, containing woody fine fibers, held in a woody, coarse-fibrous framework
	7.	Alternate layering of non-woody, fine-fibrous peat and amorphous-granular peat containing non-woody fine fibers
Fine-fibrous	8.	Non-woody, fine-fibrous peat containing a mound of coarse fibers
	9.	Woody, fine-fibrous peat held in a woody, coarse-fibrous framework
	10.	Woody particles held in non-woody, fine-fibrous peat
	11.	Woody and non-woody particles held in fine-fibrous peat
Coarse-fibrous	12.	Woody, coarse-fibrous peat
	13.	Coarse fibers criss-crossing fine-fibrous peat
	14.	Non-woody and woody fine-fibrous peat held in a coarse-fibrous framework
	15.	Woody mesh of fibers and particles enclosing amorphous-granular peat containing fine fibers
	16.	Woody, coarse-fibrous peat containing scattered woody chunks

the extent of 25% of the terrain, it is not included in the composite cover description. The most prominent class type is placed first and others follow in order of prominence. Table 2.3 gives the descriptive information for identifying topographic features of organic terrain. Based on the extent to which wood and fibres are present in muskeg, Radforth established 16 categories of peat (Ref. Table 2.4). Organic material is approximately grouped into three basic types:

1. Material composed chiefly of soils of an amorphous-granular base
2. Material chiefly made up of fine fibres, and
3. Material predominantly of wood particles and course fibres.

By the use of plasticity chart (Ref. Fig. 2-1), the basic classification of peat may be determined.

2.3 Engineering Characteristics of Peat

These can be subdivided into the following catagories:

(a) Physical or index properties (b) Mechanical properties
(c) Thermal properties
(d) Chemical properties.

2.3.1 Physical or Index Properties

The physical properties of peat which are of prime importance to engineering application are: void ratio, water content, specific gravity, organic content, fiber content and ash content. Various investigators have published data on the physical properties of peat. Figs. 2.1 through 2.8 give a wide range of values for these properties.

The relationships between water content, void ratio and specific gravity are shown in Fig. 2.2 to 2.4(a). Figs. 2.5 and 2.6 present the correlation between water content, organic content, density and specific gravity of peat. The unique relationship between specific gravity and ignition loss which reflects the organic content, is shown in Fig. 2.7. Fig. 2.8 illustrates the relationship between fiber content, ash content and density of peat.

2.3.2 Mechanical Properties

The mechanical properties of peat vary widely depending on the nature and composition of peat, water content, degree of decomposition, degree of consistency and so on. In most engineering design and construction, the principal requirements can be grouped into the following properties:

(a) Compressibility characteristics including settlement and consolidation
(b) Shear Strength of peat
(c) Hydraulic conductivity or permeability

Compressibility

Compressibility of peat and organic soils may result from vetical deformation, volume change and shear strain of lateral displacement. One of the main differences in the compression of peat and organic soils as compared to organic soils is the long term compression which appears to be an almost continuous process. Because of the high compressibility and low strength of peat, its use as foundation material has been limited to the support of low embankments, principally in roadway construction. However, recent years, with the use of preloading techniques, the support of major highways has been possible on an organic foundation.

Properties that influence the compressibility of peat are the water content, void ratio, peat type, mineral soil content and permeability. Figs. 2.9 and 2.10 show the relationships between compression index (C_c) void ratio, water content and peat type (Von Post scale). Typical consolidation test results are presented in Fig. 2.11 and 2.12. Fig. 2.13 illustrates a typical relationship between the coefficient of compressibility and void ratio.

Many investigators (3,4,5,6,7,) have reported the compressibility characteristics and consolidation of peat. Based on both laboratory and field observations, settlement and consolidation characteristics of peat may be obtained. Generally the conventional theory of consolidation cannot be applied to peat due to its high compressibility and change of permeability with time which accounts for the significant differences in consolidation behavior between organic and mineral soils.

Shear Strength

The stability of road embankment and bearing capacity of soils, including peat, depends on the shear strength characteristics. For the last two decades, extensive data has been accumulated from both laboratory and field tests which show that the conventional theory of soil mechanics shear strength principles are also applicable to peat soils, but some differences do exist.

Fig. 2.14 shows that the increase in shear strength of peat with depths. Generally, the shear strength of peat varies inversely with water content and directly with ash content. All published information (8,9,10,11,12) indicates that peat soils show significant rise in shear strength as they consolidate. As shown in Figs. 2-15 and 2-16, the stress-strain relationship of peat may be also used to determine the shear strength of peat soils.

The basic shear strength equation is:

$$S = C + \sigma \tan \phi \quad \text{(in terms of total stress)} \qquad (2.1)$$

$$\text{or} \quad C' + \sigma' \tan \phi' \quad \text{(in terms of effective stress)} \qquad (2.2)$$

The angle of internal friction, both Ø and Ø' terms are relatively high as compared to those for inorganic soils.

Permeability

The coefficient of permeability (K) of peat is generally reduced with compressibility and increased load. Due to wide variations of K, it is very difficult to assign particular range of values for different peaty soils. Some investigations (13,14) found no relationship between void ratio and coefficient of permeability (K) (Ref. Fig. 2.17).

2.3.3. Chemical Properties

The chemical properties of peat are affected by gas content and acidity. Usually peat has an acid reaction which is caused by the presence of carbon dioxide and humic acid arising from its decay. Peat generally has acidity PH of 4-8. That acidity is decreased as the organic content or fibre content increase (Ref. Fig. 2-18 and 2-19). The nitrogen content (ash free basis) increases with increased degree of decomposition. This chemical property is important because peats can be potentially corrosive to concrete or steel structures.

2.3.4 Thermal Properties

An organic mantle on the ground surface forms a natural insulator for permafrost from the thawing effects of the summer and reduces the frost penetration in the winter. This insulating value has been recognized for many years due to abnormally high water retention capacity of peat and the varying conductivity of peat in the thawed and frozen states.

The thermal properties such as thermal conductivity, volumetric heat capacity, latent heat and thermal diffusivity that have great influence in heat transfer in a soil, are essential to determine the depth of thawing or freezing. This is especially important in the permafrost regions for the design of embankment on frozen muskeg.

The heat conductivity of peat will vary between wide limits depending upon the amount of moisture, compaction and whether or not the material is frozen. Value of thermal conductivity of peat in the frozen and unfrozen states may be obtained from Fig. 2-30.

The specific heat per unit volume of soil can be found by adding the heat capacities of volume fractions of solid material, water (or ice) and air. So the volumetric heat capacity, (C) is given by: (neglecting specific heat of air which is very small)

$$C = \gamma_d \; X_s \, C_s + X_w \, C_w \tag{2.3}$$

γ_d = dry density
X_s = solid fraction
X_w = water fraction
C_s = specific heat of solid constituents = 0·6 $\mathrm{Cal/}_{\mathrm{cu.cm.°C}}$
C_w = specific heat of water = 1 $\mathrm{Gal/}_{\mathrm{cu.cm.°C}}$

MacFarlane (2) presented some values of volumetric heat capacity of peat (Ref. Table 2.5). This Table gives also the calculated value of C for frozen peat (using C_{ice} = 0·45 $\mathrm{Cal/}_{\mathrm{cu.cm.°C}}$)..

TABLE 2.5 CALCULATED VALUE OF C

X_w		0·5	0·6	0·7	0·8	0·9
X_s = 0·10	Unfrozen	0·56	0·66	0·76	0·86	0·96
	Frozen	0·29	0·33	0·37	0·42	0·46
X_s = 0·20	Unfrozen	0·62	0·72	0·82	0·92	1·02
	Frozen	0·34	0·38	0·42	0·46	0·51
X_s 0·30	Unfrozen	0·68	0·78	0·88	0·98	1·08
	Frozen	0·39	0·44	0·48	0·52	0·56

The volumetric latent heat of fusion (L) is calculated by:

$$L = 1.44\ \gamma_d.\ W \text{ or } 79.7.\gamma_d.W \text{ But/cu ft or Cal/}_{\text{cu.cm.}}$$

$$\text{or} \quad L = (79.9)\left(\frac{W_s}{V}\right)\left(\frac{W_w}{W_s}\right) \text{ in Cal/}_{\text{cu.cm.}}$$

$$= (79.9)\ (X_w)\ \text{Cal/}_{\text{cu.cm.}} \qquad (2.4)$$

if the water fraction (X_w) is known L can be calculated.

The thermal diffusivity (a) is given by the ratio of thermal conductivity and volumetric heat capacity. The thermal diffusivity (Sq. cm. per sec. or sq. ft. per hr.) of the frozen soil is 8-10 times that of the unfrozen soil.

2.4 Engineering Investigations

Engineering investigations are required before the planning of roadways and the design can be finalized after adequate procurement of data related to terrain characteristics. The type of investigation involved in terrain analysis generally consists of air-photo analysis of surface and subsurface analysis of insitu peat property measurements and sampling for laboratory testing. Table 2.6 tests the terrain features and corresponding methods by which individual features can be detected, observed, measured or otherwise treated.

TABLE 2.6 TERRAIN ANALYSIS
(After Radforth (2))

TERRAIN FEATURES		METHOD OF ANALYSIS
i)	Vegetal Cover	Direct observation, Cover classification
ii)	Surface Profile (Micro topography)	Surveying
iii)	Peat Depth	Subsoil investigation
iv)	Mineral Sublayer	Subsoil investigation
v)	Peat Structure	Visual classification, Von Post system
vi)	Peat Density	Standard test (Nuclear density)
vii)	Shear Strength	Shear Vane, Cone Penetrometer
viii)	Peat Water Content	Sampling, Nuclear Moisture Meter
ix)	Peat Temperature	Thermistor or probe
x)	Drainage Pattern	Air-photo inperpretation
xi)	Permeability	Field or lab. measurement
xii)	Water Acidity (PH)	Sampling, PH meter
xiii)	Permafrost	Air-photo interpretation, ground truth
xiv)	Frequency of Open Water	Air-photo interpretation

2.4.1 Air-Photo Analysis

This method has been used extensively in recent years to detect the distribution of organic terrain on surface. Specifically, the recognition of aerial photographs of permafrost occurance in muskeg is possible (14,15,16).

Radforth (2) described various "airform patterns" which may be viewed from low altitude (<5000 ft.) and high altitute (10,000-30,000) air photographs. These airform patterns may be interpretated to identify terrain features. Brown (14) presented the most distinctive permafrost features in muskeg. It is to be recognized that experience is essential before any reliable correlation can be made.

2.4.2 Laboratory and Field Investigations

Both testings can include measurements of water content, density, permeability, shear strength and consolidation characteristics. Laboratory testing is limited by the recovery of representative undisturbed samples. A commonly used peat sampler is shown in Fig. 2.21. Other samplers such as split spoon, shelby tube and air piston sampler may be used depending on the type of peat encountered at the site.

The shear testing of peats presents many problems depending on the nature of material, presence of woody erratics and high compressibility. Many investigators reported the laboratory results of peat soils and some of these important data are presented in Figs. 2.1 through 2.20.

<u>In Situ Tests</u>

The most common in situ measurement of peat is the shear strength by the use of vane tester and cone penetration tests. Some typical results by vane tests are shown in Fig. 2.22 and 2.23. The torque readings and vane blade dimensions are used to calculate shear strength of peat and is given by the formula:

$$\tau = \frac{3T}{2\pi(r^2)(2r + 3H)} \qquad (2.5)$$

where τ is the shear strength, T the torque applied, r the radius of shear vane and H the height of shear vane.

The main factors that influence vane shear test results are peat type, water content and the test procedure. Based on various data, it appears that the shear strength of amorphous-granular and non-woody fibrous peats can be measured by vane tests with accuracy.

The Waterways Experimental Station, U.S. Army Corps of Engineers, Vicksburg, has developed a cone penetrometer which can be used to determine the "cone index" at varying depths and thereby, the shear strength and the bearing capacity of peat can be determined. Typical cone penetrometer test results are shown in Fig. 2.24. A good correlation generally exists between the cone penetrometer (P) and shear strength (T) [$T = \frac{1}{10} P$].

In situ peat density can be accomplished by using a nuclear (gamma) density meter.

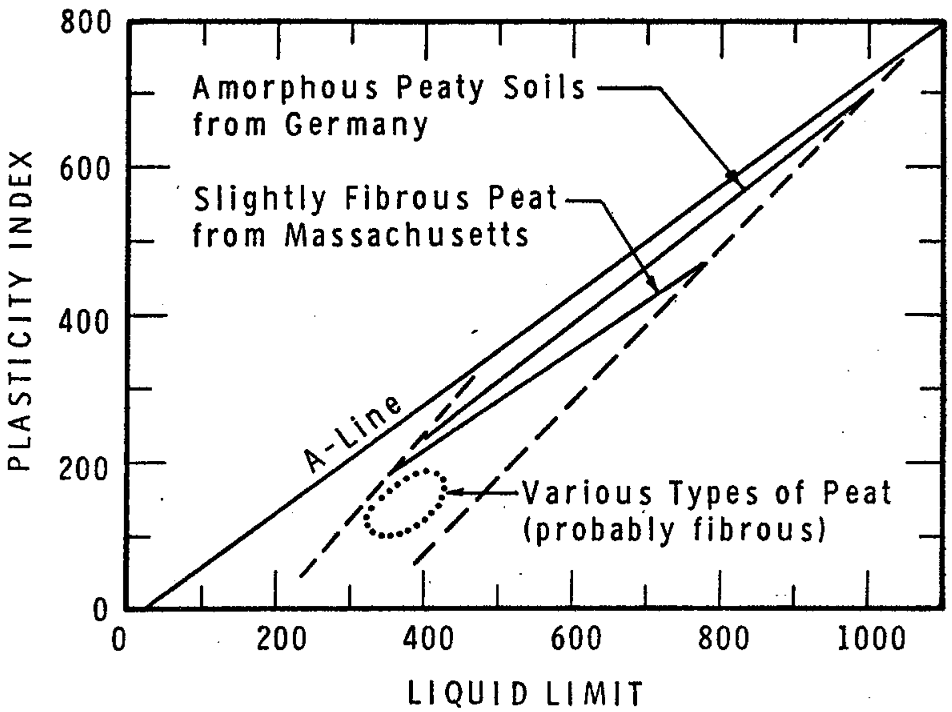
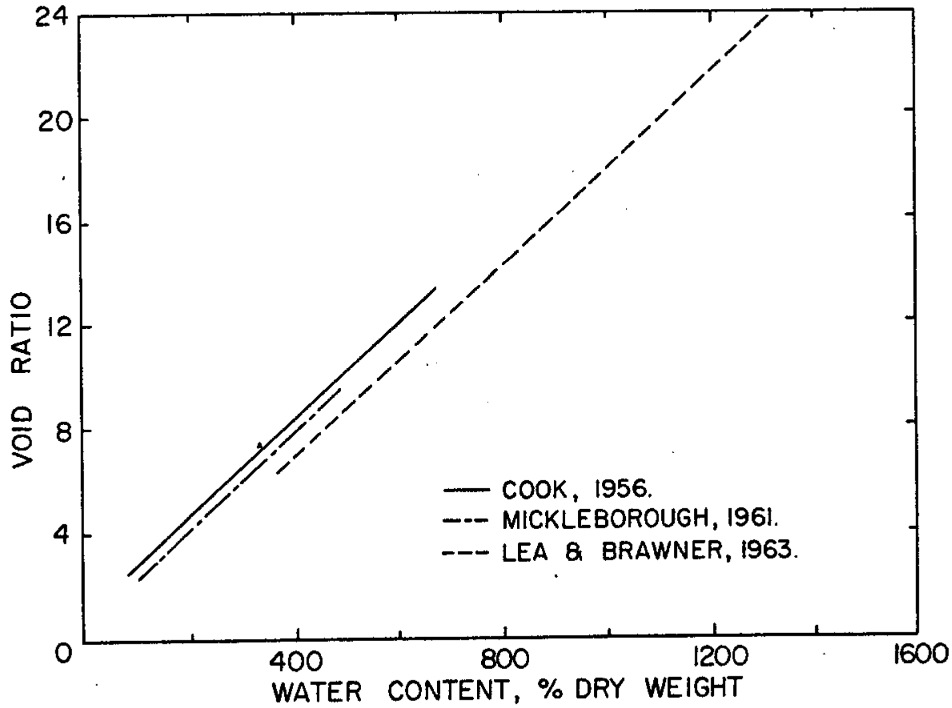


FIGURE 2.1 Plasticity chart for peat (after Casagrande 1966)



24
20
16
12
8
4
0
VOID RATIO
400
800
1200
1600
WATER CONTENT, % DRY WEIGHT
COOK, 1956.
MICKLEBOROUGH, 1961.
LEA & BRAWNER, 1963.

Figure 2.2

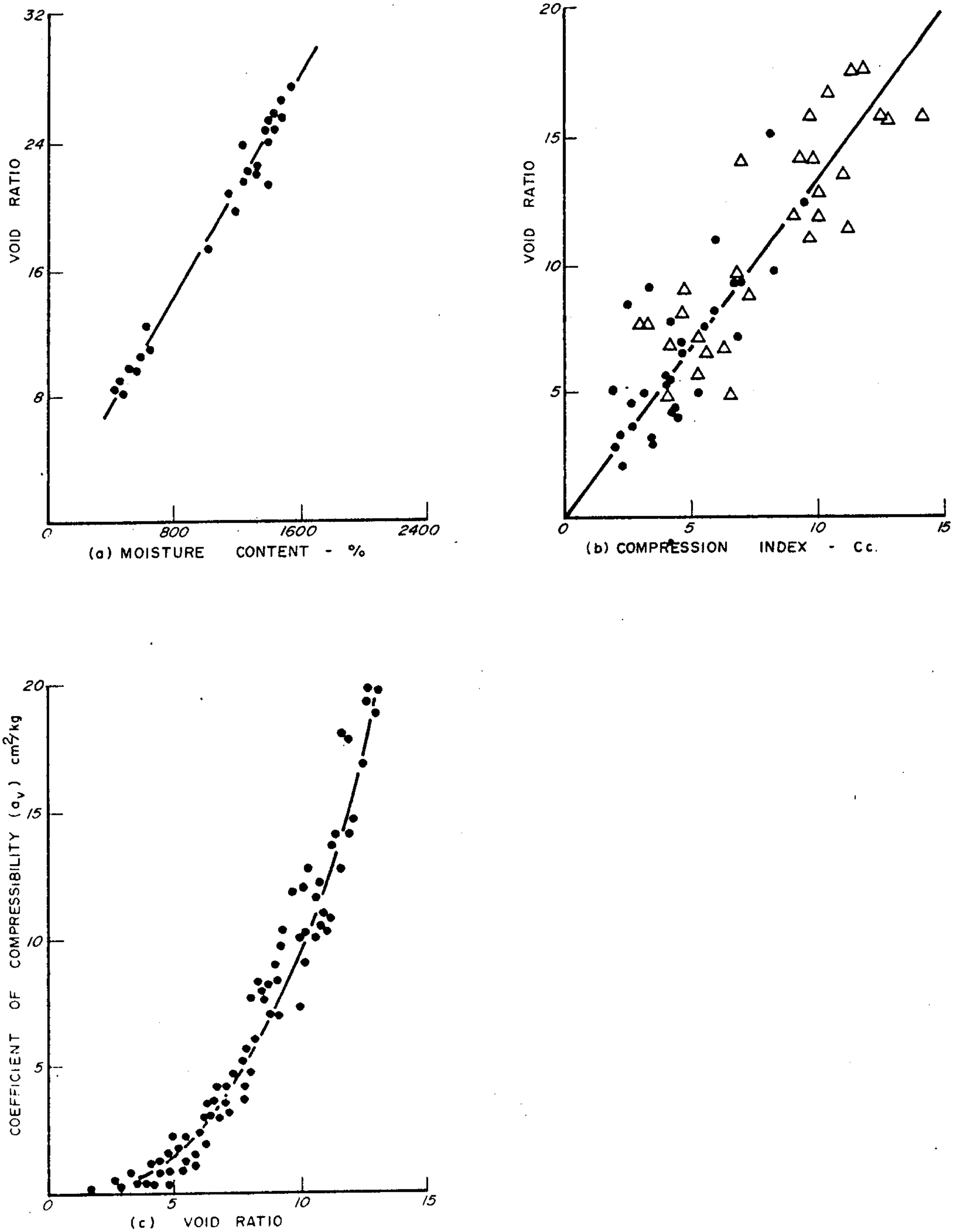
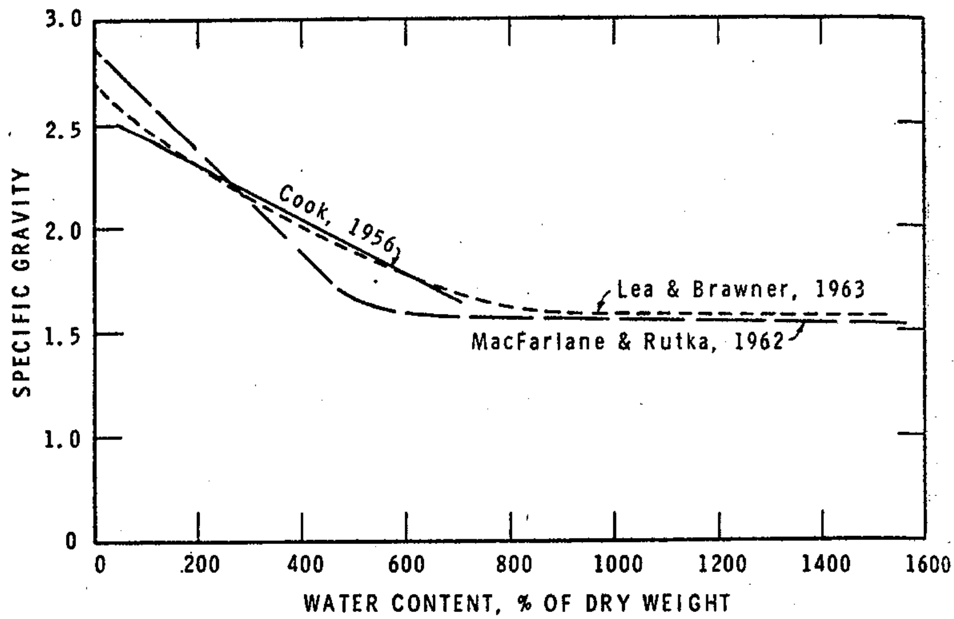

32
24
16
8
0
800
1600
2400
VOID RATIO
(a) MOISTURE CONTENT - %
20
15
10
5
0
5
10
15
VOID RATIO
(b) COMPRESSION INDEX - Cc.
20
15
10
5
0
5
10
15
COEFFICIENT OF COMPRESSIBILITY (a_v) cm^2/kg
(c) VOID RATIO

Figure 2.3



Specific gravity vs. water content

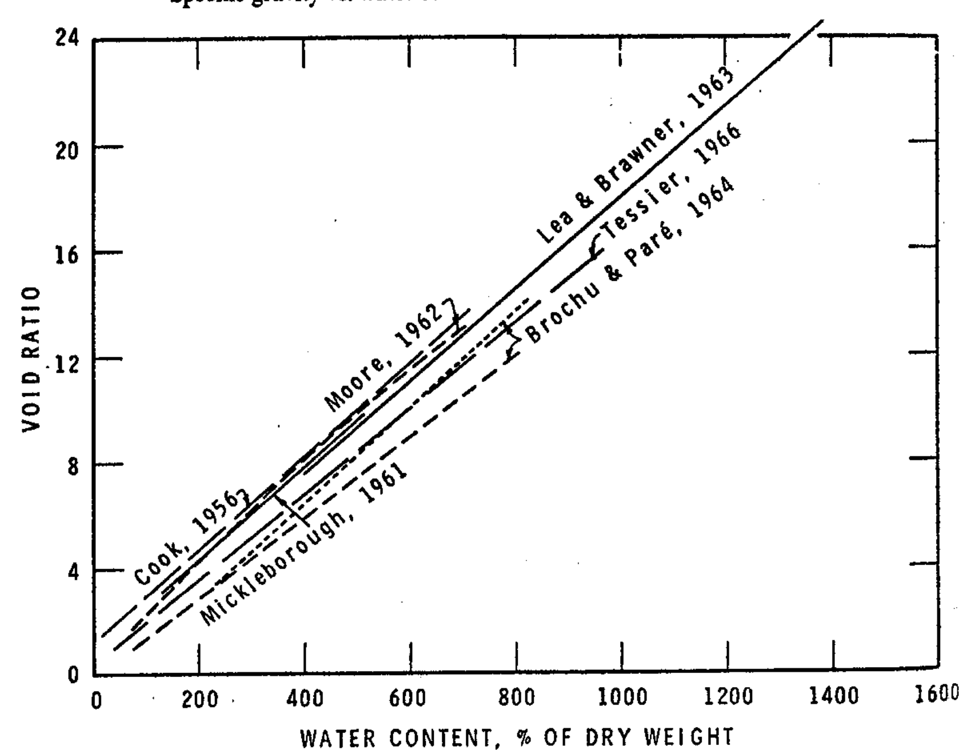
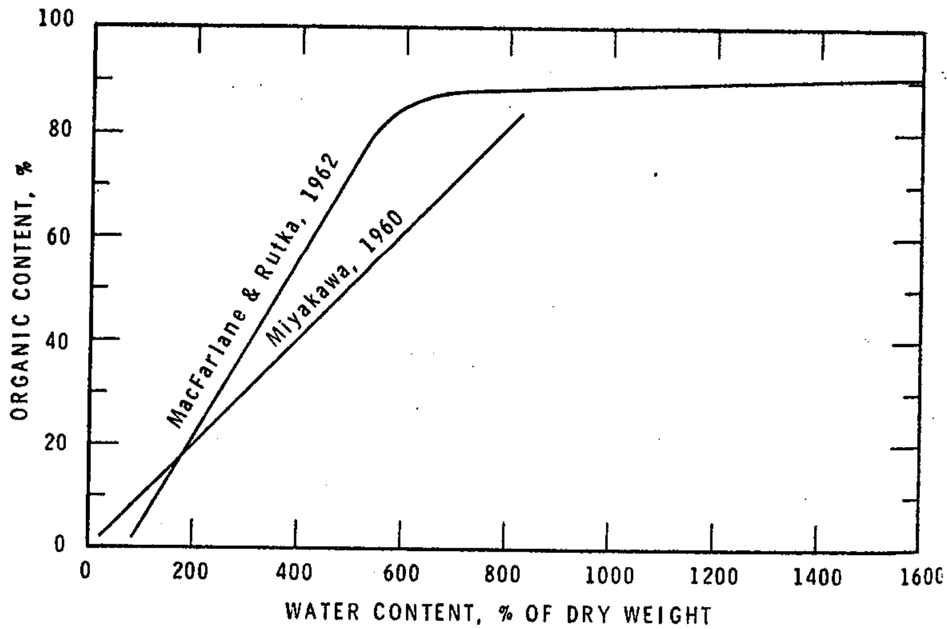
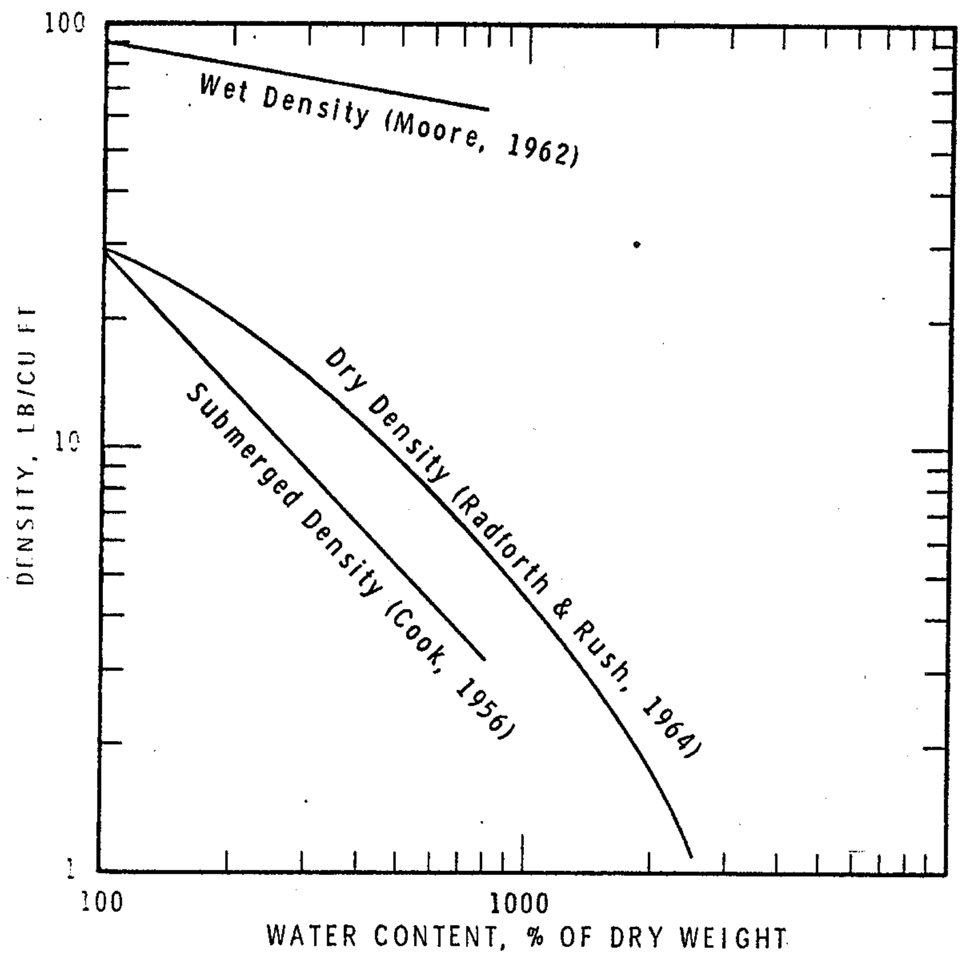


Figure 2.4

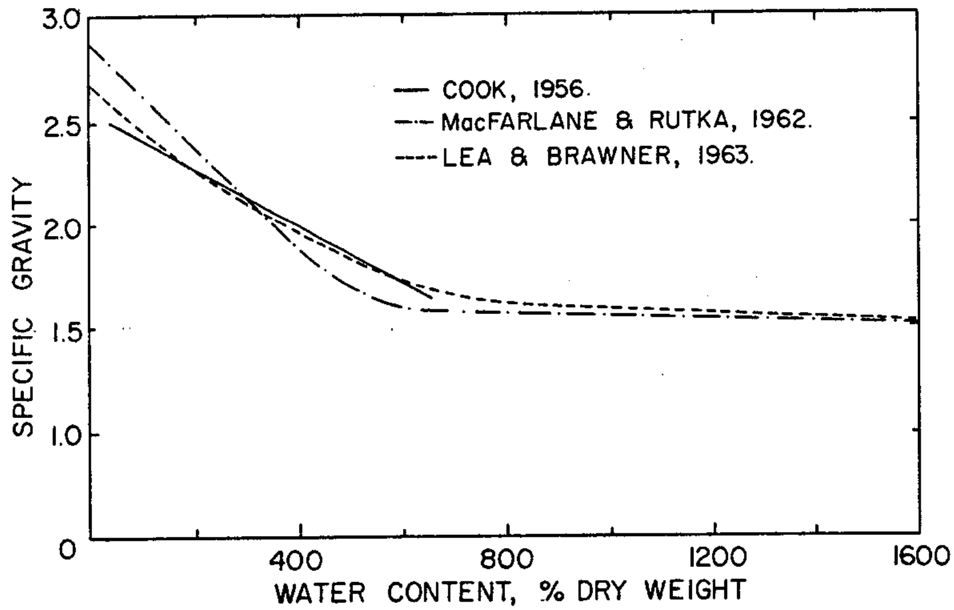


Organic content vs. water content



Density vs. water content

Figure 2.5



3.0
2.5
2.0
1.5
1.0
0
SPECIFIC GRAVITY
400
800
1200
1600
WATER CONTENT, % DRY WEIGHT
COOK, 1956.
MacFARLANE & RUTKA, 1962.
LEA & BRAWNER, 1963.

Figure 2.6

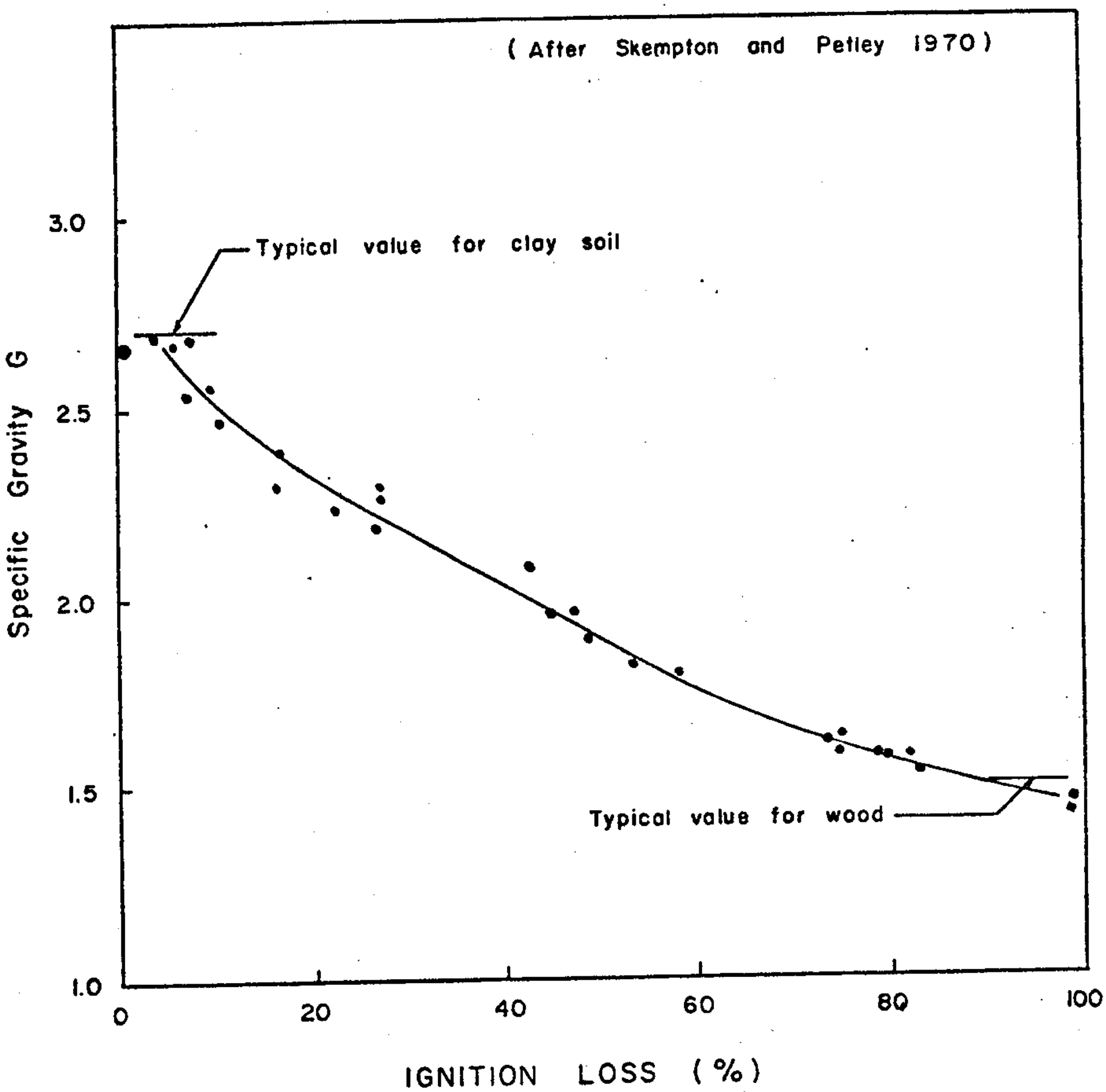


Fig. Relation between ignition loss and specific gravity of peat substance dried at 105 ° C

Figure 2.7

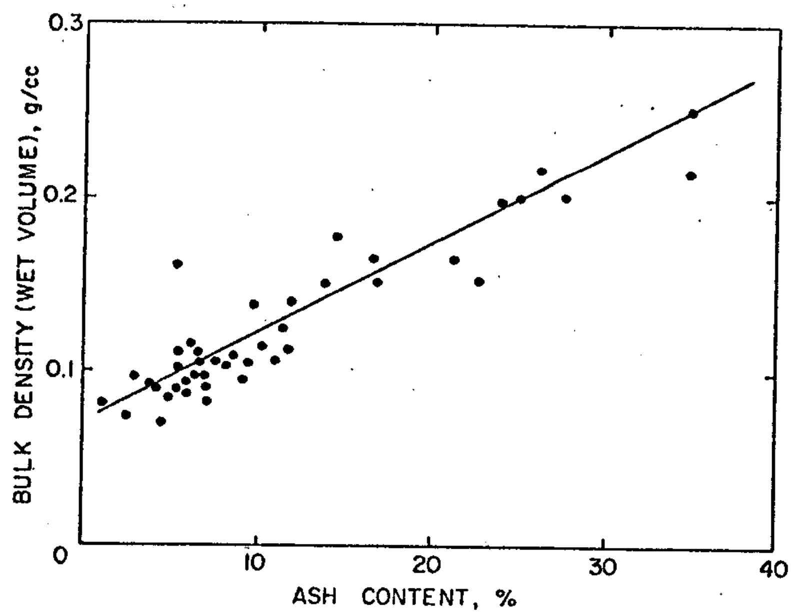


FIGURE The relationship between bulk density and ash content (after Irwin, 1966).

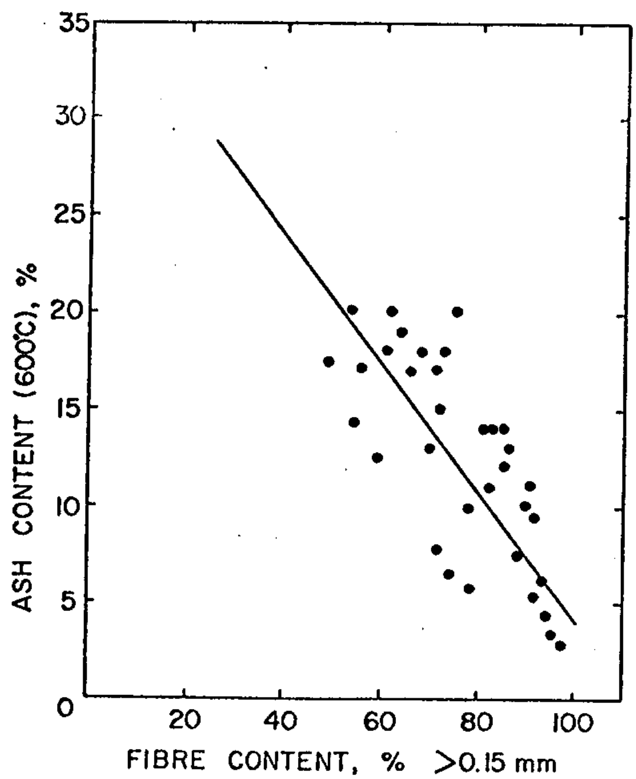
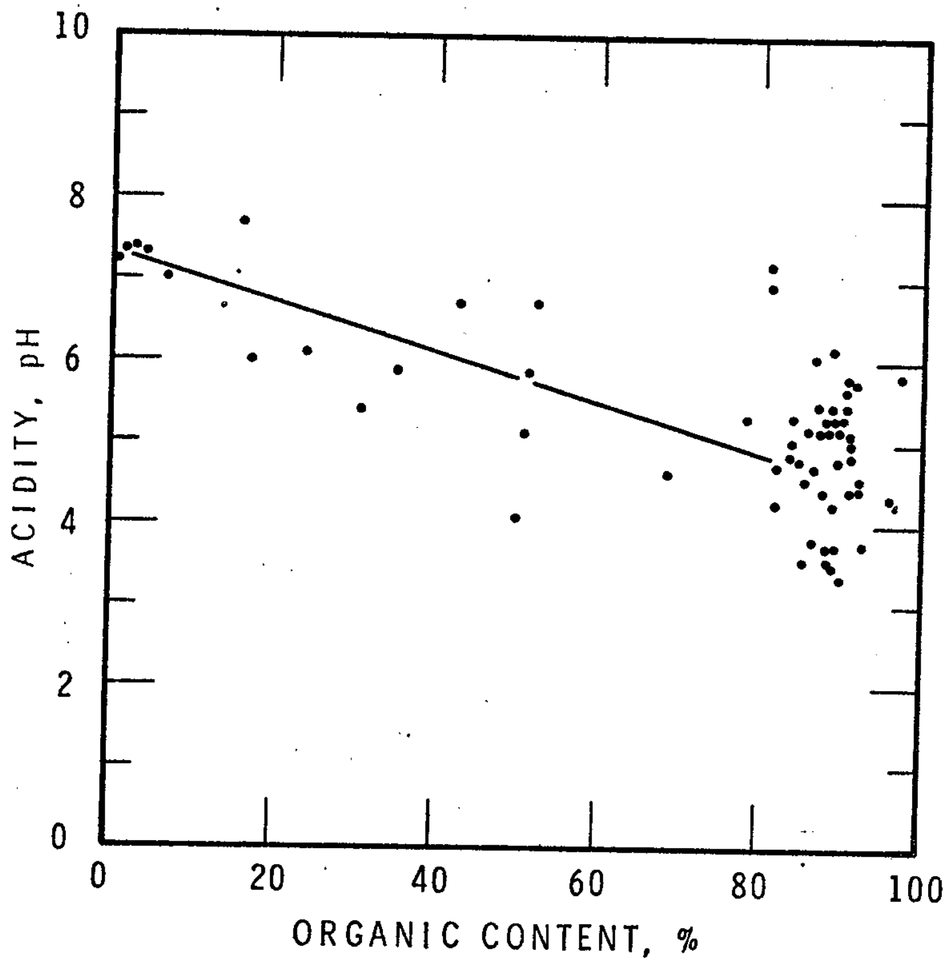


FIGURE 2.8 The relationship between ash content and fibre content for moss-dominated peats (after Padbury, 1970).



Acidity (pH) vs. organic content (after MacFarlane and Rutka 1962)

Figure 2.9

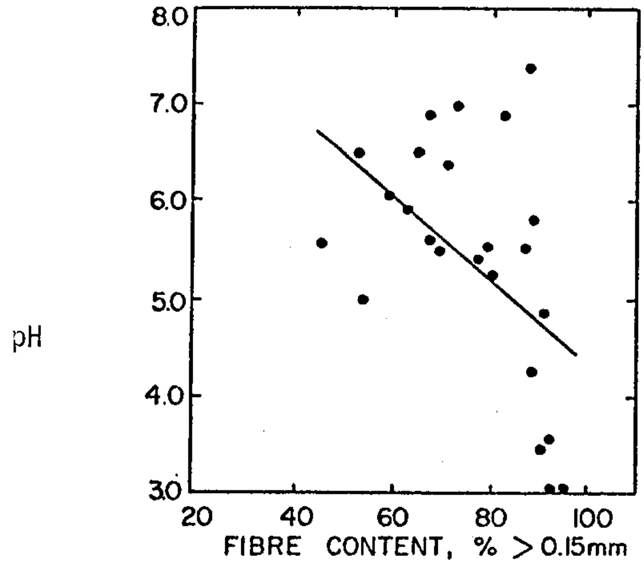


FIGURE The relationship between pH and fibre content (after Padbury, 1970).

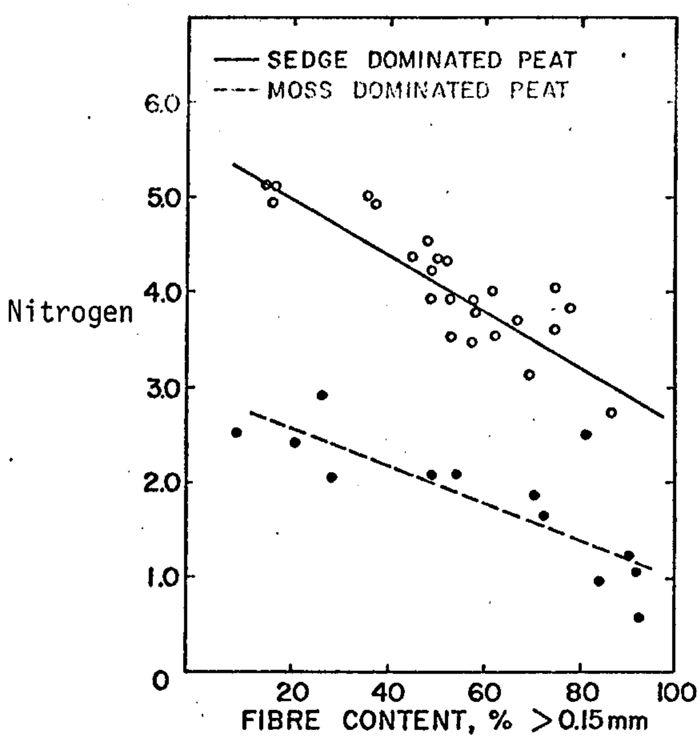


FIGURE The relationship between total nitrogen content (ash-free) and fibre content (after Padbury, 1970).

Figure 2.10

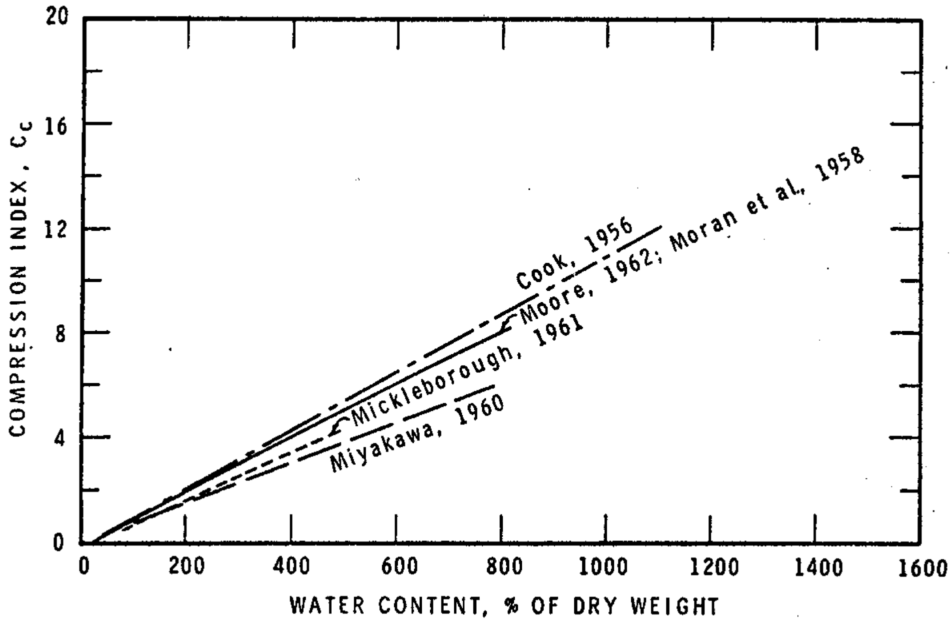


FIGURE Compression index (C_c) vs. water content

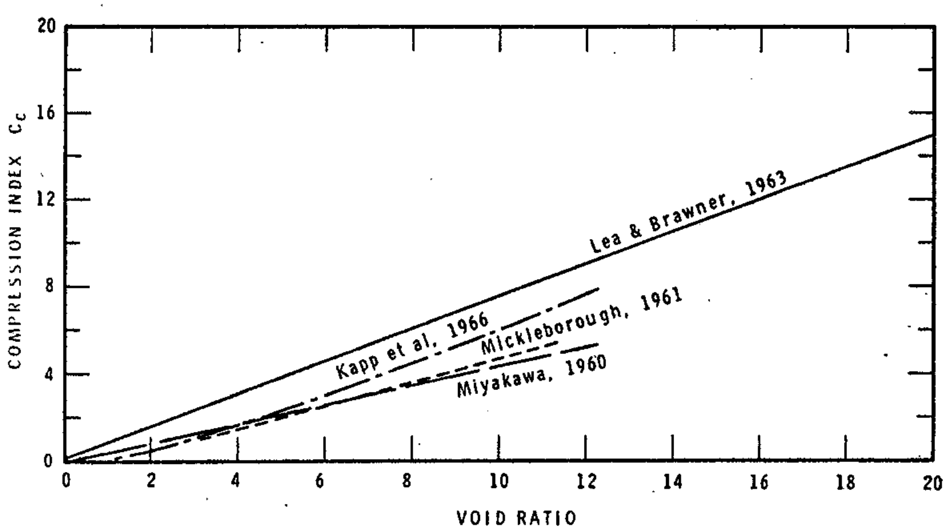
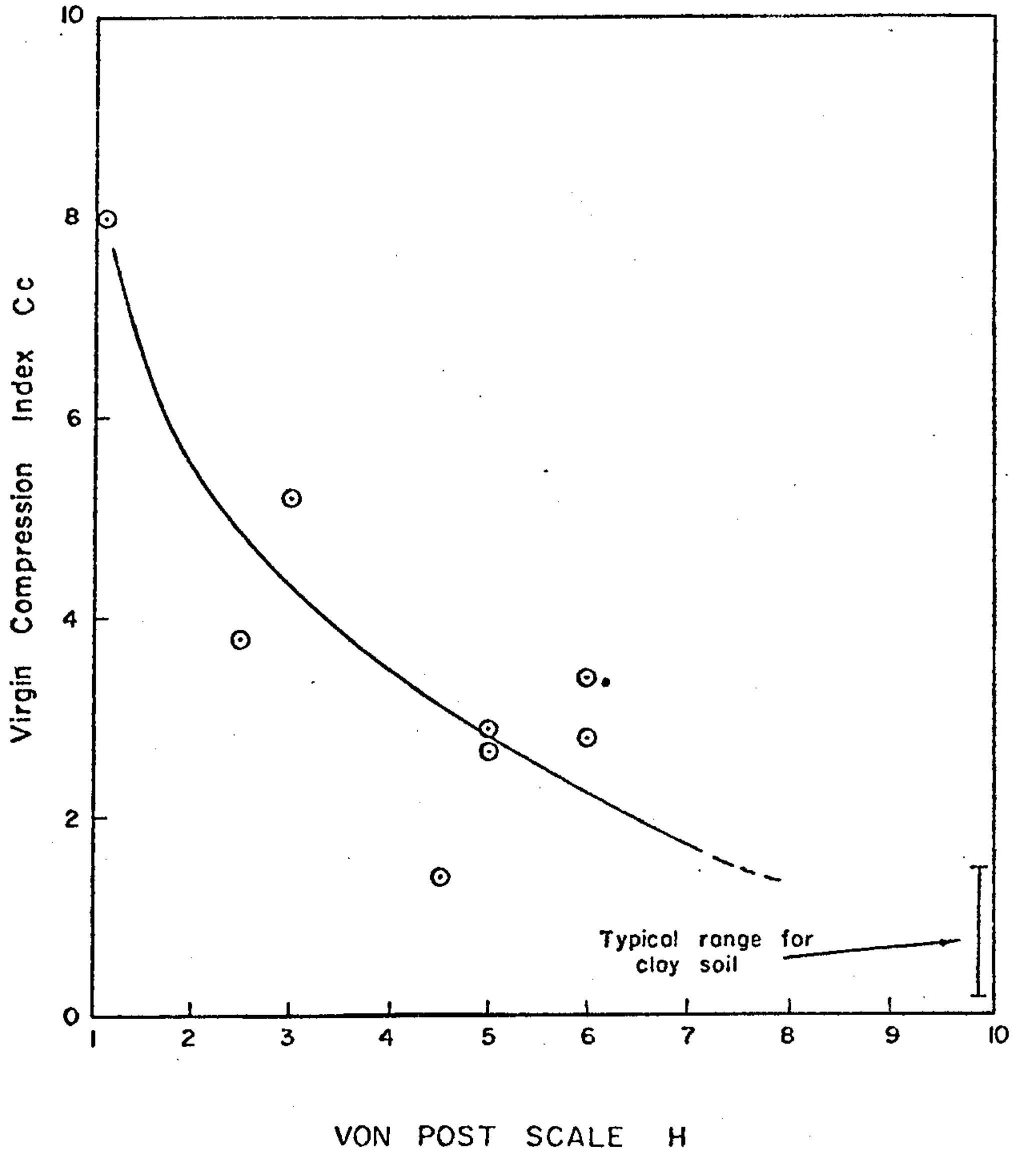


FIGURE 2.11 Compressions index (C_c) vs. void ratio



Virgin Compression Index Cc
10
8
6
4
2
0
1
2
3
4
5
6
7
8
9
10
Typical range for clay soil
VON POST SCALE H

Figure 2.12

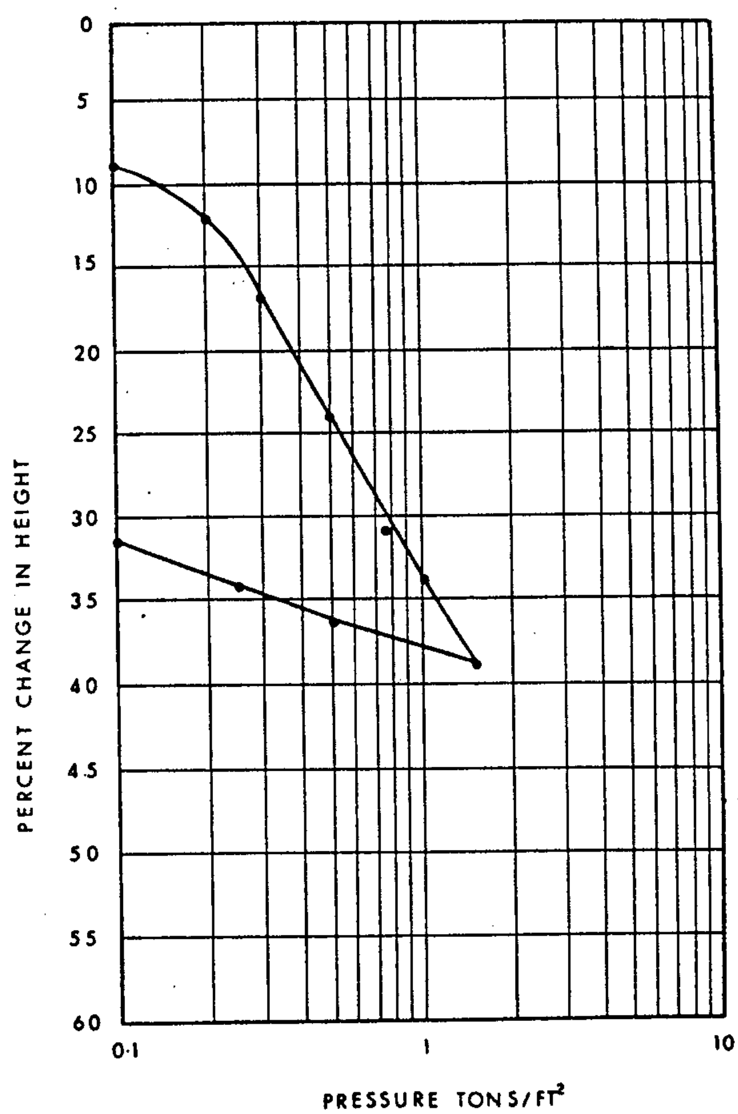
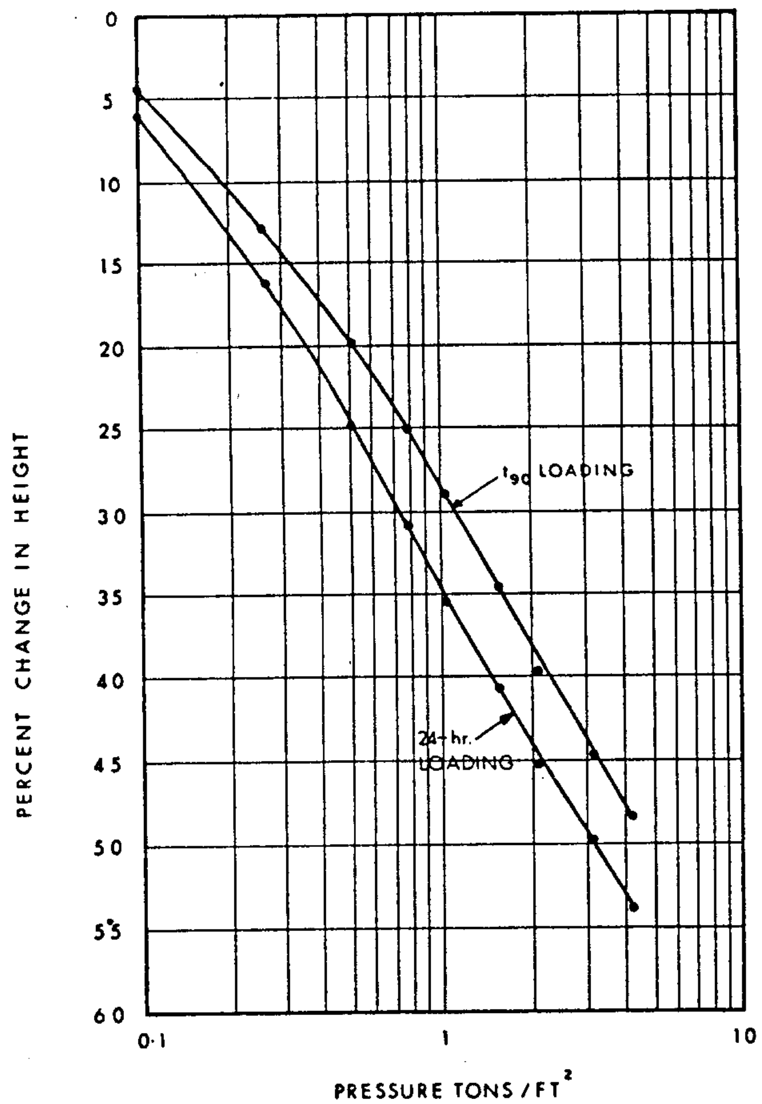

PERCENT CHANGE IN HEIGHT
0
5
10
15
20
25
30
35
40
45
50
55
60
0·1
1
10
PRESSURE TONS/FT²

Figure 2.13



0
5
10
15
20
25
30
35
40
45
50
55
60
PERCENT CHANGE IN HEIGHT
t_{90} LOADING
24-hr.
LOADING
0·1
1
10
PRESSURE TONS / FT^2

Figure 2.14

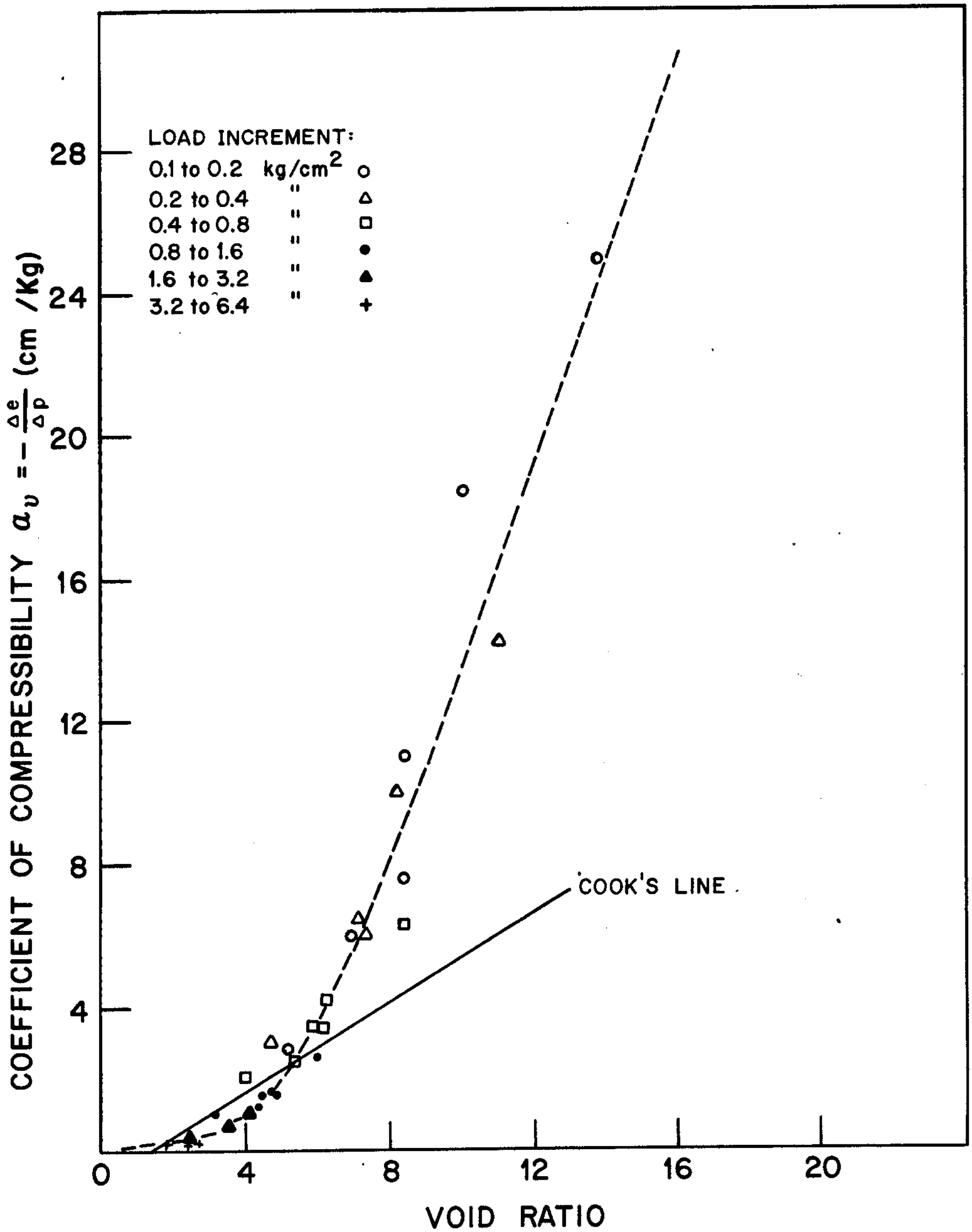
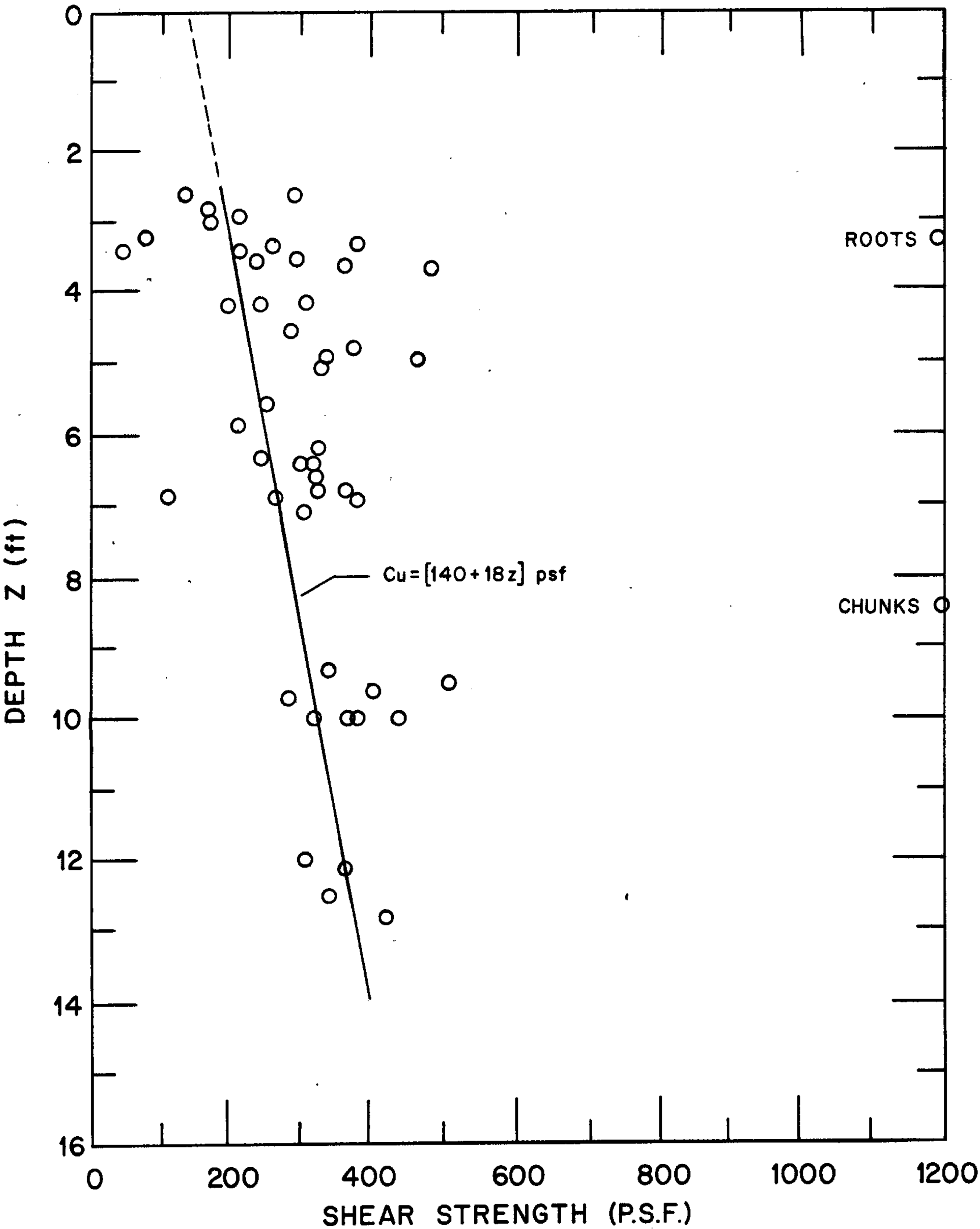
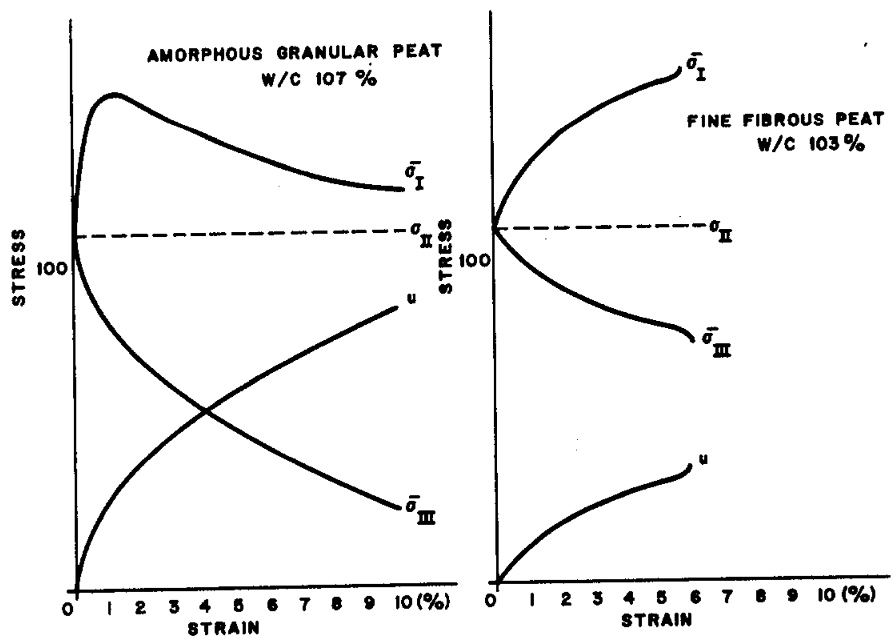

LOAD INCREMENT:
0.1 to 0.2 kg/cm^2 ○
0.2 to 0.4 " △
0.4 to 0.8 " □
0.8 to 1.6 " ●
1.6 to 3.2 " ▲
3.2 to 6.4 " +
COOK'S LINE
COEFFICIENT OF COMPRESSIBILITY $a_v = -\frac{\Delta e}{\Delta p}$ (cm /Kg)
VOID RATIO
0
4
8
12
16
20
24
28

Figure 2.15



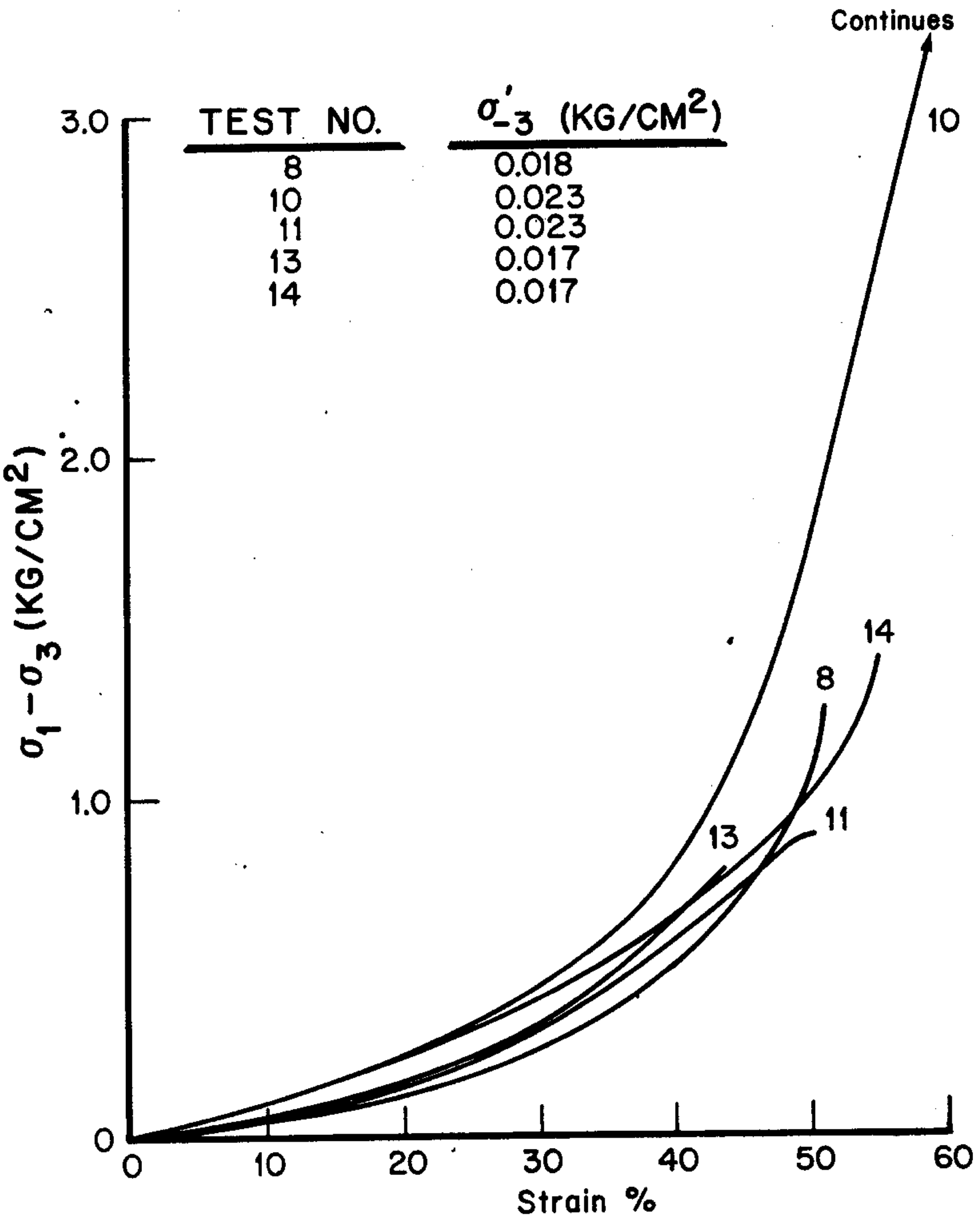
0
2
4
6
8
10
12
14
16
DEPTH Z (ft)
ROOTS
CHUNKS
Cu=[140+18z] psf
0
200
400
600
800
1000
1200
SHEAR STRENGTH (P.S.F.)

Figure 2.16



AMORPHOUS GRANULAR PEAT
W/C 107 %
STRESS
100
STRAIN
0 1 2 3 4 5 6 7 8 9 10 (%)
$\bar{\sigma}_I$
σ_{II}
u
$\bar{\sigma}_{III}$
FINE FIBROUS PEAT
W/C 103 %
STRESS
100
STRAIN
0 1 2 3 4 5 6 7 8 9 10 (%)
$\bar{\sigma}_I$
σ_{II}
$\bar{\sigma}_{III}$
u

Figure 2.17



Continues
TEST NO.
σ'_3 (KG/CM²)
8 0.018
10 0.023
11 0.023
13 0.017
14 0.017
$\sigma_1 - \sigma_3$ (KG/CM²)
3.0
2.0
1.0
0
10
14
8
13
11
0 10 20 30 40 50 60
Strain %

Figure 2.18

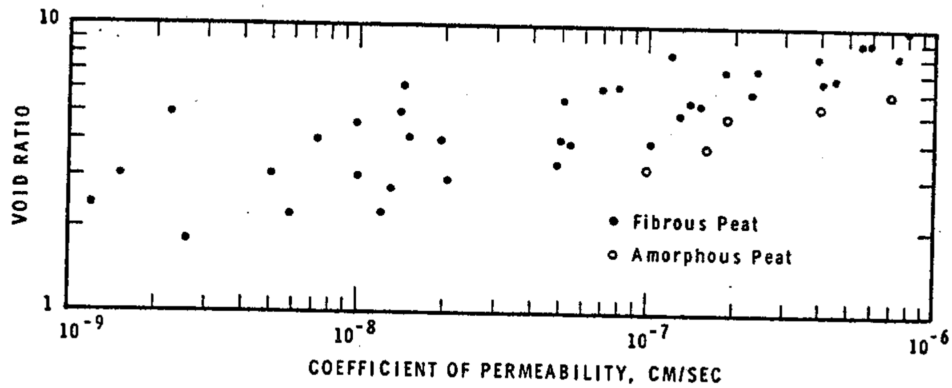


FIGURE Void ratio (*e*) vs. coefficient of permeability (*k*) (after Lea and Brawner 1963)

Figure 2.19

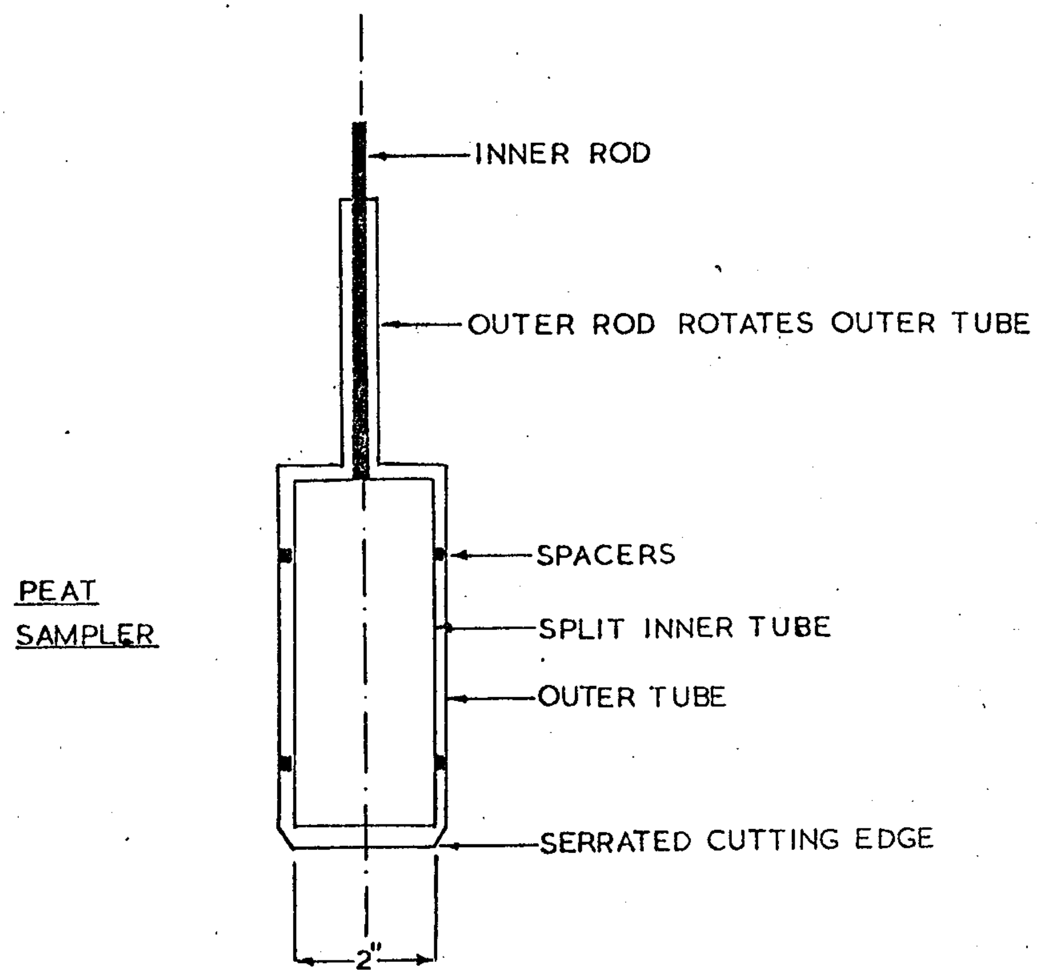

INNER ROD
OUTER ROD ROTATES OUTER TUBE
SPACERS
PEAT
SAMPLER
SPLIT INNER TUBE
OUTER TUBE
SERRATED CUTTING EDGE
2"

Figure 2.20

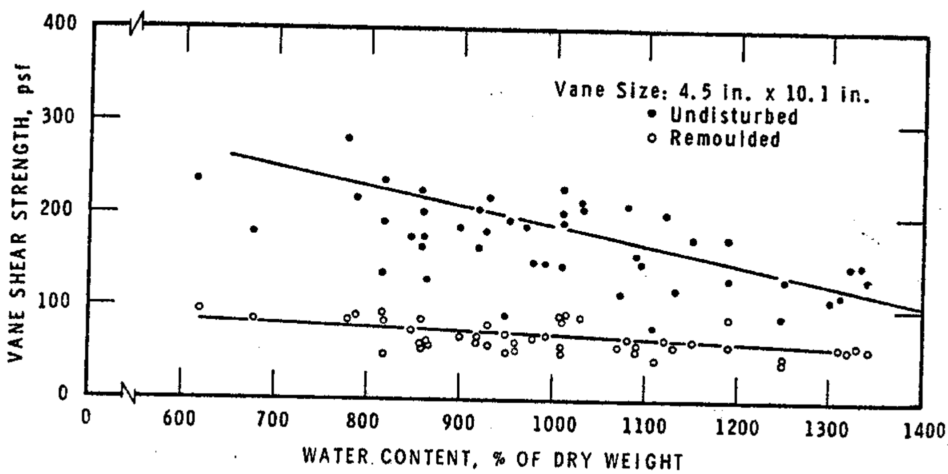
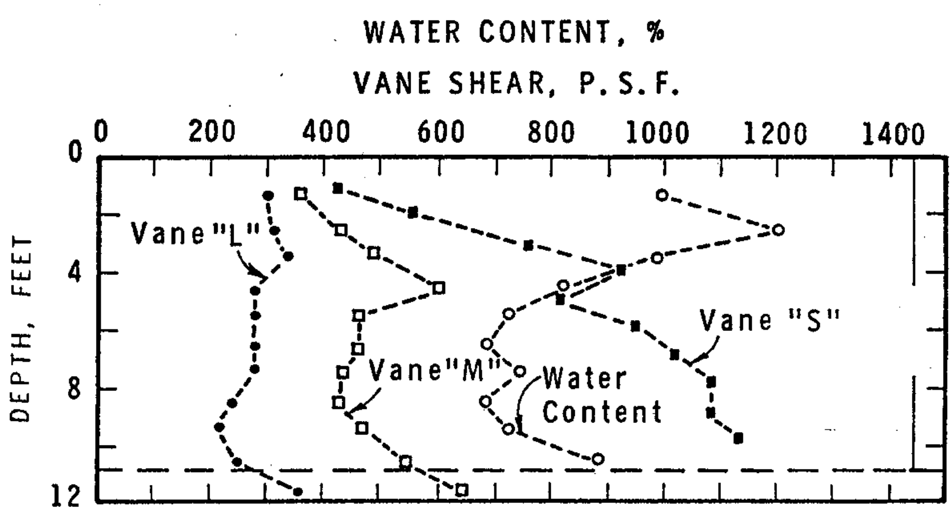


FIGURE 2.21 Shear strength vs. water content (after Anderson and Hemstock 1959)



Vane shear and water content vs. depth. L, 4 in. × 8 in. vane; M, 2.8 in. × 5.6 in. vane; S, 2 in. × 4 in. vane

Figure 2.22

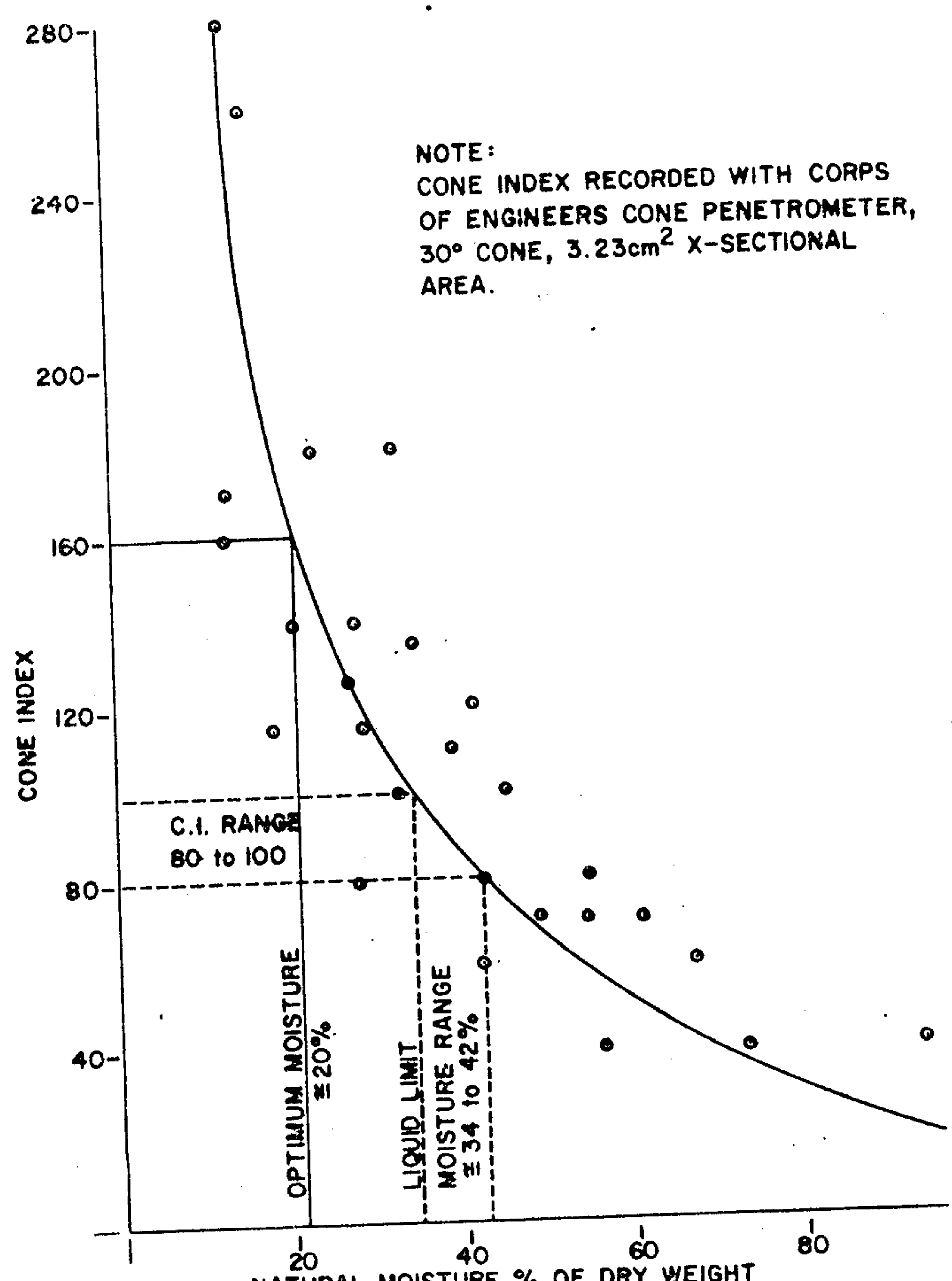


FIGURE 2.23 RELATIONSHIP BETWEEN CONE INDEX AND MOISTURE RELATED PROPERTIES IN AN ORGANIC SILT SOIL

3.0 DESIGN METHODS

3.1 General

Considerable progress has been made in recent years in the application of engineering design procedures to the construction of roadways on muskeg. Innovation has included the design of staged construction, the use of light-weight fill materials, surcharge loadings, bermed cross-sections and sequence of material placement to reduce lateral displacement in the peat. Safety and environmental impact requirements dictate a high confidence level in the design of roadways on muskeg.

There are three basic methods which can be used for the design and construction of roadways on muskeg:

1. To avoid muskeg
2. To eliminate muskeg
3. To design for and utilize muskeg.

Two general types of failure with roads built over organic terrain are: loss of stability (plastic deformation or shear failure) and excessive settlement or compression failure. In a stability failure, the weight of the fill placed on the surface of the muskeg squeezes out the underlying peat. The embankment sinks downward, while the peat at the base of the embankment is pushed up. Excessive settlement on peat can be considered as a failure due to compression. Normally, the greatest amount of settlement takes place early in the life of a structure (primary consolidation), while continuing settlement called secondary consolidation independent of the drainage process, continues slowly for many years.

3.2 Design Techniques

Depending on the type and composition of peat, thickness of peat, ground water level, frozen state and so on, various design techniques are used to determine the maximum thickness of road sections that can be constructed on muskeg. In shallower deposits, the most common design and construction method consisting of removal or excavation of compressible material and replaced by suitable sands and gravel backfill, is used. In deeper deposits partial excavation or no removal of peat may be used. Design of roadways sections may be based on:

(i) Shear strength and bearing capacity of peat
(ii) Consolidation characteristics of peat
(iii) Stability analysis.

3.2.1 Estimated Height of Fill

The maximum of height of fill that can be placed without producing failure is given by:

$$H_{ult} = \frac{K_1 T}{\gamma} \tag{3.1}$$

where T = shear strength of peat measured by vane

γ = unit weight of fill
K_1 = constant, 5.5 or 6

$$H_{allowable} = H_{ult}/\text{factor of safety} \tag{3.2}$$

As discussed in section 2.3.2 and 2.4.2, the shear strength of peat may be obtained in the laboratory and in situ testing. A typical triaxial test result is presented in Fig. 3.1.

The bearing capacity of peat is generally determined by using methods similar to those applied in the standard soil mechanics and the ultimate bearing capacity (q'_d) is obtained based on shear strength.

With the use of elastic theory, the ultimate bearing capacity is given by

$$q'_d = 2\pi T \tag{3.3}$$

and based on the theory of plasticity,

$$q'_d = (\pi + 2)\ T \tag{3.4}$$

Due to high compressibility of peat, the surface mat outside the loaded area is exposed to tensile stress which may cause punching failure.

$$q'_d = (2 + m)\ m\ \sigma_t \tag{3.5}$$

where σ_t = tensile strength of the surface peat
m = h/a, h is the thickness of peat
and a is the radius of contact area.

Based on equations 3.3 to 3.5, the allowable soil pressure (q_a) may be determined by the following relationship:

$$q_a = \frac{q'_d - \text{overburden pressure}}{\text{factor of safety}} \tag{3.6}$$

The factor of safety is generally equal to 2.

3.2.2 Consolidation Characteristic

The compressibility of peat resulting in immediate settlement and consolidation is discussed in Section 2.3.2 under mechanical properties. A typical time-settlement of peat is shown in Fig. 3.2. Fig. 3.3 presents a typical relationship between the coefficient of consolidation (C_v) and void ratio. C_v is an important factor to determine the long-term settlement. Based on e-log P consolidation tests and the time-settlement relationships, the anticipated primary and secondary settlement due to any load on peat may be ascertained. Various investigations reported the load-settlement relationship for fills over muskeg and these data are presented in Fig. 3.4. If the percent of total settlement is found to be excessive for the designed roadway fill sections, different construction techniques such as preloading, stage construction, partial removal of peat and so on may be used to eliminate the long-term settlement. The method of estimating preload requirement using settlement curves is discussed later in Section 4.

3.2.3 Stability Analysis

After the section of fill height and the corresponding consolidation analysis, a final analysis of the designed roadway sections to be built on peat, must be made. This analysis will consist of checking the stability of the embankment as a whole unit.

Soil and rock materials fail in shear if the applied shearing stresses on any surface exceed the shear strength of the materials along that surface. Stability analysis involves comparing the shearing stresses along potential failure surfaces with the available shearing resistance along those surfaces. The factor of safety (F) is defined as that factor by which the shear strength parameters must be divided (c'/F and ϕ'/F) to bring the potential sliding mass into a state of limiting equilibrium. The stress-strain characteristics of most soils are such that relatively large plastic strains may occur as the applied shearing stresses approach the shear strength of the material. In the design of a slope or embankment, the factor of safety must be greater than unity so that the strains will not exceed tolerable limits, and to allow for differences between the pore water pressures and shear strength parameters assumed in design and those that may actually exist within the slope.

Stability analysis is a procedure of successive trials. A potential failure surface is chosen and the factor of safety against sliding along that surface is determined. Different potential failure surfaces are selected and the analysis is repeated until the potential failure surface having the lowest factor of safety is found. The failure surface is known as the critical failure surface. The factor of safety against sliding along the critical failure surface is the indicated factor of safety for the slope.

The critical failure surface may be located completely within an embankment; it may lie totally outside of the embankment if it passes through retained materials and/or the foundation soils, or it may be located at any position between these two limits.

With the exception of a few special cases, all stability calculations to determine factors of safety should be based on effective stress analysis. The determination of the effective stresses requires a knowledge of the position of the phreatic surface within the embankment. For a fully compressed fill subjected to steady seepage, the effective stress can be determined from flow net. When the fill and/or the foundation is compressing under the weight of overlying material, the pore water pressures must be estimated using consolidation theory. Where the location of the phreatic surface is likely to be critical to stability, piezometers should be installed within an embankment to determine the actual location of the phreatic water surface. If the actual water pressures are found to be significantly higher than those assumed in design, the stability should be rechecked. Some modification to the design section may be required to maintain the desired safety. In the design of an embankment, the stability of the slope is usually of principal concern.

Failure Surfaces

The most commonly assumed failure surface used in stability analysis is the cylindrical surface, the axis of which is oriented parallel to the strike of the slope. On the two dimensional cross-section used for convenience in most stability analysis, the cylindrical surface is represented by a circular arc. Observation of full-scale slope failures in the field show that some failure surfaces are nearly circular. However, many carefully documented examples are available which show that the shape of the rupture surface is clearly non-ciruclar. When a slope failure occurs, differential shearing takes place along that surface on which the factor of safety is lowest. Although the calculations are made simpler if the failure surface is assumed to circular, stability analysis based soley on assumed circular failure surfacer may significantly over-estimate the factor of safety.

The true surface of sliding will deviate from the commonly assumed circular surface if the potential failure surface passes through zones having different shear strength characteristics or different pore water pressure conditions. Methods of stability analysis applicable to non-circular failure surfaces are included in the descriptions given below.

3.2.3.1 Method of Analysis

(i) Methods of Slices

With the exception of a few special cases, the methods used to calculate the factor of safety for any trial failure surface should account for changes in the shear strength parameters and varying water pressure conditions along the potential failure surface. Changes in the strength parameters and pore water pressure conditions can be taken into account by the general procedure known as the method of slices. In the method of slices, a trial failure surface is chosen and the potential sliding mass is divided into a number of vertical slices. Each slice is acted upon by its own weight, by shearing and normal forces on its vertical boundaries and by shearing and normal forces along its base.

(ii) Method of Infinite Slices

In the method of infinite slices, a circular trial failure surface is selected, and the stability of the potential sliding mass is considered as a whole, rather than the stability of each individual slice. Since the forces acting on the vertical boundaries of the slices produce zero net moment about the centre of rotation of the potentially unstable mass, the side forces are neglected. The shearing stresses and the normal stresses on the base of each slice are assumed to depend only on the weight of the slice and on the pore water pressures at its base. If the potential failure mass is divided into slices of unit width, the forces on the base of each slice will be numerically equal to the stresses on the base of the slice. This procedure is illustrated on Figure 3.5.

Factors of safety determined using the method of infinite slices will be in error on the conservative side, since the method completely neglects the side forces on the individual slices.

(iii) Simplified Bishop Method

Stability analysis using the "Simplified Bishop Method" is a variation of the method of slices and is limited to the analysis of circular arc failure surfaces. The potential circular failure surface is selected; the potential failure mass is divided into a number of vertical slices, and the stability of each slice is considered in turn using the assumption that the factor of safety for each slice is equal to the factor of safety for each of the other slices. Each slice is acted upon by its weight, and by shearing forces and normal forces on its vertical boundaries.

The shearing and normal forces acting on the vertical boundaries depend on the stress-deformation characteristics of the materials comprising the slide mass and cannot be evaluated rigorously. However, the summation of the forces acting on the vertical boundaries of the slices is zero and these forces may be neglected without serious reduction in accuracy. Neglecting the forces acting on the sides of the slices, the factor of safety is expressed by the equation:

$$F = \frac{\Sigma\ c'b \sec \alpha + N' \tan \phi'}{\Sigma\ W_o \sin \alpha}$$

where W_o = the weight of material within the slice,
c' = the effective cohesion for the soil,
b = the width of the slice,
α = the angle of inclination at the center of the base of the slice,
N' = effective normal force on the base of the slice,
ϕ' = the effective angle of internal friction,
F = the factor of safety.

For the Simplified Bish Method, N' is determined by the sum of the forces in the vertical direction according to the equation:

$$N' = \frac{Wo - b \sec \alpha\ (u \cos \alpha + \frac{c'}{F} \sin \alpha)}{\cos \alpha + \frac{\tan \phi' \sin \alpha}{F}}$$

where u = the pore pressure. Therefore,

$$F = \frac{\Sigma \left(c'b + (W_0 - ub) \tan \phi\right) \dfrac{\sec \alpha}{1 + \dfrac{\tan \phi' \tan \alpha}{F}}}{\Sigma\, W_0 \sin \alpha}$$

Since F appears on both sides, the equation must be solved by successive approximations. The procedure for calcualting the stability for a single trial failure surface is indicated on Figure 3.6.

Using a more regorous form of Bishop's analysis, the shearing forces and the normal forces acting on the vertical boundaries between adjacent slices may be taken into account. However, if the surface of sliding is circular, the improvement in accuracy is not likely to exceed 10 to 15 percent.

(iv) Morgenstern-Price Method

Where the potential failure surface deviates significantly from the circular configuration, methods of analysis that neglect the effect of the shearing and normal forces on the lateral boundaries of the slices may lead to significant error. Morgenstern and Price (1965) have presented a method for calculating the factor of safety for non-circular surfaces of sliding. This method takes into account the shearing forces and normal forces acting on the lateral boundaries of the slices and satisfies the conditions for horizontal, vertical and moment equilibrium for each slice. Owing to the number of iterative steps required to obtain solutions using the Morgenstern-Price analysis, calculations using electronic computers are virtually mandatory. Each solution indicates the factor of safety for a single trial failure surface. Additional trial failure surfaces must be selected and the factor of safety computed for each until the critical failure surface has been located.

It is emphasized that, regardless of the sophistication of the method of analysis and the capacity of the computing facilities available for stability analysis, the reliability of the calculated factors of safety is governed primarily by the degree to which input parameters are representative of the actual conditions within the embankment and its foundation.

(v) Wedge Analysis

If computing facilities and appropriate programs are not available to the designer, a reasonably accurate assessment of the factor of safety can be made for non-circular surfaces of sliding by manual computation. Where the potential surface of sliding does not differ greatly from a circular arc, the method of analysis illustrated on Figure 3.6 may be used. Where the configuration of the trial failure surface conforms approximately to two or more intersecting tangents, the factor of safety may be determined by using the wedge analysis illustrated on Fig. 3.7.

(vi) Horizontal Translation

Where a soft foundation stratum is located beneath the embankment, the factor of safety against horizontal translation should be checked. In checking the stability against horizontal translation, a trial failure surface is chosen that passes through the soft foundation layer, and the components of all of the forces acting on the potential failure mass are determined. The degree of safety is presented by the sum of the horizontal components of all forces tending to produce horizontal translation. An example showing the method of analysis is shown on Fig. 3.8.

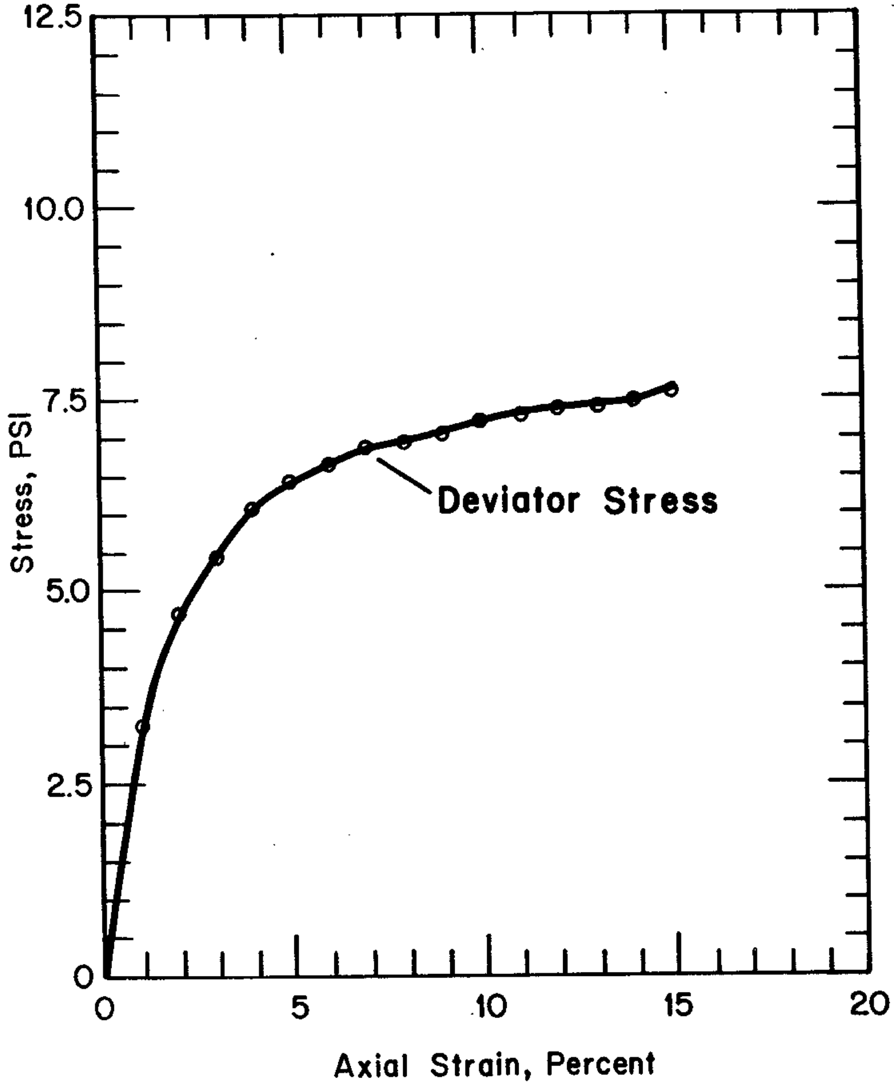
3.2.3.2 Effects of Earthquake

Where an embankment or a slope is subjected to a seismic disturbance, accelerations that accompany the ground motions produce stress fluctuations so that the dynamic shearing stresses are alternately higher and lower than the static shearing stresses.

Risk from earthquake acceleration can be reduced by either increasing the width or the crest or decreasing the slope angle. In the latter case, a reasonable basis for design is to ensure that the factor of safety indicated by an equivalent static analysis is greater than unity when the 100-year acceleration forces are included.

Strains which may occur during intervals of higher-than-static stress may result in distortion of the embankment as shown in Fig. 3.9. Procedures for estimating the magnitude of these strains have been proposed by Newmark, (1965) and by Goodman and Seed, (1965). However, these analysis are very complex. Provision to limit excessive distortion of embankments by earthquake shocks can be made by including additional horizontal acceleration forces in an equivalent static stability analysis. The value of the acceleration forces used in an analysis should be selected on the basis of the probability of earthquakes of various magnitudes occurring in the region of the embankment. Such information can be obtained from various government agencies.

TRIAXIAL COMPRESSION TEST



SPECIMEN NO.	INITIAL MOISTURE CONTENT(%)	INITIAL DRY DENSITY PCF	CONFINING PRESSURE (PSI)	DEVIATOR STRESS AT FAILURE PSI	SOIL TYPE
78A-1350	454.2	12.5	10.0	7.2	Peat

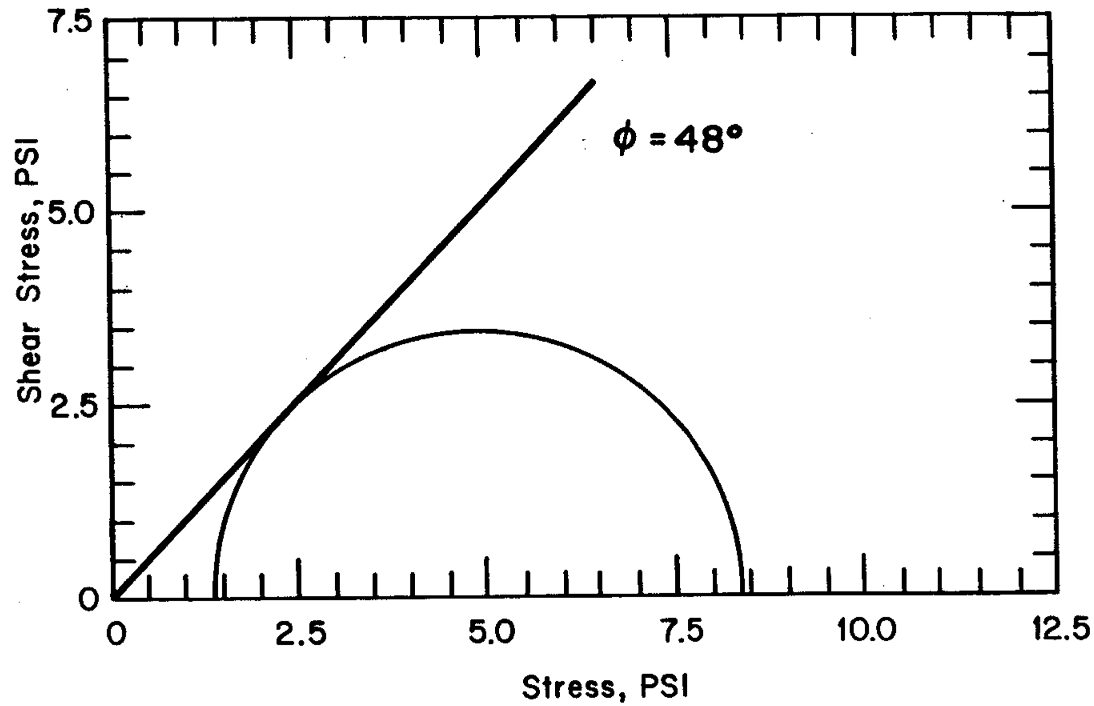


Figure 3.1

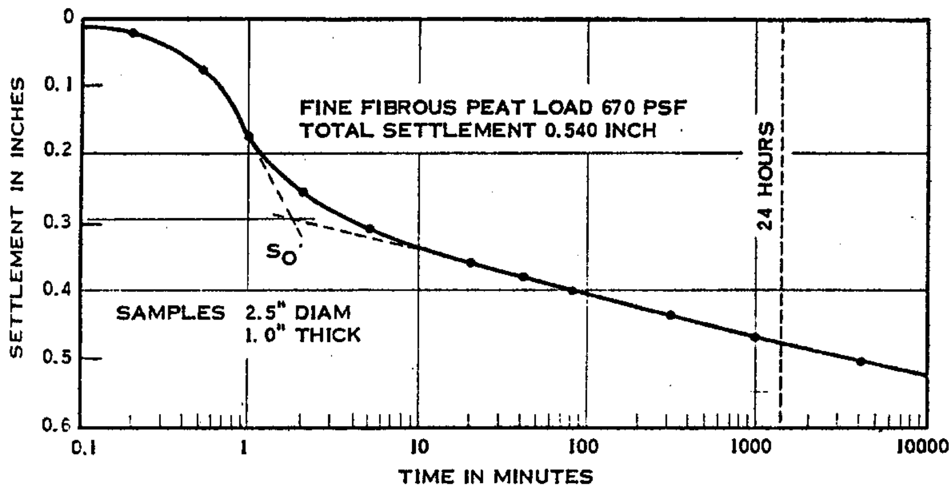
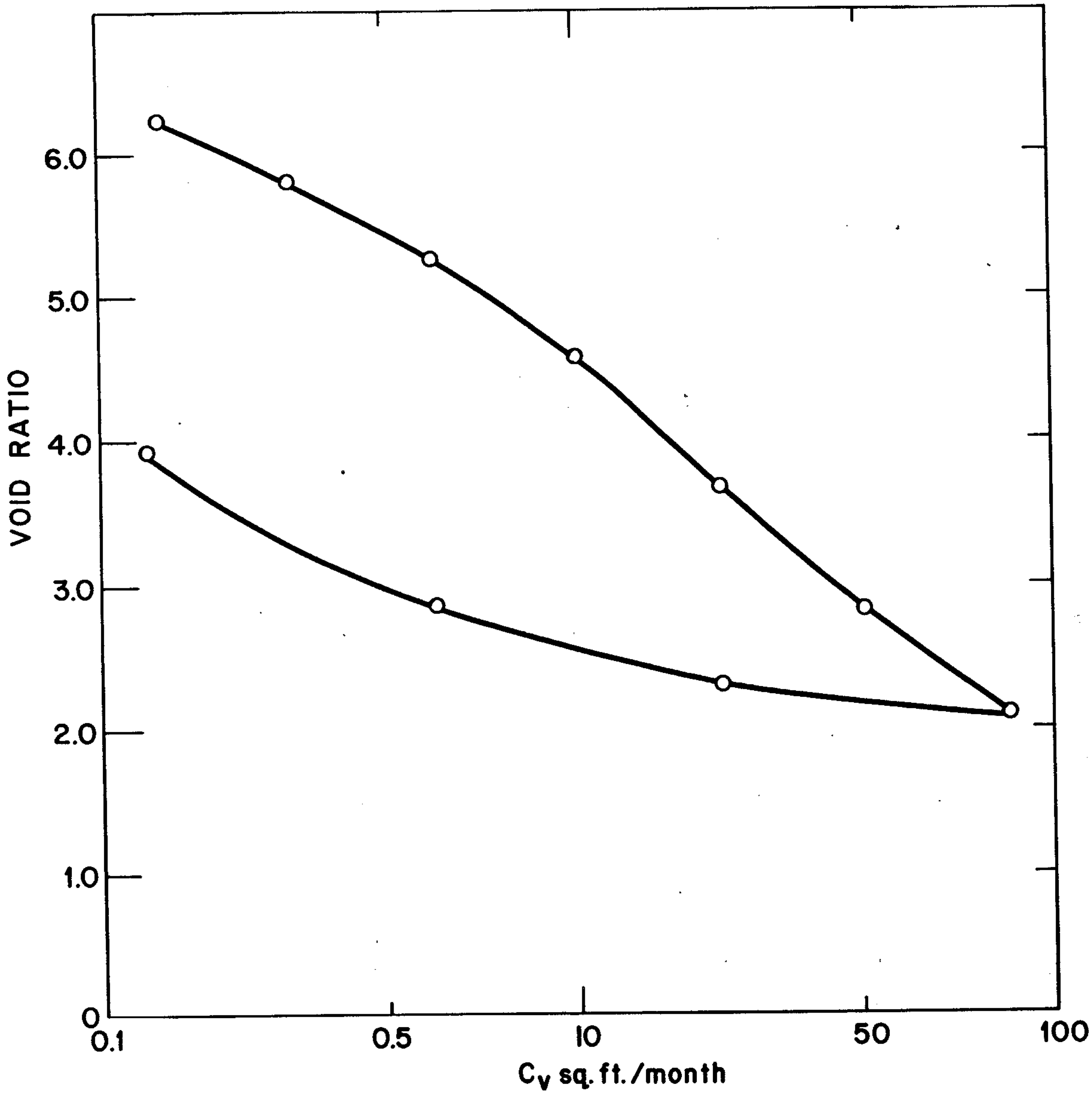


FIGURE 3.2 Typical time-settlement relationship for peat



Consolidation Test

Central Materials Load
Alaska DOT/PF

Project Name: Minnesota Drive Extension

Project Number: A36632

District: Central

Lab Number: 78A-1356

Field Number: T-105

Soils Class: Peat

Test Hole: TH-200

Depth: 5-7 ft.

Nat. Dry Density: 13.2

Nat. Moisture: 385%

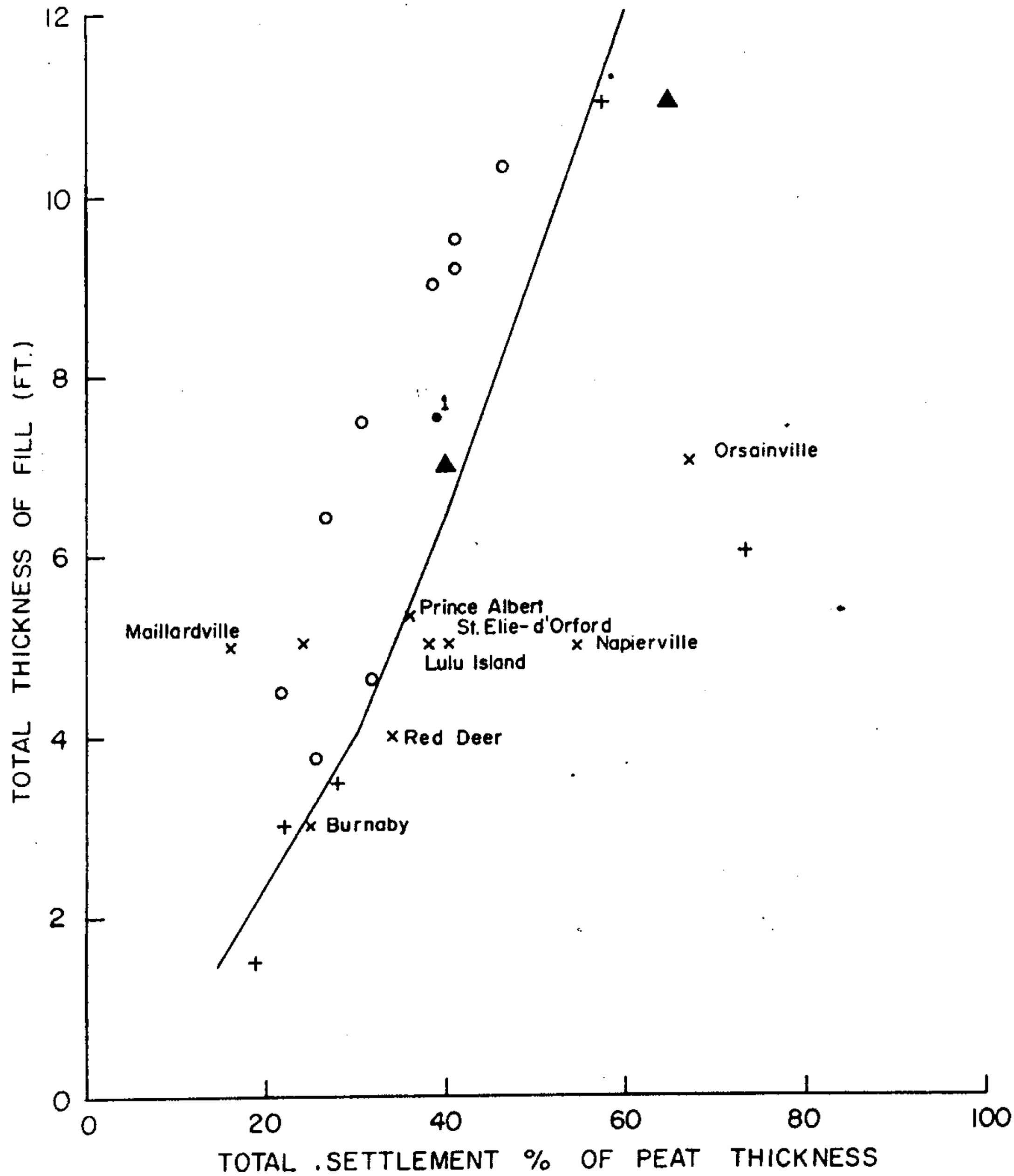
Moisture after Test: 224%

Initial Void Ratio: 6.883

Spec Gravity: 1.67

LL: PI:

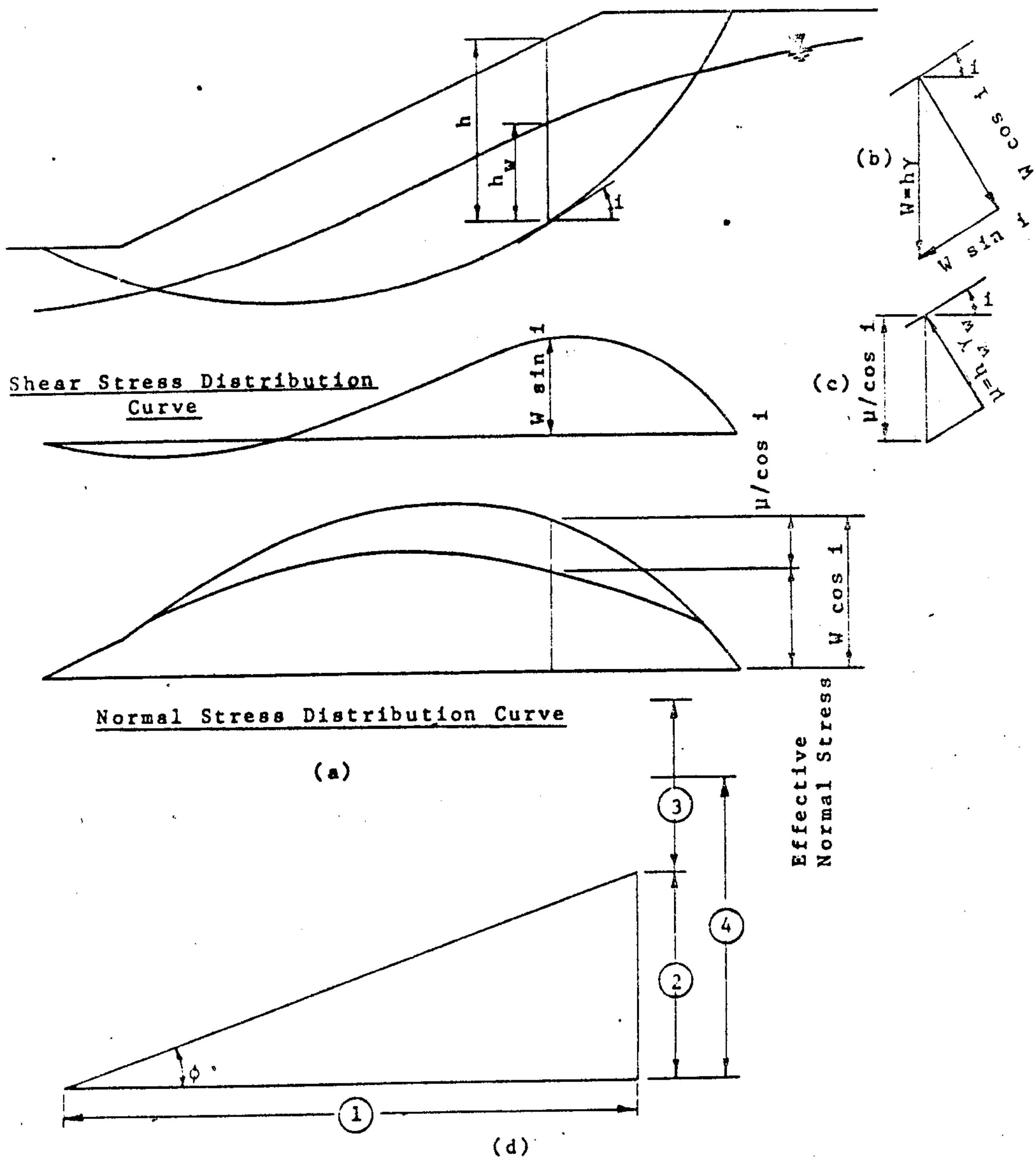
Figure 3.3



LEGEND

+ Weber , 1969
o Richmond Landfill (extrapolated to 20 yr. settlement)
× Muskeg Handbook
•[1] Ripley & Leonoff , 1961
▲ Lea ,

Figure 3.4



(1) = The total effective normal force on the trial failure surface.

(2) = The available shear resistance due to friction.

(3) = The available shear resistance due to cohesion.

(4) = The total shearing force on the trial failure surface.

$$FS = \frac{(2) + (3)}{(4)}$$

Figure 3.5(a)

PROCEDURE

1. Select trial failure surface.

2. Draw a number of approximately equally spaced vertical lines extending from trial failure surface to ground surface.

3. At each vertical line, determine the vertical stress at the failure surface due to the weight of the column of soil and water above surface.

4. Resolve the vertical stress into components normal and tangential to the trial failure surface.

5. Plot the tangential stress components as ordinates, and join plotted points to obtain the shear stress distribution curve. The area under the curve equals the total of the shearing forces along the trial failure surface.

6. Plot the normal stress components as ordinates, and join plotted points to form the total normal stress distribution curve.

7. Draw flow net to estimate neutral stress μ, (water pressure) along trial failure surface. Determine values of $\frac{\mu}{\cos i}$. A graphical method of determining $\frac{\mu}{\cos i}$ is indicated by Fig. (c).

8. Plot values of $\frac{\mu}{\cos i}$ below the total normal stress curve. The area of the hachured portion of the normal stresses distribution curve equals the total effective normal force on the trial failure surface.

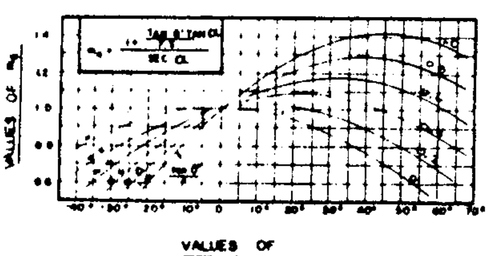
9. The factor of safety can be determined graphically as indicated on (d).

Figure 3.5(b)

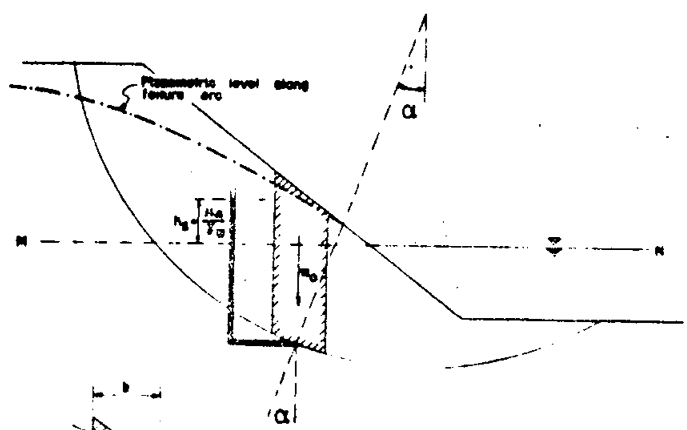
STABILITY ANALYSIS
BY METHOD OF
INFINITE SLICES

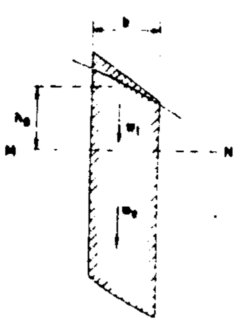
SLICE NO	OVERTURNING ELEMENTS (1)							COHESIVE ELEMENTS (2)			FRICTIONAL ELEMENTS (3)						(4)	1st Trial FS = Tan φ'/FS =		2nd Trial FS = Tan φ'/FS =	
	W OVERTURNING = W_o				α	Sin α	W_o Sin α	b	c'	c'b	h_s	$u_s = \gamma_w h_s$	$b u_s$	$W_o - b u_s$	Tan φ'	$(W_o - b u_s)$ Tan φ'	(2)+(3)	m_α	(5) = $\frac{(4)}{m_\alpha}$	m_α	(5) = $\frac{(4)}{m_\alpha}$
	z	AREA																			

$\Sigma(1) =$ $\Sigma(5) =$



NOTE α positive when failure arc slope is in the same quadrant as slope of ground.





NOTES

1. $W_o = W_1 + W_2$
2. 'W_1' is the weight of that portion of a slice above line MN and is computed from total unit weights
3. 'W_2' is the weight of that portion of a slice below line MN and is computed from submerged unit weights
4. 'h_s' refers only to the height of the piezometric level above line MN. However in the case of slopes with no partial submergence, 'h' would be measured from the failure arc.

AFTER A. W. BISHOP (GEOTECHNIQUE, MARCH 1955)

$$F.S. = \frac{1}{\Sigma(W_o \sin\alpha)} \Sigma\left[\left(c'b + (W_o - bu_s)\tan\phi'\right)\frac{1}{m_\alpha}\right] = \frac{\Sigma(5)}{\Sigma(1)}$$

TRIAL (1)

$F.S. = \frac{\Sigma(5)}{\Sigma(1)}$

= ______

=

TRIAL (2)

$F.S. = \frac{\Sigma(5)}{\Sigma(1)}$

= ______

=

TABULATED SOIL PROPERTIES			
γ			
γ_s			
φ'			
c			

FINAL F.S. =

Figure 3.6(a)

PROCEDURE

i. Select a trial circular arc failure surface.

ii. Divide the mass above the trial failure surface into approximately 10 vertical slices.

iii. Determine the weight (W_o) of each slice.

iv. Determine α, the angle of inclination of the trial failure surface at the base of each slice.

v. Compute W_o Sin α, the overturning element (1 on the table).

vi. From the width of the slice b, and the average cohesion c' along the base of the slice, compute the cohesive element (2 on the table).

vii. Calculate the pore pressure and the neutral force at the base of each slice. The pore pressure = the weight of water times the average piezometric head above the base of the slice ($\mu_s = h_s \gamma_w$), and the neutral force = the pore pressure times the width of the slice ($b\mu_s$).

viii. Compute the frictional element ($W_o - b\mu_s$) tan ϕ' where ϕ' is the effective angle of internal friction along the base of the slice.

ix. Assume a factor of safety F, and determine m_a for each slice using the graph. Calculate a new factor of safery (5) = $\frac{(2) + (3)}{m_a}$. The first approximation of the factor of safety then equals $\frac{\Sigma(5)}{\Sigma(1)}$.

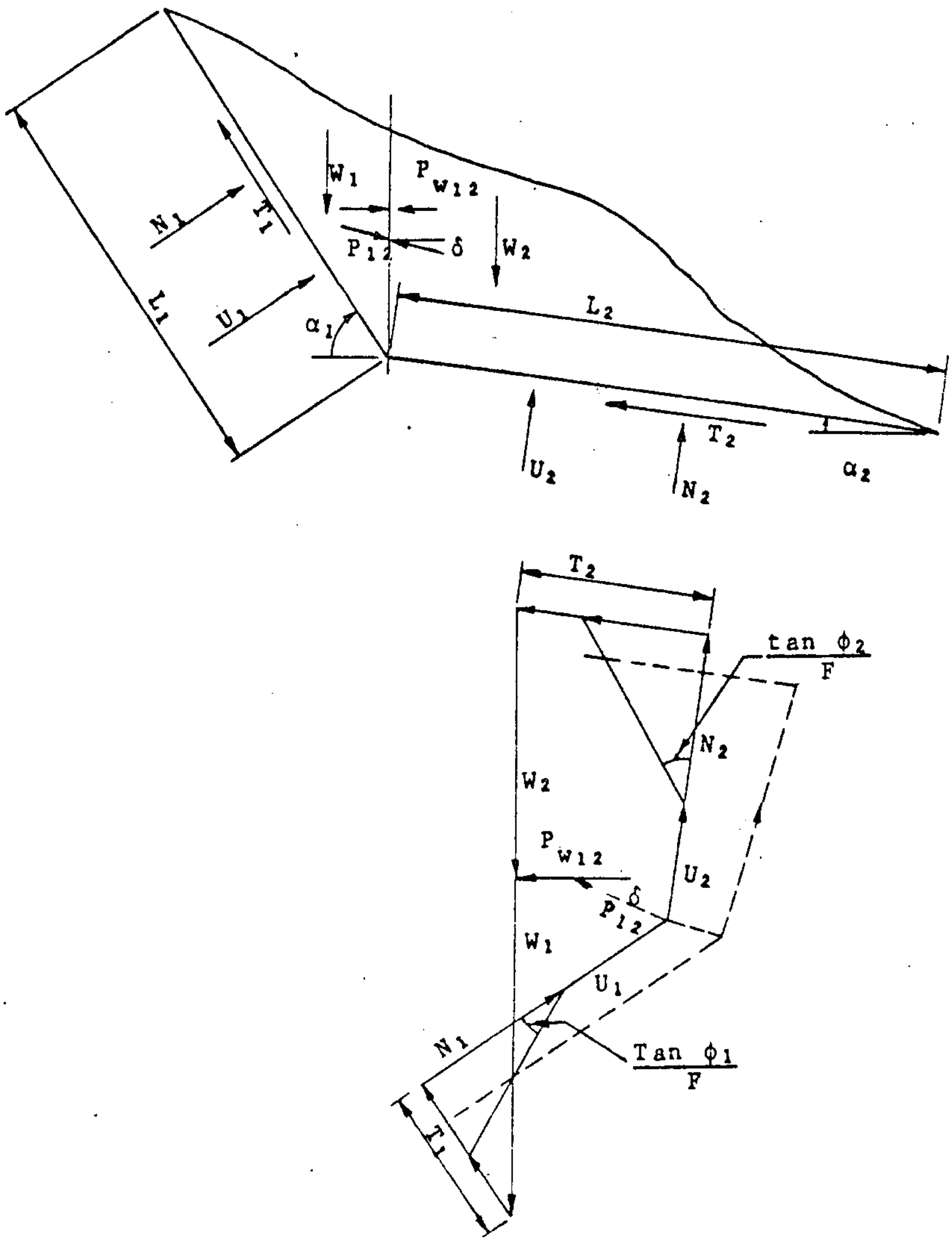
x. Using the factor of safety determined by step (ix) above, repeat step (ix) to obtain the second approximation of the factor of safety.

xi. Repeat (x) until a factor of safety F is obtained which is equal to the value of F used to determine m_a.

$$F.S. = \frac{1}{\Sigma(W_o \operatorname{Sin}\alpha} \Sigma \left[\left\{ c'b + (W_o - b\mu_s) \operatorname{Tan} \phi' \right\} \frac{1}{m_a} \right] = \frac{\Sigma(5)}{\Sigma(1)}$$

Figure 3.6(b)

STABILITY ANALYSIS
BY SIMPLIFIED
BISHOP METHOD



W_1, W_2 Weight of a wedge
U_1, U_2 Resultant water pressure along the base of the wedge
N_1, N_2 Effective normal force on base of wedge
T_1, T_2 Shear force along base of wedge
L_1, L_2 Length of base of wedge
α_1 α_2 Inclination of the base to the horizontal
P_{w12} Resultant hydrostatic force at interface
P_{12} Effective force at the interface
δ Inclination of P_{12} to the horizontal
ϕ_1, ϕ_2 Effective friction angle on base of wedge
c_1, c_2 Effective cohesion on base of wedge

Figure 3.7(a)

PROCEDURE

1. Choose a trial failure surface consisting of two or more intersecting tangents.

2. Separate the sliding mass into segments bounded by vertical lines which pass through the points of intersection of the base tangents.

3. Assume that the effective stresses across the vertical boundaries between adjacent wedges are inclined at an assumed angle δ to the horizontal. Note that the neutral (water pressure) forces across the boundaries between adjacent wedges act horizontally.

4. Choose a trial factor of safety (F).

5. Construct the force polygon for the first wedge. This gives a value for the force P_{12} which satisfies equilibrium conditions for the first wedge.

6. Using the value of P_{12} as determined in step 5 above, construct the force polygon for the second wedge. If this force polygon does not close, proceed with step 7.

7. Choose different values of F and repeat steps 5 and 6 above until a value of F is found such that the total force polygon does close.

   If δ is assumed to be zero, the shearing stresses on the vertical boundaries between adjacent wedges will also be zero, and the factor of safety will be underestimated. The maximum possible value of δ is equal to the angle of internal friction for the material through which the vertical boundary passes. Assumptions that $\delta=\phi$ leads to an overestimation of the factor of safety. The assumption that $\delta=\frac{1}{3}\phi$ usually leads to reasonable results.

Figure 3.7(b)

STABILITY ANALYSIS
FOR WEDGE FAILURE

STABILITY ANALYSIS - HORIZONTAL TRANSLATION

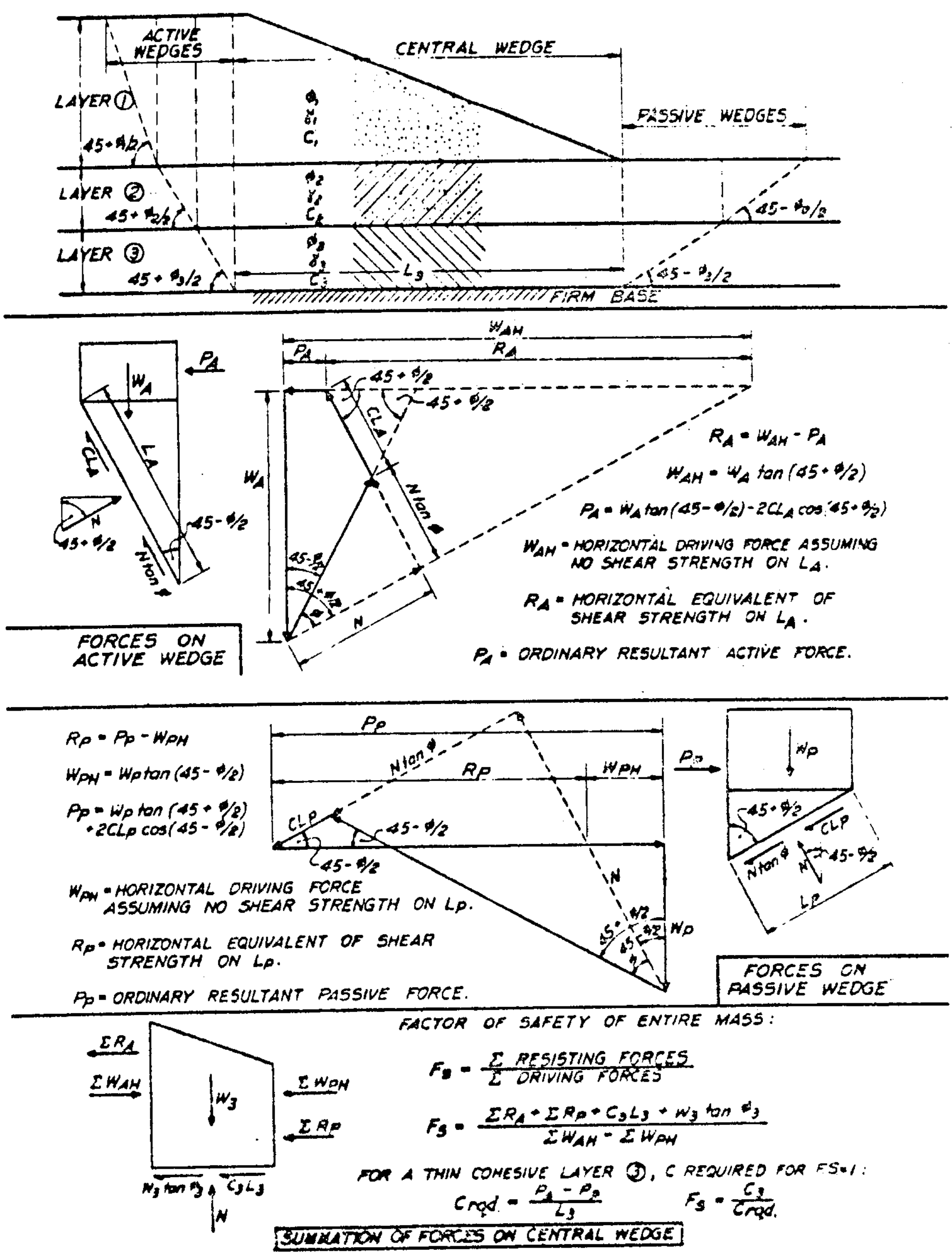
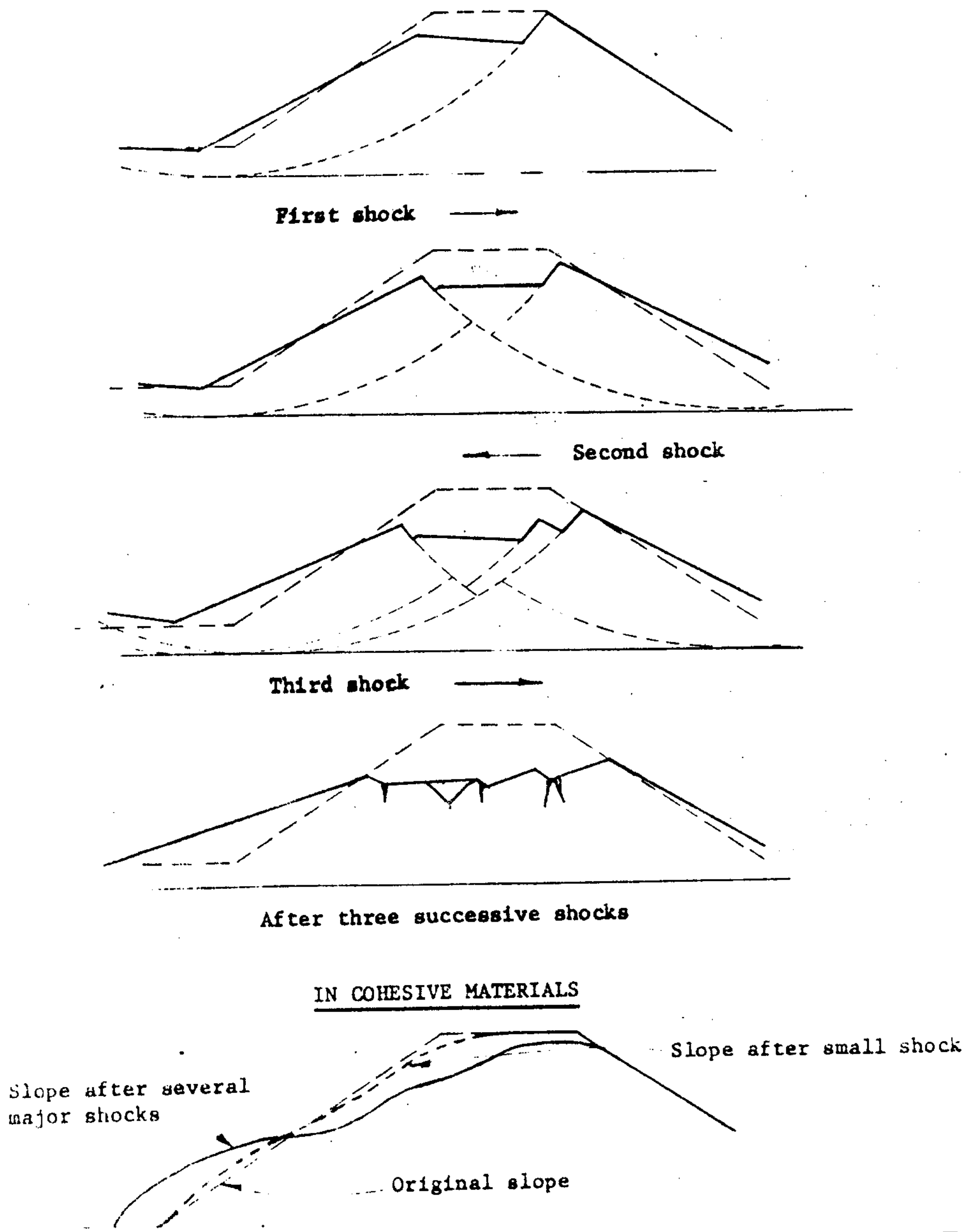


Figure 3.8



IN GRANULAR MATERIALS

Figure 3.9

MAJOR EMBANKMENT DEFORMATION DUE TO EARTHQUAKE

4.0 CONSTRUCTION TECHNIQUES

4.1 General

Many factors that influence the methods of construction of roadways over organic terrain are the characteristics of peat and the underlying soil, peat thickness, availability of construction materials, equipment required, available time, location of structures and drainage requirements. The method of construction to be used should provide the desired standard of road at the lowest cost possible. Fundamentally there can be only three basic approaches to construction in muskeg. Thet are:

1. Relocation or avoid muskeg
2. Elimination or removal of muskeg
3. Construction in muskeg.

In primary road construction, it is often difficult to avoid or to utilize muskeg as a subgrade which dictates the removal of muskeg with abnormal costs. An obvious approach to the problem of road construction over muskeg is to excavate unstable material and backfill with appropriate inorganic soils. It is usually only economically feasible for shallow depths and opinion differs on the maximum feasible depth (approxiamtely 3.5 m in U.S.A. and Canada). So, many structures have been built lately over muskeg with appropriate design considerations.

Basic design considerations in muskeg are:

1. Stability duration construction and long term condition
2. Deformation includes immediate settlement or consolidation with time
3. Selection of suitable construction techniques, availability of construction materials and associated problems
4. Economical optimization.

Based on the engineering properties of subsoil conditions and configurations, stability analysis must be carried out to determine the factor of safety. The geometry of the structure must be changed or alternate suitable construction technique has to be applied if an adequate factor of safety is not feasible. (Stability analysis are discussed in the previous Section 3.0)

One of the major problems in building over peat is to predict the magnitude and rate of settlement. Field settlement is largely due to consolidation. This deduction is confirmed by field observation providing the shear strength is not approached or exceeded. At a number of locations, toe stakes and tilt meters have beed used to measure horizontal displacement. During construction, horizontal movements are usually considerable, i.e. in the order of 1 to 3 feet, but these movements account for less than 10% of the settlement. Except in cases of serious instability when corrective measures are required, the horizontal movements decrease rapidly as consolidation takes place and seem to be of no concern after surcharge is removed. Soft clay under the peat does, of course, introduce complications. The construction technique to be applied over muskeg is to be related to cost analysis.

Table 4.1 presents various techniques which can be used successfully to build roadways on muskeg. These construction methods generally improve the site conditions and thereby minimize the unacceptable limits as discussed in previous sections. A typical pavement thickness performance is shown in Fig. 4.1. Typical roadway sections are shown in Fig. 4.2.

4.2 Peat Excavation and Backfill

The complete removal of peat and its replacement by good fill to provide a solid foundation is usually used for roads crossing shallow deposits. Current practices in North American appear to favor mechanical excavation although deep peat deposits are still troublesome. In those cases gravity displacement methods, with or without partial excavation, are often utilized. Explosives are used less now than they were prior to the last forty years due to the unpredictable results. The use of hydraulic stabilization or jetting is confined to limited states, although the method has considerable potential where large amounts of granular fill and water are available. Figs. 4.3 through 4.6 illustrate typical excavation and fill procedures.

TABLE 4.1 CONSTRUCTION METHODS IN MUSKEG

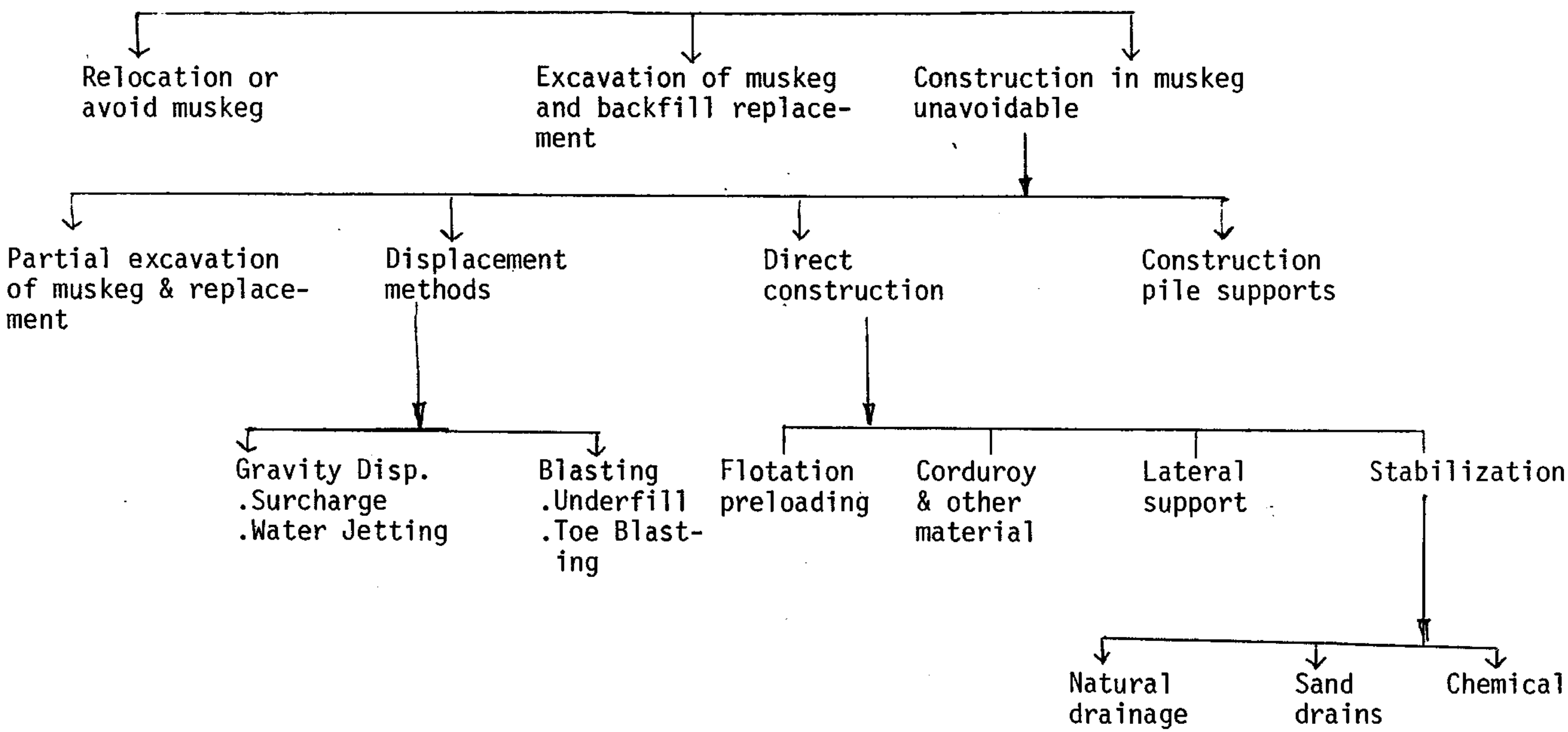


TABLE 4.2 USE OF EXPLOSIVES

	Toe Blasting	Trenching	Underfill Blasting
1. Peat Depth Limitations	10-20 Ft.	0-10'	7-20 ft.
2. Explosive Charges	1.5 lb/ft depth of peat, spacing 6 to 8 ft. apart.	3 to 4 lb/yd^3 longitudinal rows	3 to 5 lb/ft. depth of unstable material in a single row of holes spaced at 10 to 15 ft. apart.

Mechanical excavation consists of mechanical digging and disposal of the peat. The operations of initial excavation, backfilling and removal of replaced materials are normally performed simultaneously. Where excavation results in a relatively stable trench, backfilling can be performed as a separate operation. In North America, the maximum economic depth of mechanical excavation is considered to be 6 to 12 ft. Some other countries, 12 to 18 ft. has been recommended.

Peat excavation in permafrost areas is not usually employed, particularly for road construction. The natural insulating properties of peat and the high ice content bonds peat into a tough mass in frozen conditions which is suitable for the road construction without disturbance.

4.2.1 Gravity Displacement

Gravity displacement consists of building an embankment over peat so that the peat is continually displaced laterally by the weight of the fill. Due to surface effect, the gravity displacement method may need trenching or partial excavation and even blasting of the surface mat.

The technique of gravity displacement appears to be most useful at sites where the peat has been relatively low bearing and shear strength and where an adequate granular fill material is availble. It is a comparatively slow method and careful control must be employed to remove peat under the embankment.

The most common methods of placing fill to produce gravity displacement of the underlying peat are: end tipping, asymmetrical tipping and symmetrical tipping. End tipping is commonly used in North America, the other methods are employed mainly in Europe.

In the end tipping method, the embankment is slowly advanced along the centerline of the road by depositing fill at the head of the embankment. The displaced peat in front of the embankment is excavated and deposited along the sides of the advancing fill or hauled away as waste. Peat displacement is aided by a surcharge and by maintaining the fill head in a v-point. This also assists in displacing a maximum volume of peat to the sides so that an unmanageable peat wave is not built up ahead of the fill. The resulting lateral peat waves usually require only a minimum of leveling or trimming.

4.2.2. Use of Explosives

Explosives have been used principally in North America. The inability to insure that no troublesome pockets of peat are left below the embankment, has limited the use of this method, especially for primary road construction where very little or no future settlement can be tolerated. However, blasting techniques still have potential uses, particularly in secondary construction to augment the gravity displacement technique.

The most common methods of blasting are toe shooting, trenching and underfill blasting. Figs. 4.4 to 4.7 illustrate these methods. The amount of explosives used for these different methods is described in Table 4.2 and this varies with the type of peat and the effect desired.

4.3 Direct Construction on Peat

This is restricted by stability and settlement consideration. With stability assured, the placing of roadways directly on peat is a matter of design and the structure can later accommodate large and possible differential settlements. Different methods which may be used for the direct construction on peat are flotation, preconsolidation, lateral support and stabilization.

With flotation methods, the limited bearing capacity of peat is utilized and continued settlements are accepted. Some accommodation to settlements can be provided by using flexible pavements and corduroy construction methods which spread the load over a larger area for additional support which decreases settlements. The supporting capacity and stability may also be obtained by using lateral supports, such as berms.

4.3.1 Flotation

The fill is floated on the muskeg. The living organic mat in muskeg areas has some strength to support loads and care must be taken to ensure that the mat is not broken by the fill material. This method is generally the cheapest in initial cost and may be suitable over very deep muskeg deposits where removal is neither technically or economically feasible. However, severe differential settlements with continued high maintenance costs can often occur. For these reasons the flotation method is limited to light traffic areas or where lower standards of performance are required.

4.3.2 Corduroy

This is the oldest method of construction on peat. The peat and the fill is placed over brush, straw or logs to form a mattress or raft. The principal use of corduroy construction is:

(1) As a raft to provide a bouyant effect to support the structure
(2) To distribute a load as evenly as possible over peat
(3) To prevent fill material from breaking and penetrating through the surface of the peat.

The third method is also limited to minor roads over peat where the traffic is light and the requirement for good pavement profiles are lower. Fig. 4.8 illustrates the corduroy method of construction.

4.3.3. Preconsolidation

The preloading or preconsolidation method is used to eliminate or partially eliminate the long term compression. A preload of sufficient magnitude and duration is applied to cause the compression of the peat which would normally occur under the proposed design load over the expected life of structure. A typical preload construction technique is shown in Fig. 4.9. The method of estimating the magnitude and duration of the preload is shown in Figs 4.10 and 4.11. The surcharge selected must be checked for stability against shear failure and the effectiveness will depend on the ratio of surcharge to final load. This ratio in turn will depend on the final design height, the thickness of peat and its characteristics, and nature of the underlying soil. A ratio of 1.5 to 2 may be used and for deep peat the feasibility of preloading is quite uncertain.

Brawner (17) reported that the major problems of low shear strengths, high differential settlements, excessive quasi-deflection; and poor drainage chracteristics can be overcome by preconsolidation. Shear strength can be increased by a slow rate of fill placement and shear failures may be prevented by the installation of field instruments that will indicate impending instability. Fig. 4.12 shows a typical test section.

4.3.4 Stabilization

This method consists of natural drainage, use of sand drains and chemical grouting. The sand drain method of stabilization was developed specifically to accelerate settlement and consolidation of soft and compressible soils and has been used extensively for silty soils. The time required for consolidation of a soil varies directly as the square of the length of the escape path of the soil moisture. The purpose of vertical sand drains is to reduce the distance the water has traveled by permitting it to travel vertically as well as horizontally, thus shortening the period of consolidation.

As shown in Fig. 4.13, vertical holes are made by drilling, jetting or driving a mandrill and back filled with clean sand. To allow a horizontal flow of water as well as vertical flow a sand blanket or sawdust cover is spread over the surface of the peat before the embankment fill is placed. The sequence of construction is described in the figure. Another case history for sand drains is prescribed in Fig. 4.14.

4.4 Special Construction

Pile foundations transmit loads directly to a firm stratum underlying the muskeg and thereby settlement is eliminated. The method is particularly suitable for use in built-up areas where the proximity of buildings and public services, such as water, gas, electricity, etc., make the use of any other method impractical. The use of pile foundations is very expensive and has been limited to roads in built-up areas.

4.5 Construction in Permafrost Regions

In Alaska and the northern part of Canada, the lower road requirements, climate and the existence of permafrost have resulted in variations of the flotation construction technique. The gravity displacement method of construction obviously has limited application in permafrost regions and much of its success will depend on local conditions.

In permafrost areas the increased strength of peat when frozen may be maintained or induced by an insulating fill. In areas of continuous permafrost the underlying peat can be frozen during the winter and then protected against thawing by an insulating fill. The maintenance of a frozen condition in an area of discontinuous permafrost requires careful study of the "thermal balance" and success can be ensured only by careful and properly scheduled construction.

Winter roads are used only seasonally and rely on the increased bearing strength caused by freezing of the subgrade, muskeg and/or above ground layer of compacted snow or ice. Although maintenance after construction depends on the continuance of cold weather, construction and surfacing with snow are inexpensive and simple.

Other design and construction techniques used for roadways on permafrost, [Phukan (18)], may also be applied in frozen muskeg encountered in the permafrost areas. Factors such as type and characteristics of peat, thickness, temperature profile, ice content and so on, will dictate the suitable methods of construction on muskeg. Thermal degradation or disruption must be eliminated for the successful construction method.

4.6 Other Construction Techniques

The problem of construction on muskeg has also been solved by various construction techniques such as the use of fabric, lightweight materials, horizontal reinforcement nets and insulation. These techniques are discussed under the following headings.

4.6.1 Fabrics in Road Construction

Various investigators () have reported the role of fabrics in reinforcing weak soil subgrades for road construction when fabrics are placed between the natural soil and the fill material. Two vital functions are served:

1. The fabric acts as a separator between the natural soil subgrade and the fill material, preventing an intermixing of the two materials. This prevents contamination of the aggregates by the underlying soil, thereby maintaining its required strength characteristics.

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1. The fabric acts as a separator between the natural soil subgrade and the fill material, preventing an intermixing of the two materials. This prevents contamination of the aggregates by the underlying soil, thereby maintaining its required strength characteristics.

2. When soil subgrades are poor, that is, consist of soft, compressible soils, the fabric plays a major role of reinforcement duPont (39) has determined that fabric placed on a soil will, acting alone, add as much as 4 CBR percent to the soil. A test section of road in the Tongrass National Forest near Petersburg, Alaska (Ref. 37) consisted of about 10 feet of muskeg at the surface and was to be used to support logging traffic. The average vane shear strength of the peat was 250 p.s.f. and the saturated soil has a water content of approximately 960%. The test section was subdivided in stations along its length into zones of a double layer of fabric, a single layer of fabric and no fabric. The fabric wsed has Fibertex. The test was planned to illustrate the differences in the thickness of rockfill required to reach a stable and permanent road surface. Settlement and thickness measurements indicated that where no fabric had been used, depths of fill from 5 to 7.5 feet are required. With fabric (either single or double thickness) the required depth of fill was 3.5 and 5.5 ft. Thus, the savings of rock fill amounts to 28%. Clearly, the presence of fabric prevented local bearing capacity failures from occuring and proved the value of fabric utilization.

At present various types of fabrics are available in the market (Ref. 35) and depending on their uses, fabrics may be selected to do the required job.

4.6.2 Lightweight Fills

The problems of construction on muskeg has also been solved by the use of lightweight fill materials, a method mainly applicable to embankments (40,41). A lightweight fill either praticially replaces excavated heavier soft material or rests on the undisturbed soft material.

If the weight of excavated material compensates for a large portion of weight of the structure, the net stress increase will be small. If such compensation is not possible, partial excavation of soft soils has limited usefulness. The costs of lightwight fill and excavation are the controlling factors in determining the feasibility of partial excavation. Where a vegetation mat exists, excavation may be detrimental because the vegetation, if undisturbed, can provide support for both construction equipment and the lightweight fill. In addition, any excavated material has to be disposed of in an environmentally acceptable manner, which may be both difficult and costly.

The controlling criterion for this type of embankment construction is the availability of lightweight fill materials in sufficient quantity at an economical price. Some lightweight materials now in use are sawdust (66 lb/ft^3 = 1060 kg/m^3); expanded shale or clay (55 lb/ft^3 = 880 kg/m^3), oyster shell (70 lb/ft^3 = 1120 kg/m^3), and fly ash (75 lb/ft^3 = 1200 kg/m^3). The use of logs reclaimed from site clearing operations as well as installation of lightweight culverts to reduce the weight of embankments has also been successful in many instances.

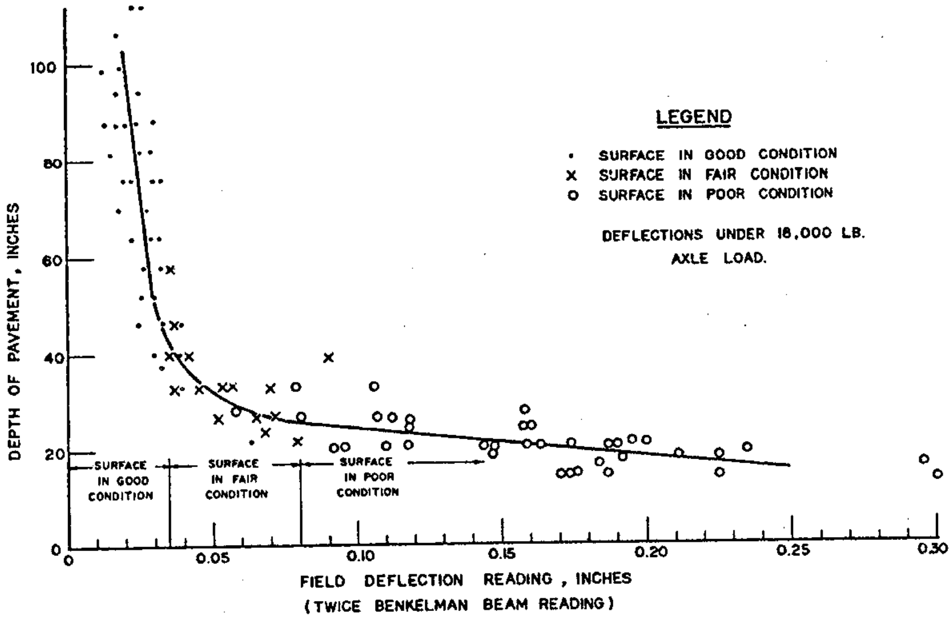
Lightweight fills are particularly suitable on soft organic soils or soft clays. The use of organic soils for foundation or fill material may be acceptable when strengths and settlemtns are within acceptable limits.

4.6.3. Horizontal Reinforcement Nets

As shown in Fig. 4.15, fabric may be used as reinforcement to increase the stability of an embankment. Specifically, the potential differential settlements and cracks near the side slopes may be reduced by placement of fabrics across the tensile stress zones.

4.6.4 Insulated Gravel Pad

This method is useful to maintain the existing frozen conditions of muskeg in discontinuous permafrost regions. Design approaches discussed by the author (18) may be used to calcualte the gravel and insulation thickness required to prevent the thermal degradation of frozen muskeg. If the ground temperature is close to freezing, the settlement due to creep must be considered and the height of embankment may be limited in such cases.



LEGEND
SURFACE IN GOOD CONDITION
SURFACE IN FAIR CONDITION
SURFACE IN POOR CONDITION
DEFLECTIONS UNDER 18,000 LB. AXLE LOAD.
DEPTH OF PAVEMENT, INCHES
100
80
60
40
20
0
SURFACE IN GOOD CONDITION
SURFACE IN FAIR CONDITION
SURFACE IN POOR CONDITION
0
0.05
0.10
0.15
0.20
0.25
0.30
FIELD DEFLECTION READING, INCHES
(TWICE BENKELMAN BEAM READING)

Figure 4.1

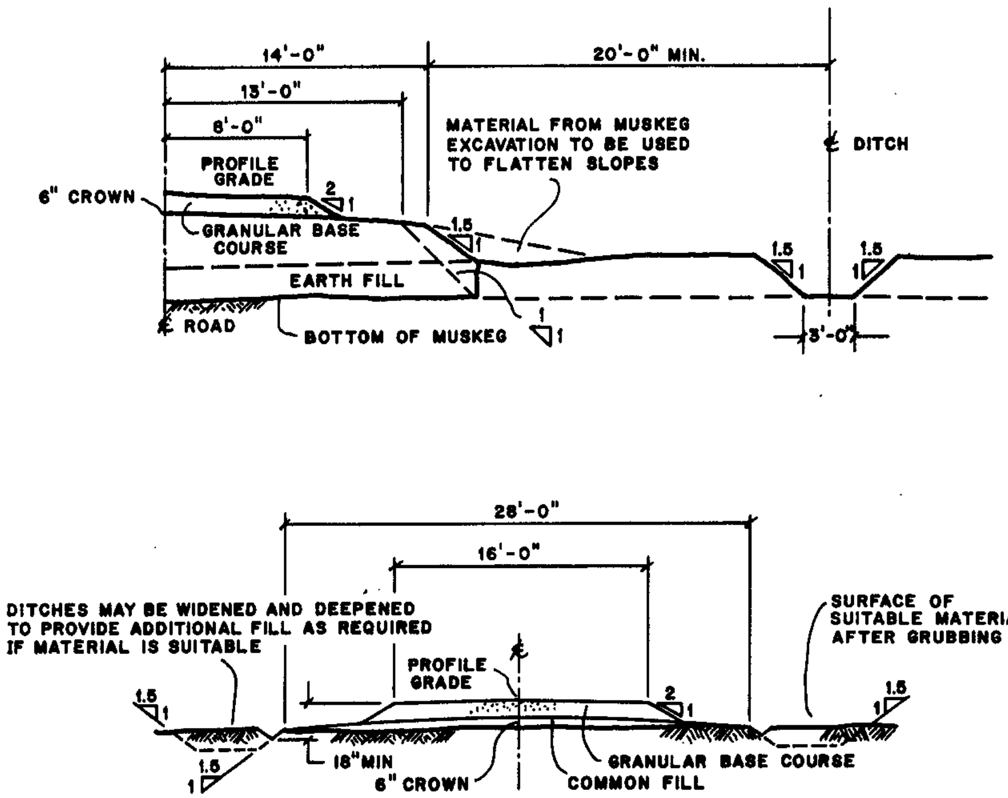
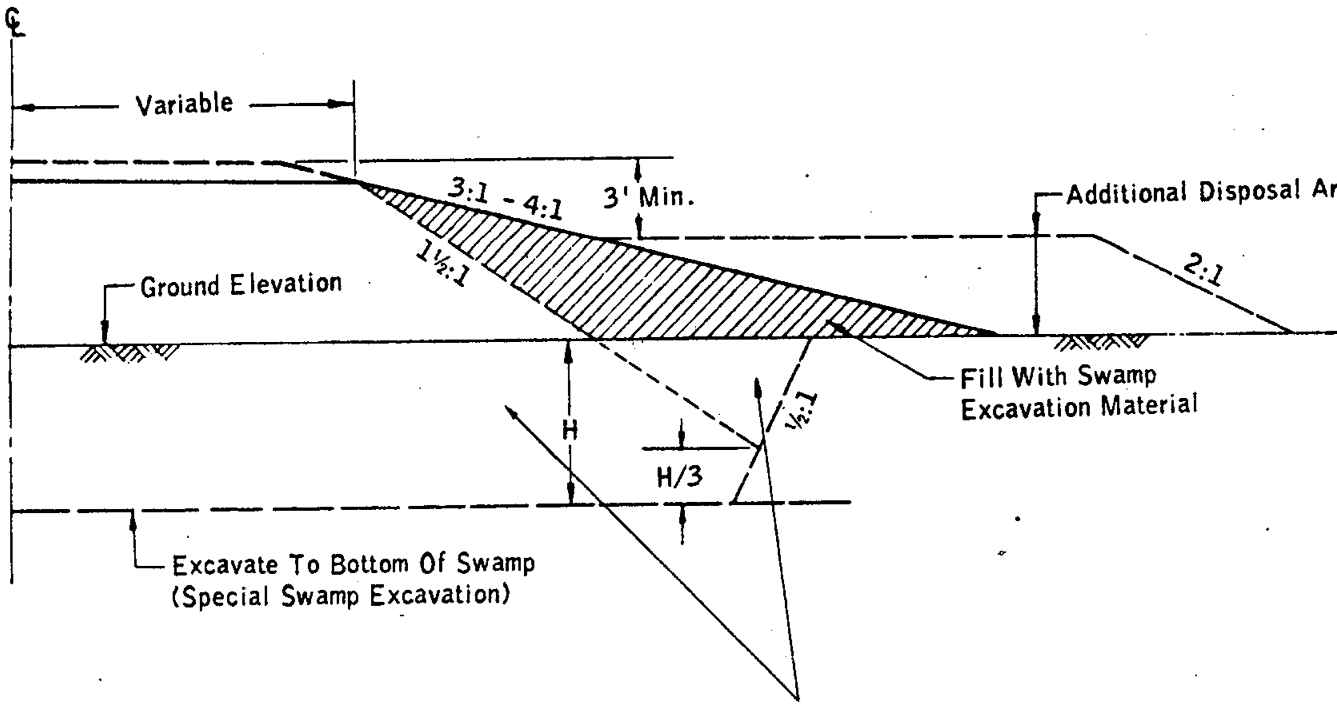
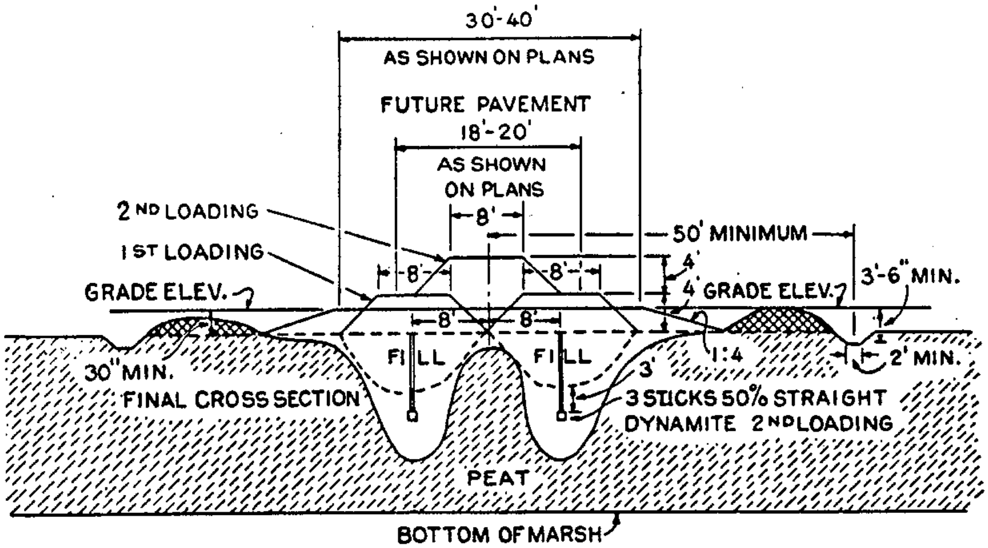

14'-0"
13'-0"
8'-0"
20'-0" MIN.
MATERIAL FROM MUSKEG
EXCAVATION TO BE USED
TO FLATTEN SLOPES
℄ DITCH
PROFILE
GRADE
6" CROWN
GRANULAR BASE
COURSE
EARTH FILL
℄ ROAD
BOTTOM OF MUSKEG
3'-0"
28'-0"
16'-0"
DITCHES MAY BE WIDENED AND DEEPENED
TO PROVIDE ADDITIONAL FILL AS REQUIRED
IF MATERIAL IS SUITABLE
SURFACE OF
SUITABLE MATERIAL
AFTER GRUBBING
PROFILE
GRADE
18"MIN
6" CROWN
COMMON FILL
GRANULAR BASE COURSE

Figure 4.2



Note:
Depths of Swamp Excavation, Backfill Material, And Shrinkage Factor (including Subsidence) To Be Determined By Soils Engineer.

Figure 4.3


30'-40'
AS SHOWN ON PLANS
FUTURE PAVEMENT
18'-20'
AS SHOWN
ON PLANS
2ND LOADING
1ST LOADING
8'
50' MINIMUM
4'
4' GRADE ELEV.
3'-6" MIN.
GRADE ELEV.
30" MIN.
FILL
FILL
1:4
2' MIN.
FINAL CROSS SECTION
3'
3 STICKS 50% STRAIGHT
DYNAMITE 2ND LOADING
PEAT
BOTTOM OF MARSH

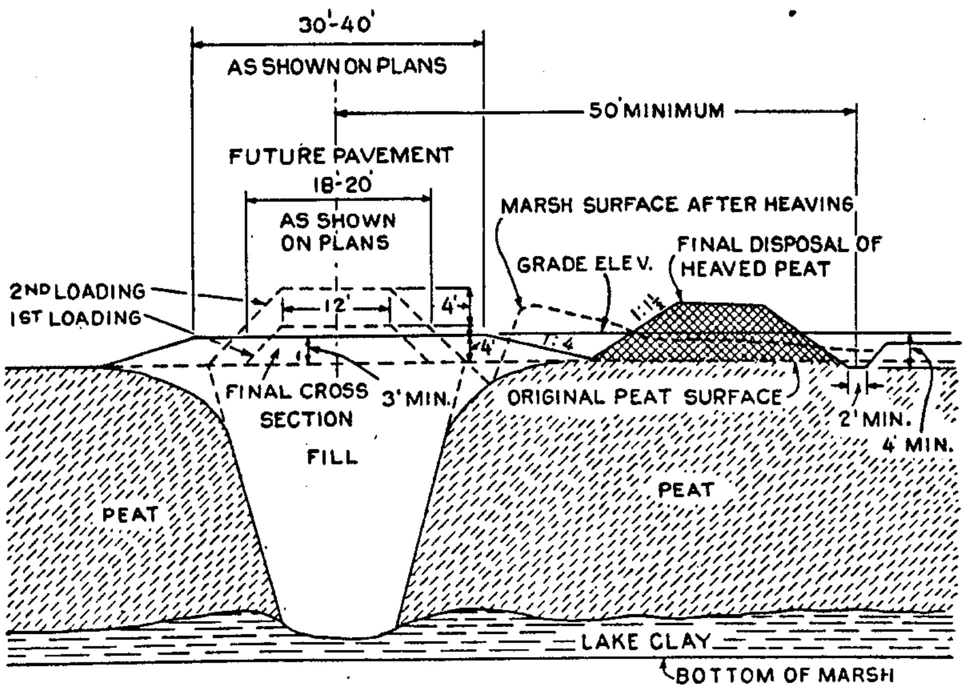
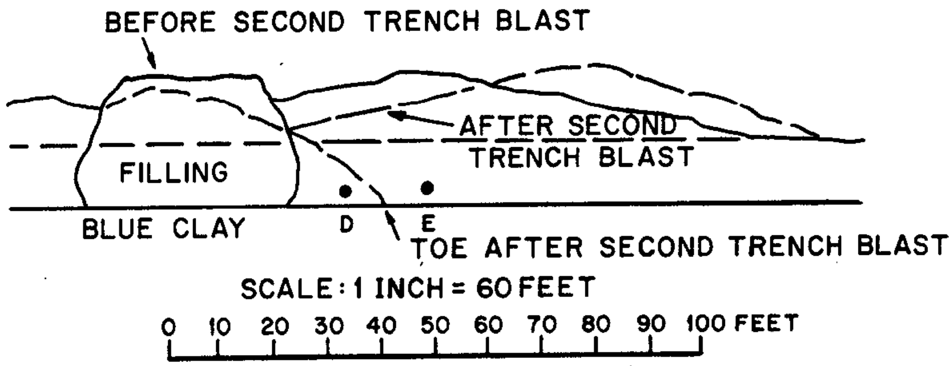

30'-40'
AS SHOWN ON PLANS
50' MINIMUM
FUTURE PAVEMENT
18'-20'
AS SHOWN
ON PLANS
MARSH SURFACE AFTER HEAVING
GRADE ELEV.
FINAL DISPOSAL OF
HEAVED PEAT
2ND LOADING
1ST LOADING
12'
4'
1:4
FINAL CROSS
SECTION
3' MIN.
ORIGINAL PEAT SURFACE
2' MIN.
4' MIN.
FILL
PEAT
PEAT
LAKE CLAY
BOTTOM OF MARSH

Figure 4.4


BEFORE SECOND TRENCH BLAST
AFTER SECOND TRENCH BLAST
FILLING
BLUE CLAY
D
E
TOE AFTER SECOND TRENCH BLAST
SCALE: 1 INCH = 60 FEET
0 10 20 30 40 50 60 70 80 90 100 FEET

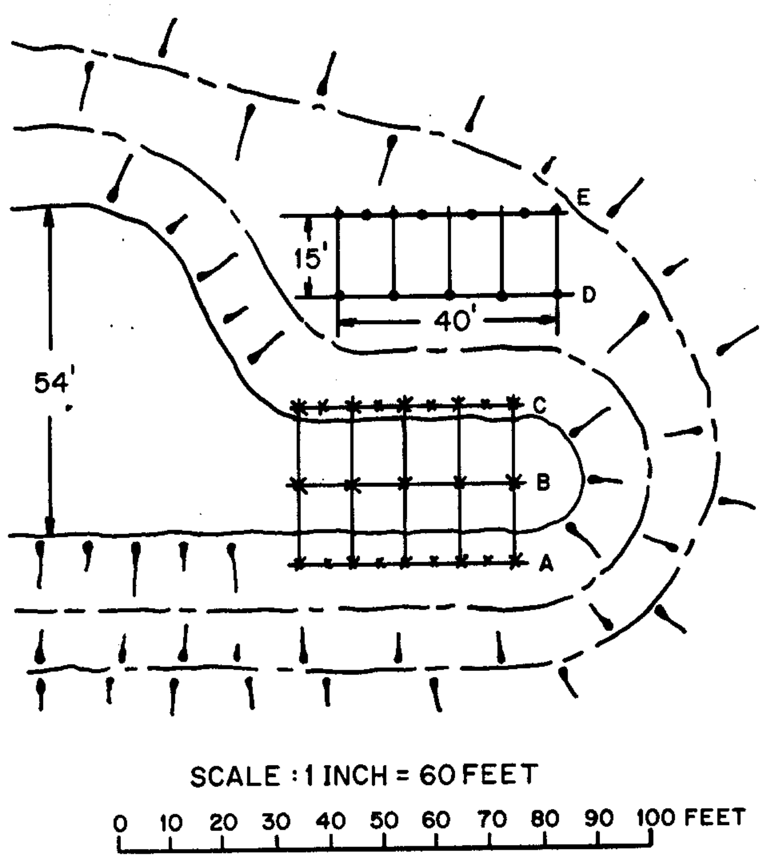

E
15'
D
40'
54'
C
B
A
SCALE: 1 INCH = 60 FEET
0 10 20 30 40 50 60 70 80 90 100 FEET

Figure 4.5

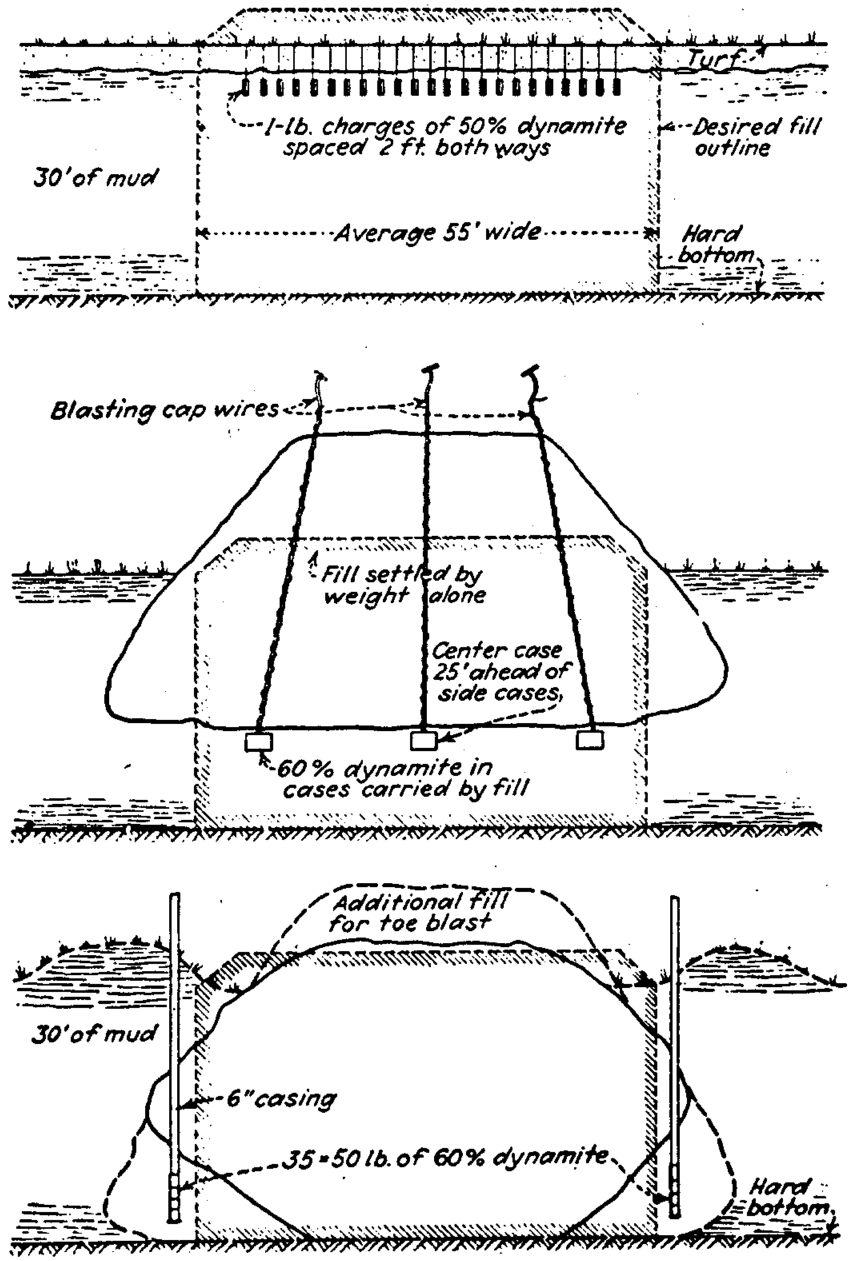
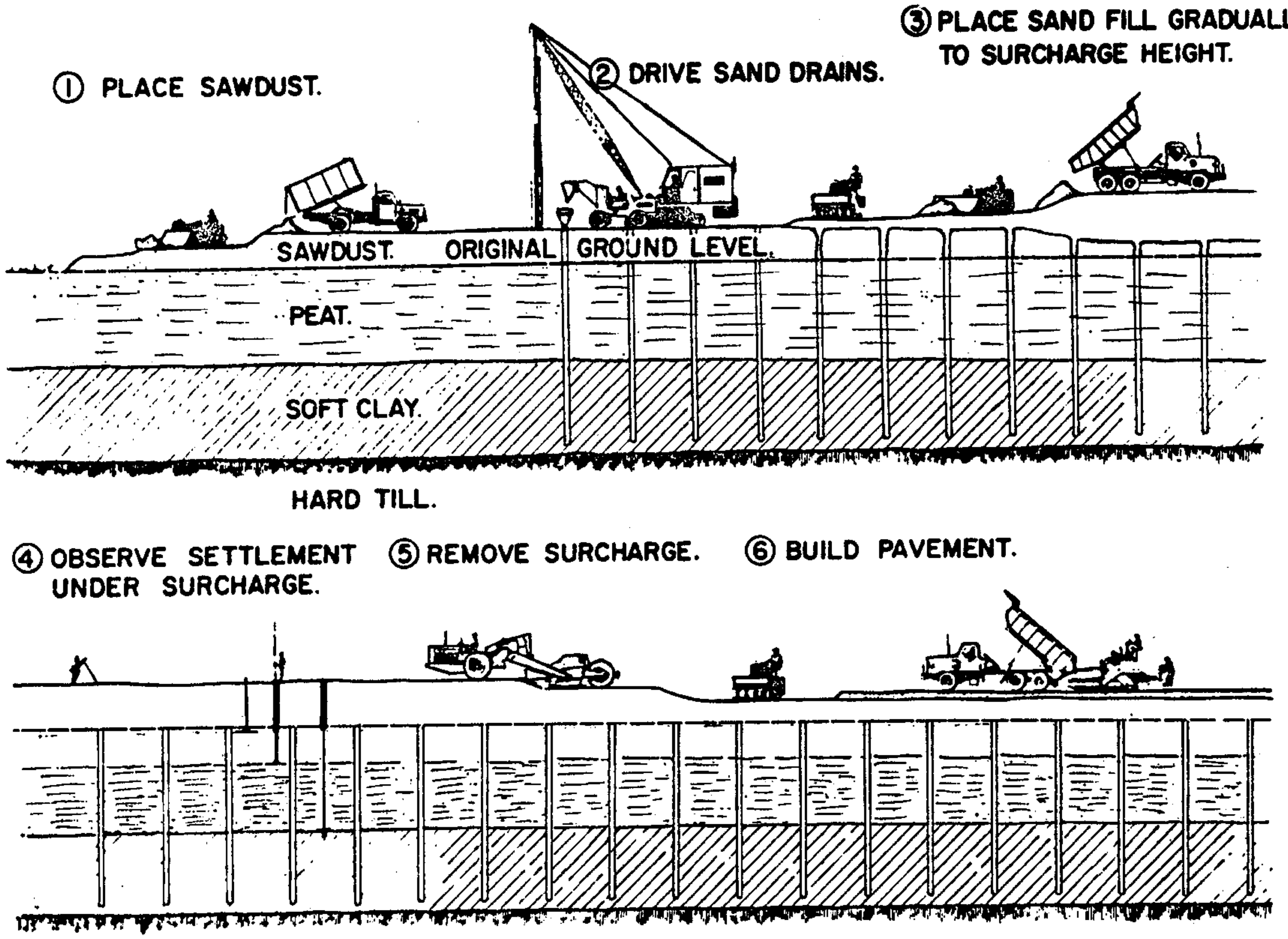
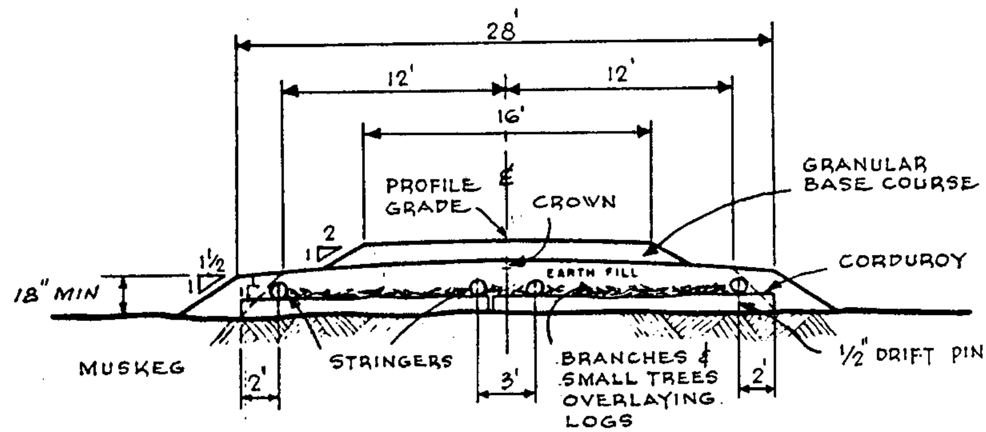

Turf
1-lb. charges of 50% dynamite spaced 2 ft. both ways
Desired fill outline
30' of mud
Average 55' wide
Hard bottom
Blasting cap wires
Fill settled by weight alone
Center case 25' ahead of side cases
60% dynamite in cases carried by fill
Additional fill for toe blast
30' of mud
6" casing
35-50 lb. of 60% dynamite
Hard bottom

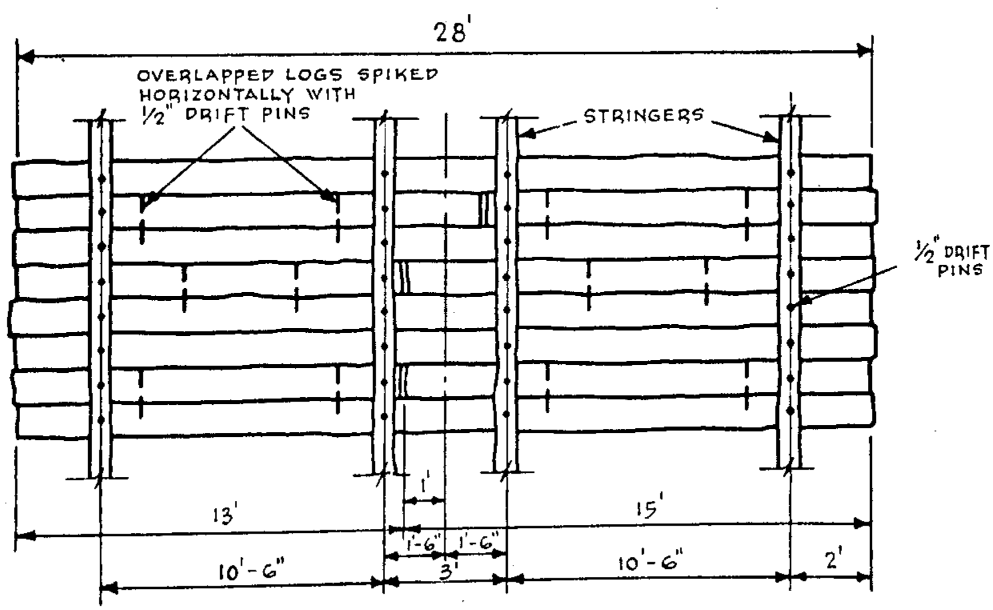
Figure 4.6



① PLACE SAWDUST.
② DRIVE SAND DRAINS.
③ PLACE SAND FILL GRADUALL
TO SURCHARGE HEIGHT.
SAWDUST.
ORIGINAL GROUND LEVEL.
PEAT.
SOFT CLAY.
HARD TILL.
④ OBSERVE SETTLEMENT
UNDER SURCHARGE.
⑤ REMOVE SURCHARGE.
⑥ BUILD PAVEMENT.

Figure 4.7





Plan of corduroy.

Figure 4.8

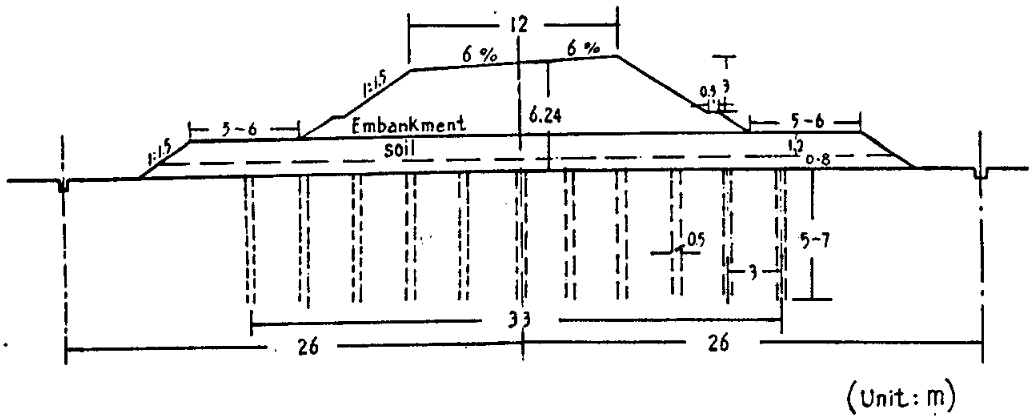

12
6 %
6 %
1:1.5
6.24
0.3
3
Embankment
soil
1:1.5
5~6
5-6
1.2
0.8
0.5
5-7
3
33
26
26
(Unit: m)

Figure 4.9

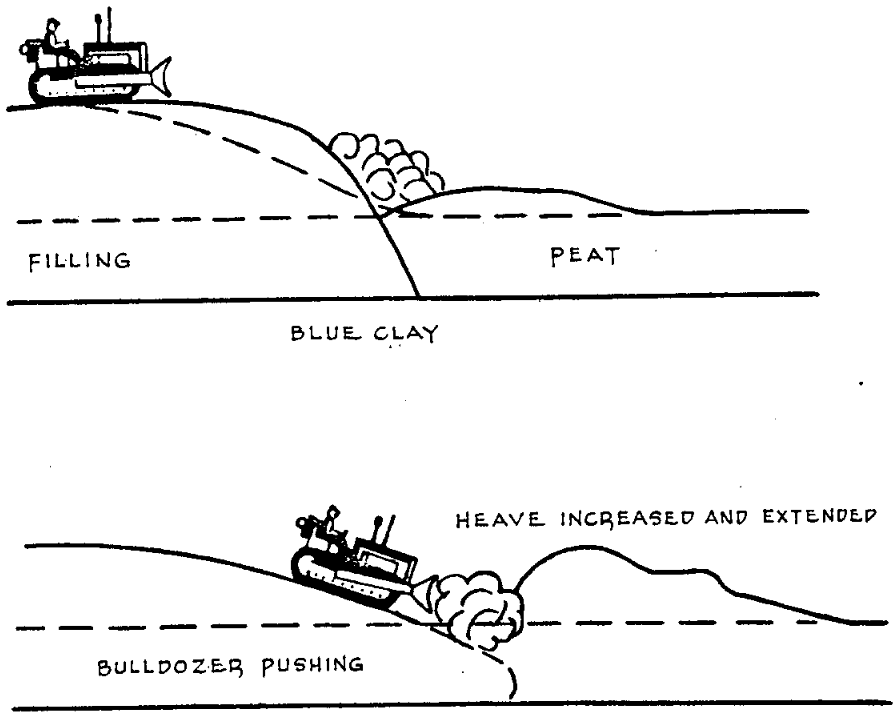

FILLING
PEAT
BLUE CLAY
HEAVE INCREASED AND EXTENDED
BULLDOZER PUSHING

Figure 4.10

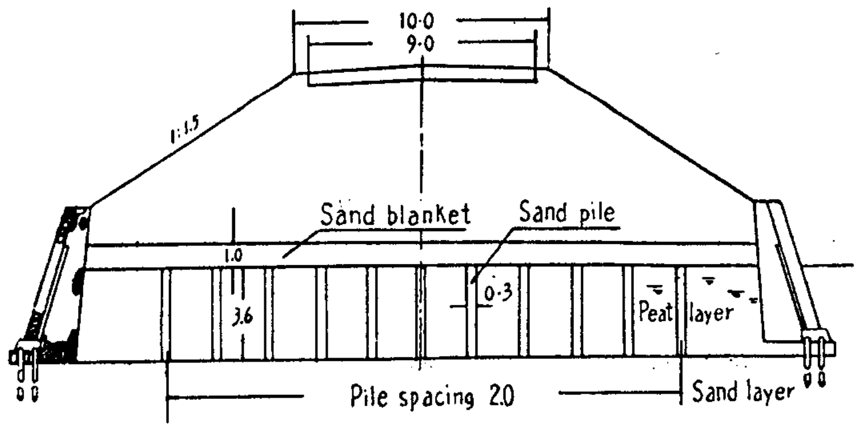

10·0
9·0
1:1.5
Sand blanket
Sand pile
1.0
3.6
0·3
Peat layer
Pile spacing 2.0
Sand layer

Figure 4.11

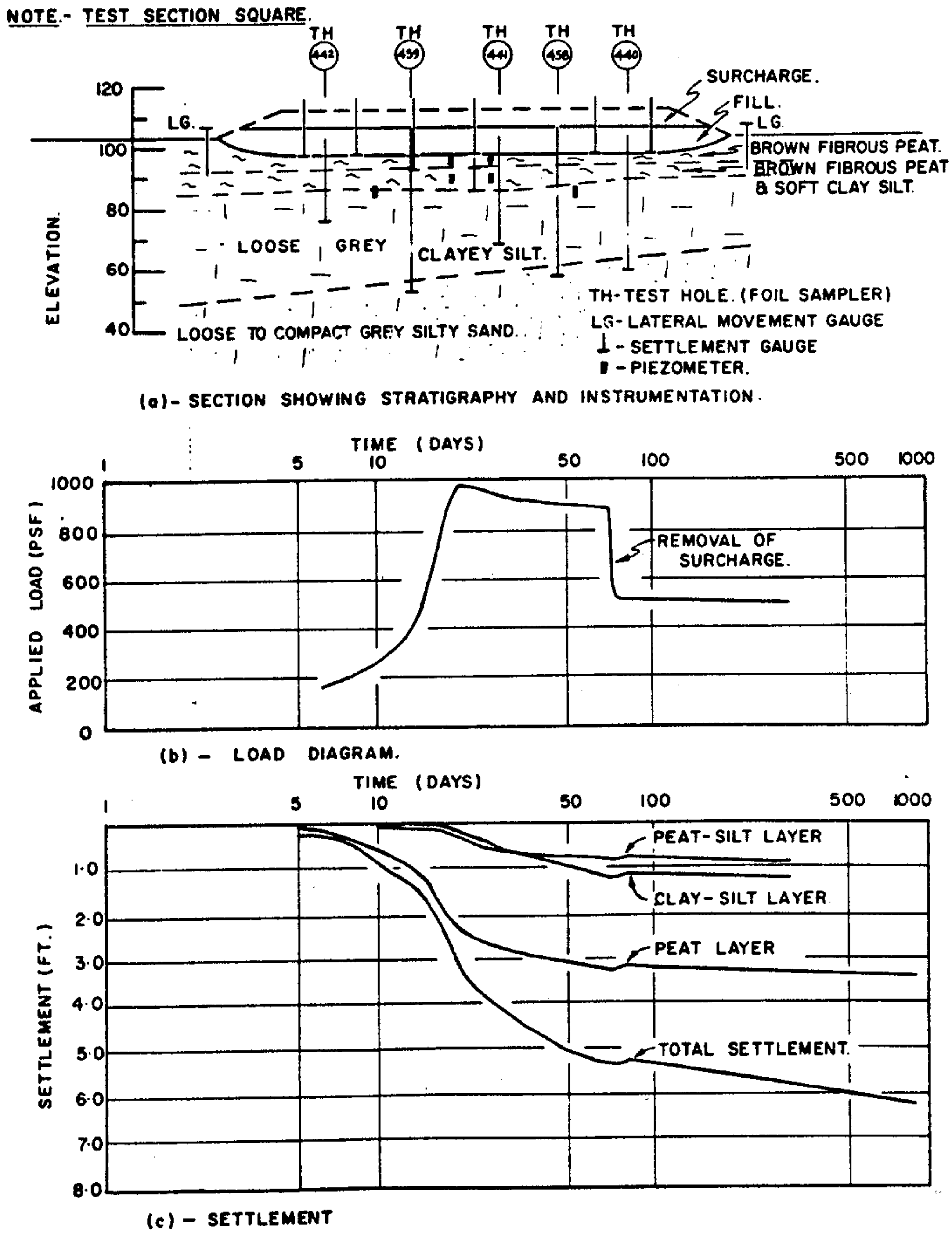

NOTE.- TEST SECTION SQUARE.
TH 442
TH 439
TH 441
TH 456
TH 440
SURCHARGE.
FILL.
LG.
BROWN FIBROUS PEAT.
BROWN FIBROUS PEAT & SOFT CLAY SILT.
ELEVATION.
120
100
80
60
40
LOOSE GREY CLAYEY SILT.
LOOSE TO COMPACT GREY SILTY SAND.
TH-TEST HOLE. (FOIL SAMPLER)
LG- LATERAL MOVEMENT GAUGE
- SETTLEMENT GAUGE
- PIEZOMETER.
(a)- SECTION SHOWING STRATIGRAPHY AND INSTRUMENTATION.
TIME (DAYS)
1
5
10
50
100
500
1000
APPLIED LOAD (PSF)
1000
800
600
400
200
0
REMOVAL OF SURCHARGE.
(b) - LOAD DIAGRAM.
TIME (DAYS)
SETTLEMENT (FT.)
1·0
2·0
3·0
4·0
5·0
6·0
7·0
8·0
PEAT-SILT LAYER
CLAY-SILT LAYER
PEAT LAYER
TOTAL SETTLEMENT.
(c) - SETTLEMENT

Figure 4.12

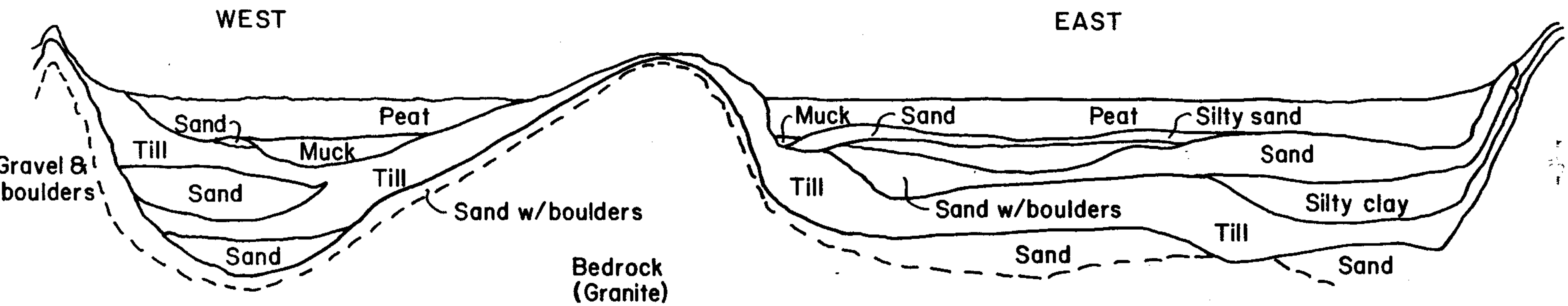

WEST
EAST
Sand
Peat
Till
Muck
Gravel &
boulders
Sand
Till
Sand w/boulders
Sand
Bedrock
(Granite)
Muck
Sand
Peat
Silty sand
Sand
Till
Sand w/boulders
Silty clay
Till
Sand
Sand

Figure 4.13

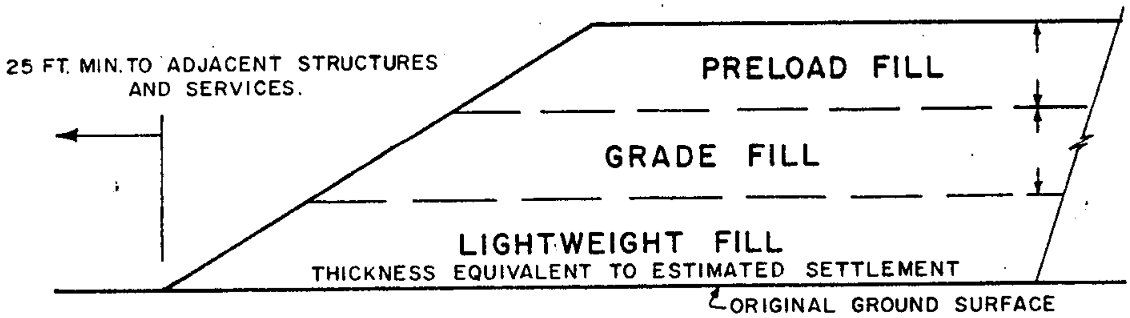
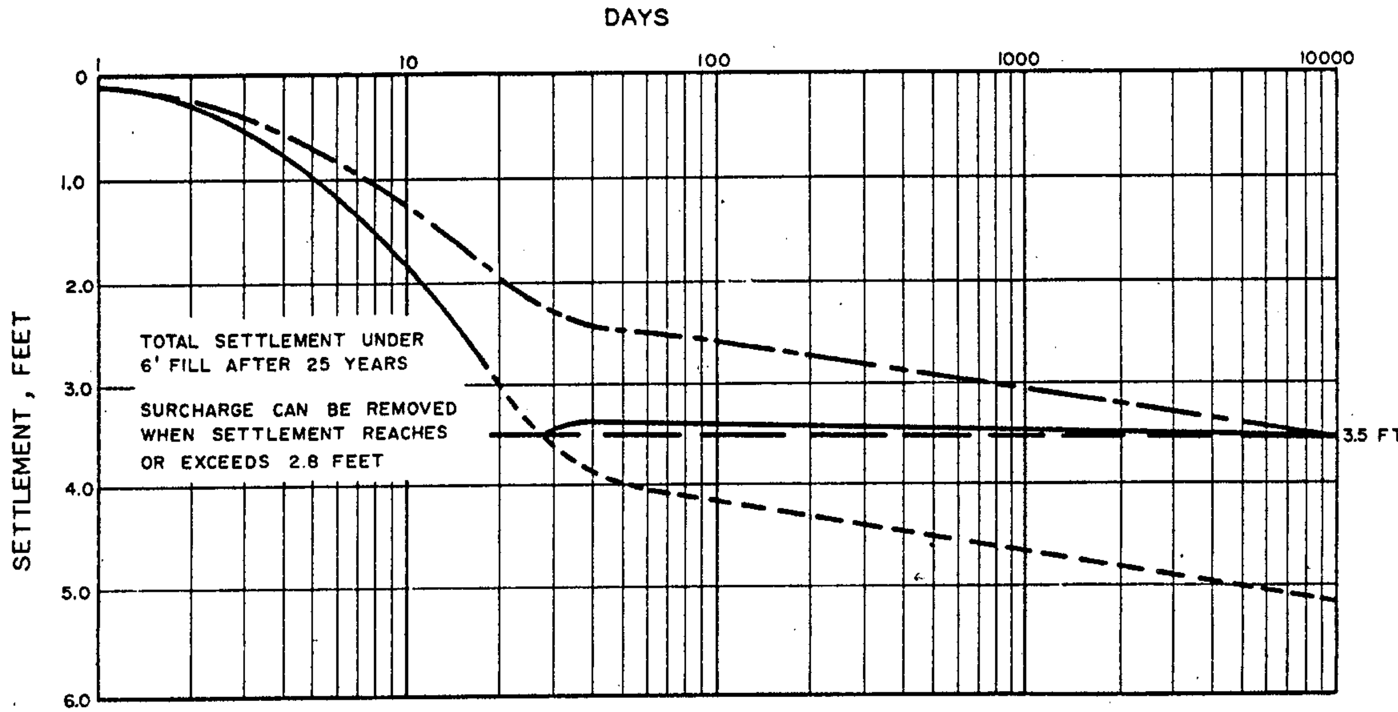

25 FT. MIN. TO ADJACENT STRUCTURES AND SERVICES.
PRELOAD FILL
GRADE FILL
LIGHTWEIGHT FILL
THICKNESS EQUIVALENT TO ESTIMATED SETTLEMENT
ORIGINAL GROUND SURFACE

Figure 4.14



DAYS
1
10
100
1000
10000
0
1.0
2.0
3.0
4.0
5.0
6.0
SETTLEMENT , FEET
TOTAL SETTLEMENT UNDER 6' FILL AFTER 25 YEARS
SURCHARGE CAN BE REMOVED WHEN SETTLEMENT REACHES OR EXCEEDS 2.8 FEET
3.5 FT

Figure 4.15

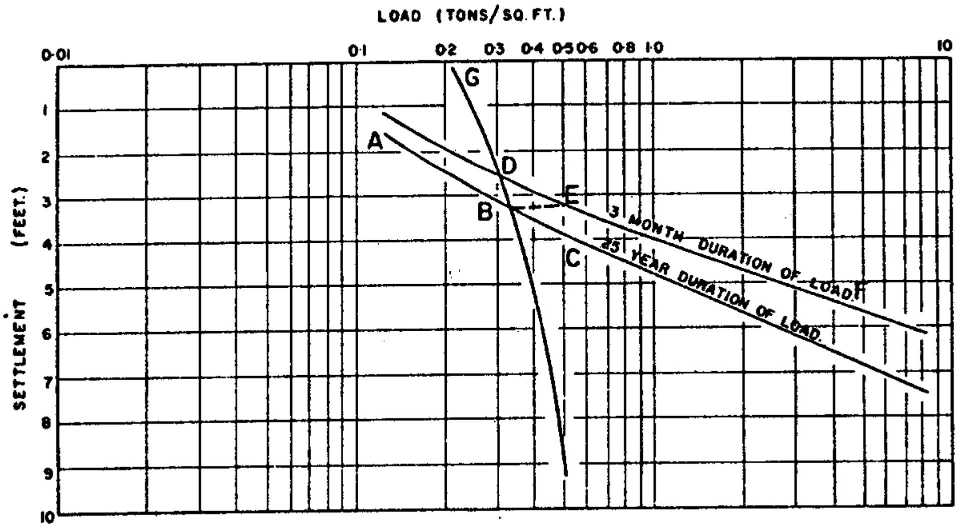

LOAD (TONS/SQ.FT.)
0·01
0·1
0·2
0·3
0·4
0·5
0·6
0·8
1·0
10
1
2
3
4
5
6
7
8
9
10
SETTLEMENT (FEET.)
G
A
D
E
B
C
F
3 MONTH DURATION OF LOAD.
25 YEAR DURATION OF LOAD.

Figure 4.16

5.0 CONSTRUCTION COSTS

5.1 General

The cost of roadway section consists of design and engineering cost, survey right-of-way costs, exploration of sites, testing of materials and quality control of construction. Road construction costs are discussed in this section. Cost per mile is generally used for highway projects. The lack of information and the difficulties of correlating individual available reports make cost estimates extremely difficult.

5.2 Cost Estimate

Table 5.1 presents the construction elements of a typical roadway section. These items shown may be broken down further and individual costs may be calculated in terms of labor, material and equipment. Three types of cost estimates which are commonly used in highway project are preliminary, final "engineers estimate" and contractor's bid type. Several factors such as degree of accuracy required at each stage of construction, lowest cost estimate, amount of load involved in the project and availability of time, should be considered before a particular cost estimate process is selected.

A typical quantity take-off and item cost workup form is shown in Table 5-2. The item cost may be prepared by the use of form shown in Table 5-3.

The Alaska construction cost index is presented in Table 5-4 and the composite price index for the last seven years is shown in Fig. 5-1. For roads over muskeg, the structural requirements of the roadway, depth and strength of the peat and underlying soil, consolidation characteristics of the peat and underlying soil, presence of adjacent structures, drainage requirements, availability of stable granular fill or replacement material, construction timing and control, construction equipment required and so on, influence the construction cost. Maintenance costs involving releveling a nonuniform subgrade, snow removal, drainage and ditch maintenance, slope stability etc. should also be considered for the economic evaluations.

In terms of construction costs, the "flotation" technique described in the previous section is the most inexpensive and the preconsolidation method may be the most expensive. However, in terms of performance, the preconsolidation method gives the best result where "floating the road" may cause serious differential settlements.

TABLE 5-1 ROADWAY CONSTRUCTION ELEMENTS

Site Preparation	Grading	Drainage	Fill or Overlay	Subbase	Base Course	Asphalt Pavement	Misc Items
Clear	Excavation	Trench Excavation	Borrow Material	Borrow Material	Aggregate Preparation	Aggregate Preparation	Borrow Material
	Borrow Material	Underdrain	Spreading	Spreading	Spreading	Aggregate Mix & Spread.	Placement
		Culvert	Compaction	Compaction	Compaction	Compaction	

TABLE 5-2 QUANTITY TAKE-OFF AND ITEM COST WORKUP

General Estimate
Combination Quantity Take-offs and Item Cost Workup Sheet

Project ____________________	Take-off By ____________	Estimate No. __________
Location ____________________	Extensions By ____________	Sheet No. __________
Architect-Engineer ____________	Price By ____________	Date __________
Classification ______________	Checked By ____________	Date Due __________

Description	No.	Dimensions	Item	Quantity	Unit	Unit Price	Material Cost	Unit Price	Labor Cost	Total

TABLE 5-3 ITEM COST WORKUP

General Estimate
Item Cost Workup

Item__________________ Sheet_________ Date_______

Job No. & Name_______________ Est. No._______ By_________

Field Change No.__________ Plan Dev. No._______ Extra Work Directive No.____

Description	Quantity	Unit Labor	Unit Material	Labor	Materials	Remark

Unit of Equipment	Hrs.	Rate	Amount	Summary

Labor______________

Materials__________

Equipment__________

Total______________

YEAR	UNCL. EXC. Bid Price	UNCL. EXC. Index	BORROW SEL.-MAT. Bid Price	BORROW SEL.-MAT. Index	AGG. BASE Bid Price	AGG. BASE Index	ASPHALT PAVEMENT Bid Price	ASPHALT PAVEMENT Index	ROADWAY INDEX	STRUCT. STEEL Bid Price	STRUCT. STEEL Index	REINFORCING STEEL Bid Price	REINFORCING STEEL Index	FT.^{2} BRIDGE DECK Bid Price	FT.^{2} BRIDGE DECK Index	BRIDGE INDEX	COMPOSITE INDEX
1967	1.01	100	0.62	100	4.55	100	13.65	100	100	0.53	100	0.29	100	40.70	100	100	100
1968	1.00	99	1.15	185	3.73	82	18.93	139	134	0.48	91	0.30	103	22.67	56	68	131
1969	0.95	94	0.63	102	3.51	77	16.74	123	99	0.54	102	0.27	93	36.44	90	94	98
1970	2.08	206	0.99	160	3.32	73	15.39	113	148	0.47	89	0.28	97	47.82	117	109	133
1971	1.32	131	1.40	226	3.80	84	10.78	79	139	0.58	109	0.41	141	30.52	75	82	134
1972	1.15	114	0.90	145	3.54	78	12.70	93	113	0.64	121	0.30	103	41.25	101	104	116
1973	1.61	159	1.32	213	4.68	103	15.50	114	162	0.53	100	0.26	90	44.98	111	104	132
1974	2.94	291	1.80	290	6.51	143	19.03	139	263	1.11	209	0.88	303	44.63	110	174	231
1975	2.53	250	2.18	352	6.94	153	24.26	178	278	1.10	208	0.63	217	56.14	138	156	241
1976	2.39	237	2.85	460	6.16	135	20.24	148	279	0.91	172	0.50	172	69.62	171	171	253
1977*	1.93	191	3.03	489	6.88	151	24.14	177	288	0.86	162	0.73	252	76.96	189	189	266
1977	2.07	205	3.05	492	7.35	162	20.67	151	267	1.06	200	0.73	252	81.63	201	201	255
1978*	2.25	223	2.49	402	7.85	173	27.41	201	270	**			**	97.13	239	239	270
1978	2.39	237	2.95	476	7.90	174	21.02	154	310	2.07	357	0.52	179	65.68	161	168	295
1979*	3.74	370	2.56	413	7.77	171	20.74	152	360	1.35	255	2.77	955	108.36	266	259	337
1979	3.11	308	3.80	613	8.09	178	19.26	141	334	1.43	270	0.94	324	88.57	218	233	319
1980*	2.58	255	3.82	616	9.14	201	17.66	129	392	**		0.89	307	86.53	213	216	370
1980	2.50	248	2.92	471	9.26	204	19.69	144	291	2.58	487	0.80	276	85.37	210	274	289
1981*	4.69	464	4.39	708	11.66	256	25.94	190	412	1.91	360	1.03	355	96.04	236	268	397

Bid Price is weighted average price for period indicated.

* First Half

** No Work Bid

6.0 RECOMMENDATION

6.1 General

It has been shown that various design and construction concepts are available for roadways in muskeg areas. The following is a brief assessment of the topics on which the current state of the art seems to be most lacking, and on which further research has the most promise of improving the cost effectiveness of roadway design in muskeg areas.

6.2 Research on Shear Strength and Consolidation Characteristics of Peat

The shear strength of peat warrants a considerable amount of additional study. The further development of in situ tests and the documentation of actual shear failures are considered to be a first approach to this end.

At present there is no sufficient information for definite conclusions on many aspects of consolidation. Many questions on primary and secondary consolidation, rate of settlement, and plastic flow are still not fully answered.

6.3 Research on Engineering Classification of Muskeg

A research program should be undertaken with the aim of classifying peat by its engineering properties rather than by qualitative and visual methods.

6.4 Research on Different Construction Techniques

The importance of drainage related to various construction techniques on muskeg is universally acknowledged. The preconsolidation method combined with effective drainage systems appears to be the most promising to reduce the anticipated settlement of roadway embankment on muskeg. Such technique needs studies for different muskeg conditions and climatic regions.

The effectiveness of fabric membrane and soil reinforcement in different positions of roadway sections needs to be assessed. Specifically, the potential use of reinforced embankments to prevent side slope movements and cracking should be evaluated.

6.5 Research on Factors Fundamental to Energy Exchange

There is an interaction between muskeg and permafrost. The role of vegetation and its influence on the energy exchange at the ground surface as well as the thermal properties of peat warrant considerable study. It is especially applicable where the construction on muskeg may cause aggregation or degradation of permafrost.

7.0 REFERENCES

1. Korpijaakko, E.O. and Woolnoug, D.F., 1973. Peatland and Survey and Inventory, Chapter 3, Muskeg and the Northern Environment in Canada, Published by University of Toronto.

2. Muskeg Engineering Handbook, edited by Ivan C. MacFarlane and published by University of Toronto Press, 1969.

3. Lea, N.D. and Brawner, C.O., 1963. Highway Design and Construction Over Peat Deposits in Lower British Columbia, Highway Res. Board No. 7, Washington, D.C. pp. 1-33.

4. Hawson, H.H. and Butler, R.C., 1975. Concepts in the Reclamation and Development of Muskeg Terrain, Proc. Sixteenth Muskeg Research Conf., Montreal, Oct. 7, NRC Publication.

5. Cook, P.M., 1956. Consolidation Characteristics of Organic Soils, Proc. Ninth Con., Soil Mec. Conf., NRC ACSSM Tech. Memo. 41, pp. 82-87.

6. Mickleborough, B.W., 1961. Embankment Construction in Muskeg at Prince Albert, Proc. Seventh Muskeg Res. Conf., NRC ACSSM Tech. Memo. 71, pp. 164-185.

7. Lieszkowszky, L.P. and Ozden, Z.S. 1977. The Effects of Volume Change and Alkalinity of Water on the Permeability of Peat, Proc. Seventeenth Muskeg Research Con., NRC Tech. Memo. 122, pp. 25-36.

8. Wiesner, W.R. and Hardy, R.M., 1977. Design and Construction Experiences with Fluid Retaining Embankments on Peat, Proc. Seventeenth Muskeg Res. Conf., NRC ACSSM Tech. Memo. 122.

9. Raymond, G.P., 1969. Construction Method and Stability of Embankments on Muskeg, Can. Geo. J. Vol-6.

10. Wilson, N.E., 1977. The Contribution of Fibrous Interlock and the Strength of Peat, Proc. Seventeenth Muskeg Conf., NRC ACSSM Tech . Memo. 122.

11. Hanrahan, E.T., 1952. The Mechanical Properties of Peat with Special Reference to Road Construction, Bull. Ins. of Civil Engineers of Ireland Vol. 78, No. 5, pp. 179-215.

12. Hanrahan, E.T., 1954. An Investigation of Some Physical Properties of Peat, Geotechnique, Vol. 4, No. 3. pp. 108-123.

13. Anderson, K.O. and Hemslock, R.A., 1959. Relating Some Engineering Properties of Muskeg to Some Problems of Field Construction, Proc. Fifth Muskeg Res. Conf., NRC ACSSM Tech. Memo. 61.

14. Muskeg and the Northern Environment in Canada, 1973. Edited by N.W. Radforth and C.O. Brawner, University of Toronto Press.

15. Mollard, J.D., 1960. The Role of Photo Interpretation in Areas Containing Muskeg and Permafrost, Sixth Muskeg Research Con., NRC ACSSM Tech. Memo. 61.

16. Radforth, N.W., 1955. Organic Terrain Oragnization from the Air, Handbook No. 1, Dept. of Nat. Defense, Canada, Defense Res. Board, DR 95.

17. Brawner, C.O., 1959. Summary of the Principle of Preconsolidation in Highway Construction Over Muskeg, Proc. Fifth Muskeg Research Conf.. NRC ACSSM Tech. Memo. 61, pp. 13-15.

18. Phukan, A., 1981. Design Guide for Roadways on Permafrost, Report to State of Alaska, DOTPF, Div. of Planning and Programming, Oct. 81.

19. DOTPF Report, Material Section, Anchorage.

20. Casagrande, L., 1966. Construction of Embankments Across Peaty Soils J. Boston Soc. Civil Engr., Vol. 53, No. 3, pp. 272-317.

21. Jarrett, P.M., 1975. The Strength of the Surface Mat in a Fibrous Peat, Proc. Sixteenth Muskeg Res. Conf., NRC ACSSM Tech. Memo. 116.

22. MacFarlane, L.C. and Rutka, A., 1962. An Evaluation of Pavement Performance Over Muskeg in Northern Ontario, Highway Res. Board Res. Bull. 316, Washington, DC., pp. 32-43.

23. Moore, L.H., 1962. A Correlation of the Engineering Characteristics of Organic Soils in New York State (preliminary). New York State Dept. Public Works, Bureau of Soil Mech., Tech Rept. 13 leaves.

24. Tressider, J.O., 1958. A Review of Existing Methods of Road Construction over Peat, DSIR. RRI, Tech. Paper No. 40. London.

25. Brochu, P.A. and Pare, J.J., 1964. Construction de Routes Sur Tourbieres Dans La Province de Quebec, Proc. Ninth Muskeg Res. Conf., NRC ACSSM Tech. Memo. 81, pp. 74-108.

26. Miyakawa, I. 1959a. Soil Engineering Research on Peaty Alluvia: Reports 1 to 3, Civil Eng. Res. Inst., Hokkaido Development Bureau Bull. No. 20, Sapporo, 88 pp. English Transl. by K. Shimizu, NRC Transl. No. 1001, Ottawa, 1962.

27. Radforth, N.W. and Rush, E.S., 1964. Trafficability Tests on Confined Organic Terrain (Muskeg): Report I: Summer 1961 Tests, U.S. Army Corps of Engrs., Waterways Exp. Sta., Tech Rept. No. 3-656, Vicksburg, Miss., 110 pp.

28. Ripley, C.F.., and Leonoff, C.E., 1961. Embankment Settlement Behavior on Deep Peat, Proc. Seventh Muskeg Res. Conf., NRC ACSSM Tech. Memo. 71 pp. 185-204.

29. Flaate, K. and Rygg, N., 1964. Sawdust as Embankment Fill on Peat Bogs, Proc. Ninth Muskeg Res. Conf., NRC ACSSM Tech. Memo. 81 pp. 136-149.

30. Irwin, R.W., 1966. Soil Water Characteristics of Some Ontario Peats, Proc. Third Int. Peat Congress, pp. 219-223.

31. Padbury, G.A., 1970. Properties of Organic Soils in Relation to Classification, M. Sc. Thesis, Dept. of Soil Sci. Univ. of Saskatchewan

32. Kapp, M.S., York, D.L., Aronowitz, A., and Sitomer, H., 1966. Construction on Marshland Deposits: Treatment and Results, Highway Res. Board, Highway Res. Rec. No. 133, Washington, D.C., pp.1-22.

33. Skempton, A.W. and Petley, D.J., 1970. Ignition Loss and Other Properties of Peat and Clay from Avonmonth, King's Lynn and Cranberry Moss., Geotechnique 20, No. 4, pp. 343-356.

34. Flaate, K., 1965. The Classification of Peat and Peat Deposits for Engineering Purposes. The Eleventh Muskeg Research Conference, Sudbury, Ont. NRC ACAR Tech. Memo. 87.

35. Koerner, R.M. and Welsh, J.P. 1980. Construction and Geotechnical Engineering Using Synthetic Fabrics, Published by John Wiley & Sons.

36. Bell, J.R. et al., 1977. Construction and Analysis of a Fabric Reinforced Low Embankment, CR. Con. Inst. Soils Text, Vol. 1, pp. 71-75.

37. Vischer, W., 1975. Use of Systematic Fabrics on Muskeg Subgrades in Road Construction, U.S. Dept. of Agriculture, Forest Service, Region 10 Report.

38. Steward, J., Williamson, R., and Hohney, J., 1977. Guidelines For Use of Fabrics in Construction and Maintenance of Low Volume Roads, Forest Service, Pacific Northwest Region, U.S. Dept. of Agriculture, Washington, D.C.

39. duPont, E.I., A Method for Constructing Aggregate Bases Using "Typar" Spunbonded Polypropylene, Unpublished Report.

40. American Society of Civil Engineers, 1978. Soil Improvement: History Capabilities and outlook.

41. American Society of Civil Engineers, 1978. Symposium on Earth Reinforcement. Pittsburg.

Acknowledgements

This report is completed under the financial assistance provided by the State of Alaska, Department of Transportation and Public Facilities, Division of Planning and Programming, Research Section in co-operation with U.S. Department of Transportation, Federal Highway Administration.

The author gratefully acknowledges Mr. David C. Esch, P.E., Project Manager of this project and chief of Highway Research Section, for his guidance on the project. The author also acknowledges Mrs. Joan Cook for her assistance in typing the manuscript and Mr. Jim Barton for drafting the figures.

GLOSSARY OF TERMS

Acidity See pH

A-line In a plasticity chart, the A-line represents the empirical boundary between typical inorganic clays which are generally above this line, the the plastic soils containing organic colloids which are below it.

Amorphous-granular A descriptive term applied to one of the primary macroscopic elements of peat which is granular in nature but to which no particular shape can be ascribed.

Ash content The ash or mineral residue remaining after a peat sample has been ignited, expressed as a precentage of the dry weight; also known as ignition loss. Expressed as ratio of dry weight it is known as ignition loss ratio.

Axon A well-preserved non-woody, fossilized plant component of peat, consisting of tubular axis, system of leafy appendages, and with the cell structure clearly defined. The maximum outside diameter of the linear component of the axon (when wet) is 1 mm.

Bog An area of confined organic terrain, the limits of which are imposed by the physiogrpahy of the local mineral terrain. Differential from "muskeg" mainly in terms of area but often because variations in coverage, peat structure, and topography occur more frequently than in extensive areas of organic terrain.

Bog, blanket Equivalent of blanket mire, also "Terrainbedeckendmoore." Peat formation initiated in basins, drainage axes, and on all water partings where the drainage slope is not too great, the peat forming a blanket over all gently undulating terrain. In Ireland it includes a variety of peat-land types, both ombrogenous (water supplied by precipitation) and soligenous (water supplied by high water table), occuring between 200 and 700 m. Variation in surface vegetation is related to climate (and perhaps supply of atmospheric nutrients), geology, topography, state of erosion, land use, etc.

Bog mire Confined organic terrain: equivalent to bog.

Bog raised Evquivalent to raised mire and "Hoochmoore." Peat development initiated in basins, peat growth producing a dome or cupola rising above the mineral ground water table. The classic case is the Baltic raised bog with ring lagg.

Box, spruce See Spruce bog

Coarse-fibrous A descriptive term applied to one of the primary macroscopic elements of peat which may be woody or non-woody and has a diameter greater than 1 mm.

Coefficient of compressibility A stress-strain ratio of a soil. Numerically it is the slope of the void ratio - pressure curve from a consolidation test.

Coefficient of consolidation Obtained by plotting degree of consolidation against square root of time for any particular consolidation test.

Coefficient of secondary compression Is expressed as the shape of the settlement - log time plot divided by the thickness of the peat sample at the beginning of the long-term or sgraight-line stage: is also expressed in terms of change in void ratio over 1 cycle of the logarithmic scale on a plot of void ratio vs. log time.

Compression index The slope of the void ratio - log pressure curve from a consolidation test. The larger the compression index the greater the cmpressibility of a soil.

Cone index An index of the shearing resistance of the soil obtained with the cone penetrometer. The number, although usually considered dimensionless in trafficability studies, actually denotes pounds of force on the handle divided by the area of the cone base in square inches.

Consolidation, primary The gradual compression of a soil due to a weight acting on it, which occurs as water is squeezed out of the voids in the soil.

Muskeg The term designating organic terrain, the physical condition of which is governed by the structure of peat it contains, and its related mineral sublayer, considered in relation to topogrpahic features and the surface vegetation with which the peat co-exists. (See Organic terrain)

Organic terrain A tract of country comprising a surficial layer of living vegetation and a sublayer of peat or fossilized plant detritus of any depth, existing in association with various hydrological conditions and underlying mineral formations.

Peat A component of oragnic terrain consisting of more or less fragmented remains of vegetable matter sequentially deposited and fossilized.

pH The measure of soil acidity. Defined as the negative logarithm of the hydorgen ion concentration in an aqueous suspension of the soil.

Preconsolidation A construction technique whereby a load in excess of that which will be finally carried by the soft stratum is placed and allowed to settle until the ultimate settlement that would occur under the final load has been reached. The excess load or surcharge is then removed and the construction is completed; also called preloading.

Spruce bog A term in common use loosely applied to confined areas of organic terrain where coniferous trees (often not spruce) are a prominent feature of the vegetal coverage.

Swamp Similar to marsh but usually with higher water table and interruptions in the vegetal mat.

Thermal conductivity An empirical coefficient defined as the number of calories per second flowing through a plate 1 cm square and 1 cm thick with a temperature difference of 1°C between the two surfaces of the plate.

Thermal diffusivity An index of the facility with which a substance will undergo a temperature change.