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DESIGN CONSIDERATION FOR ROADWAYS ON PERMAFROST

FINAL REPORT

by

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16. Abstract

Numerous design conceptions and construction techniques regarding roadways on permafrost have been published recently. In addition to varying climatic factors, difficult subsurface conditions are encountered in permafrost environments. This design guide was prepared to include all aspects of permafrost engineering pertinent to roadways, to bridge the gap between the latest works and the design principles used previously.

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TABLE OF CONTENTS

PAGE

1.0 INTRODUCTION 1

1.1 Scope. 1
1.2 Summary. 3

2.0 EXPLORATION OF DATA 7

2.1 General. 7
2.2 Permafrost Distribution. 7
2.3 Climatic Factors & Ground Temperature Profile. 8
2.4 Frost Action 8
2.4.1 Frost Heaving Process. 9
2.4.2 Thaw Weakening 10
2.5 General Soil and Frost Susceptibility Classification . . 10
2.6 Characterization Materials 11
2.7 Stress Distribution. 13
2.8 Site and Route Exploration 15

3.0 DESIGN METHODS 33

3.1 General. 33
3.2 Design Methods for Cold Regions. 33
3.2.1 Roadway Design Under Seasonal Frost Area 34
3.2.1.1 Complete Protection. 34
3.2.1.2 Limited Subgrade Frost Protection. 35
3.2.1.3 Reduced Subgrade Thaw Protection 36
3.2.1.4 Material Characterization. 37
3.3 Roadway Design Under Permafrost Areas. 37
3.3.1 Depth of Thaw Penetration or Freezing. 38
3.3.2 Non-Frost Susceptible (NFS) Subgrade 42
3.4 Complete Protection Method 43
3.5 Reduced Subgrade Strength Method 43
3.6 New Design Concepts. 44
3.6.1 "Excess Fines" Concept 44
3.6.2 Design Based on Resillient Modules. 48
3.6.3 Frost Protection Layers.. 48

TABLE OF CONTENTS (continued)

		PAGE
4.0	ROADWAY CONSTRUCTION ON PERMAFROST REGIONS	68
4.1	General.	69
4.2	Embankment Construction.	69
4.3	Subgrade Preparation	71
4.4	Fill Materials	72
4.5	Cut Sections	73
4.6	Other Construction Techniques.	74
4.6.1	Insulated Embankments.	74
4.6.2	Peat Overlay Construction Method	76
4.6.3	Membrane Encapsulated Soil Layer (MESL).	77
4.6.4	Prethawing Methods	78
4.6.5	Soil Reinforcement	79
5.0	CONSTRUCTION COSTS	99
5.1	General	99
5.2	Cost Estimates.	99
6.0	RECOMMENDATIONS	105
6.1	General	105
6.2	Research on Factors Fundamental to Frost Susceptibility to Soils.	105
6.3	Research on Factors Fundamental to Thaw Weakening . . .	106
6.4	Research on Material Characterization	106
6.5	Research on Different Construction Techniques	106
7.0	RREFERENCES	108
ACKNOWLEDGEMENTS		112
GLOSSARY OF TERMS.		113
APPENDIX A	DEPTH OF THAW PENETRATION OR FREEZING CALCULATIONS . . .	118

1. INTRODUCTION

1.1 Scope

Recently the construction activities related to resource development including petroleum, mining and hydro-electric works in the permafrost regions of the world have increased significantly, primarily as a result of world concern for future energy and mineral resources. Although the north has been committed to resource development since human occupation, it is only within recent years that sufficient transportation facilities have become available to make extensive development possible.

In Alaska, where approximately 80% of the land mass consists of permafrost, roadways have been built on permafrost to serve the transportation needs and the demand for the future expansion of roadways in such environment will increase as prospective engineering projects are developed. The performance of the existing roadways on permafrost has been unsatisfactory and the maintenance cost has been high. This is primarily due to various factors, such as frost action and thaw-settlements during the freezing and thawing periods. During the freezing and thawing cycles, the pavement serviceability is seriously affected by various modes of pavement distress unless proper long term design and construction techniques are used. The requirement for pavement serviceability throughout the year in permafrost regions imposes special consideration of roadway design and maintenance. Modes of distress in pavements are presented in Table 1-1.

Numerous design conceptions and construction techniques regarding roadways on permafrost have been published recently. In addition to varying climatic factors, difficult subsurface conditions are encountered in permaforst environment. A design guide to include all aspects of permafrost engineering pertinent to roadways is desired to bridge the gap between the latest works and the design principles used previously. It is anticipated, therefore, that the manual will play an important role to reduce the cost of construction and maintenance of roadways on permafrost.

The essential difference between the two types of pavements, flexible and rigid, is well known and their design concepts are well developed (Ref. 2,3,4). This guide is primarily concerned with the design of roadway sections consisting of base ànd subbase layers, with fill materials below the surface course of pavement. The "roadway" term is used to refer to the materials placed below the road surface (see Fig. 1-1). This manual will only address the design of this roadway section under a flexible pavement.

Roadway terminology to be used is detailed in the Glossary of Terms from the 1972 AASHO Interim Guide for Design of Pavement Structures (pg. 1). The objective of this manual is to produce a design guide related to roadway design and construction in permafrost areas. The manual covers the latest design and construction principles so that designers, researchers, planners, builders, operators or inspectors can systematically analyze the available information, assemble required data and design the roadways. General aspects of 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) Available design
(iii) Evaluations of various design methods and their relative usefulness
(iv) Construction problems and control
(v) Construction costs
(vi) Recommendations.

Particular emphasis has been placed on designing the supporting fill thickness against the detrimental effects of frost action and to preserve the thermal equalibrium of the permafrost. New construction techniques which are not presently practiced are also outlined. It is emphasized that further field studies will be required to use these new concepts.

1.2 Summary

Development of the permafrost regions will undoubtedly increase in the next decade and the expansion of present roadway facilities in Alaska in this region will play an important role to meet the future development.

Two extreme approaches are possible in the design of roadways on permafrost. Make the fill section thick enough to prevent thaw penetration and frost penetration into the permafrost; or to allow thaw penetration and frost penetration into the subgrade. Whether one of these, or an intermediate approach is taken, the roadway must be adequately stable against cracking, differential settlement and frost heaving. Frost susceptibility of construction materials and subgrade soils are to be evaluated in conjunction with the thaw weakening characteristics. Factors affecting mechanical and thermal stability, such as climatic factors, soil type and composition, precipitation, energy exchange components and others must be considered. This basic data is described in Section 2.

The design should consider the economics of alternate construction techniques, use of existing native soils, nature of foundation soils and thermal regime. Construction methods may be geared toward the minimal thermal degradation of existing permafrost conditions or toward removal or thawing of permafrost. Suitability of the use of native soils and other construction methods, such as insulated sections, peat overlays etc., must be considered to reduce the cost of construction. The available design methods are described in Section 3.

In addition to the minimization of thermal degradation of permafrost during the construction periods, factors such as topography, subsurface conditions, drainage, economics, scheduling and environmental impact, must be considered. The design and construction of any roadway is based on the optimum utilization of available, suitable construction materials and the cost of alternate sources. Section 4 includes the various aspects of constructions of roadways on permafrost.

Construction costs including engineering design and survey, testing of materials, construction quality control, and insulation products are described in Section 5.

Recommendations regarding future needs are given in Section 6. These include the determination of permafrost characteristics, its distribution, ice content and temperature profile; measurement of temperature, differential settlement and heave on selected existing roadway sections; method of improved construction techniques and stability of embankments, such as use of berm, insulation, and soil reinforcements.

TABLE 1-1 MODES OF DISTRESS IN PAVEMENTS*

DISTRESS MODE	GENERAL CAUSE	SPECIFIC CAUSATIVE FACTOR
Cracking	Traffic load	-Repeated loading (fatique) -Slippage (resulting from braking stresses)
	Other	-Thermal changes -Moisture changes -Shrinkage of underlying materials
Distortion (may also lead to cracking)	Traffic load	-Rutting or pumping and faulting (from repetitive loading) -Plastic flow or creep
	Other	-Differential heave Frost action in subgrades or bases Swelling of clays in subgrade -Differential settlement Permanent from long-term Transient from reconsolidation after heave (may be accelerated by traffic) -Curling of rigid slabs from moisture and temperature differential
Disintegration	May be advanced stage of cracking mode of distress or may result from differential effect of certain materials contained in the layered system or from abrasion by traffic. May also be triggered by freeze-thaw effects.	

(* After Ref. (1))

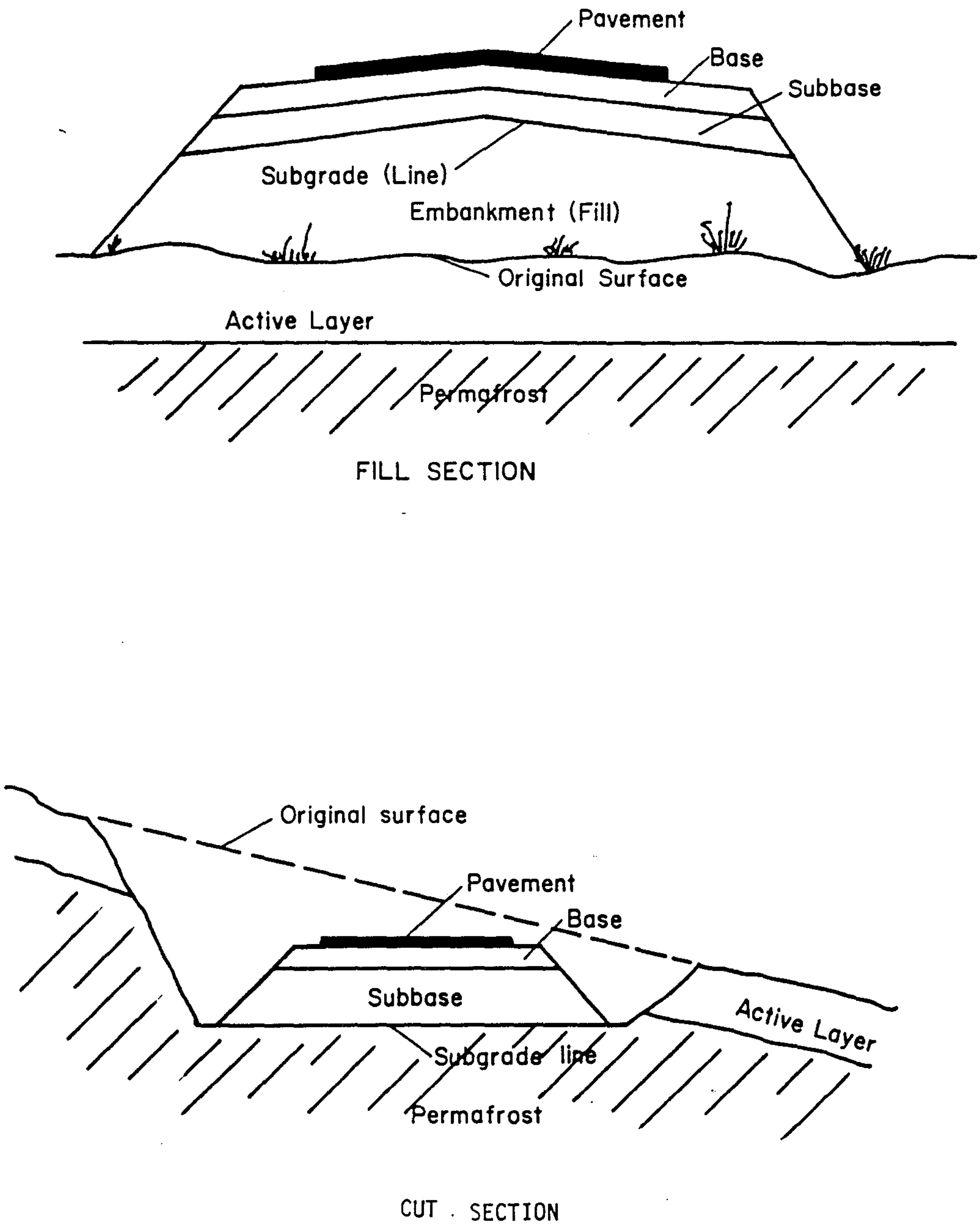


FIG. 1-1 ROADWAY SECTIONS

2. EXPLORATION OF DATA

2.1 General

The general influences of environmental and physical factors on the design and construction of roadways on permafrost are described in this section. The fundamental aspects of permafrost, climatic factors, material characterization and exploration approaches which are essential to determine the roadway thickness are given.

2.2 Permafrost Distribution

Since 1963, there has been considerable advance in knowledge regarding the permafrost distribution and its relation to the environment. An up-to-date detailed permafrost map of Alaska (scale 1:2,500,000) showing the distribution, thickness and ground temperature was published by the U.S. Geological Survey (5). During the past years, many ground and air surveys of permafrost distribution have been carried out in Alaska and Canada so that the general characterictics of these zones are well known (see Figs. 2-1 and 2-2).

On the basis of various studies (6), it became well known that in the continuous permafrost zone, permafrost occurs everywhere beneath the ground surface, except in unconsolidated sediments where the climate has just begun to impose its influence on the ground thermal regime. The thickness of permafrost is about 60-90 m at the southern limit of the continuous zone, possibly increasing steadily to about 1000 m in the northern part of the zone in Alaska and Canada. The temperature of the permafrost in this zone at the level of zero amplitude ranges from -5°C in the south to about -15°C in the extreme north. In the discontinuous zone, frozen and unfrozen layers occur together and the thickness of permafrost varies from a few centimeters or meters at the southern limit to about 60-90 m at the boundary of the continuous zone. The temperature of permafrost in the discontinuous zone at the level of zero annual amplitude generally ranges from a few tenths of a degree below 0°C at the southern limit to -5°C at the boundary of the continuous zone.

2.3 Climatic Factors and Ground Temperature Profile

Climate (air temperature), vegetation and drainage conditions are basic factors in the formation and existence of permafrost. Other climatic parameters such as wind, precipitation and the energy exchange components (radiation, evaporation, surface conductive capacity) play an important role in the thermal condition of permafrost. Detailed climatic information for Alaska is published by AIDC (7). Figs. 2-3 to 2-4 present some of the useful climatic data of Alaska.

The ground temperature profile is affected by climatic factors and ground surface characteristics. Typical ground temperature profiles in the continuous and discontinuous regions of Alaska are shown in Fig. 2-5.

It is to be noted that the climatic factors and precipitation play an important role in the design of the roadway and such basic data is a necessary background for design analysis.

2.4 Frost Action

Frost action is one of the major factors affecting the roadway performance. The term frost action includes both frost heave and loss of subgrade support during the thawing period. The problem of frost action damage is widespread. Two destructive effects pertinent to frost action of soils within or beneath a roadway section are the expansion and lifting of the soil (frost heaving) in winter and the loss of soil-bearing strength by thaw weakening in the spring. Soils that show one or both of these characteristics are referred to as frost susceptible.

2.4.1 Frost Heaving Process

Three conditions that must exist simultaneously for frost heaving to occur in soils are well known. These are a soil-moisture supply, sufficiently cold temperatures to cause soil freezing and a frost susceptible soil. The heaving of frost-suseptible soil can be attributed to two processes, namely freezing of in-situ pore water during frost penetration, and ice lens formation, or ice-segregation which occurs in soils due to migration of soil-moisture toward the freezing frost. Thus, heaving can be attributed mainly due to the growth of ice lenses in frost susceptible soils as the first process contributes only a nine percent volume change in the soil water volume unless pore water expulsion occurs. These ice lenses usually form parallel to the isothermal freezing plane and heaving is always in the direction of heat flow. Heave may be uniform or nonuniform, depending on variations in the characteristics of the soils and the ground water conditions.

Frost heave in roads is indicated by the raising of the pavement. When nonuniform heave occurs in pavement, there are appreciable differences in the heave of adjacent areas, resulting in objectionable unevenness or abrupt changes in grade at the pavement surface. Conditions conductive to irregular heave generally occur in areas where subgrades vary between clean nonfrost susceptible soils and silty frost susceptible soils or in places where abrupt transitions occur. Nonuniform heaves also occur in places where abrupt transitions from cut to fill sections are made with the ground water table close to the surface and where excavation cuts are made into water bearing strata.

Uniform heaving also occurs when soil and moisture conditions conducive to ice segregation exist under a pavement, but are uniform in longitudinal and traverse directions. Uniform heave which raises the pavement uniformly is not noticeable to motorists even though the vertical displacement may amount to several inches, and has no effects on serviceability of the pavement as long as the frozen and heaved condition lasts. Undesirable effect of heaves, whether uniform or nonuniform, may become noticeable during the spring when the thaw weakening and release of water into the base course may increase the rate of deterioration and thus affect the pavement's performance.

2.4.2 Thaw Weakening

Soil weakening after thawing is particularly observed early in the spring when thawing is occurring at the top of the subgrade and the rate of melting is rapid in comparison to the rate of drainage. As shown in Figure 2-6, melting of the ice from the surface downward releases water which cannot drain through the still frozen soil below, or redistribute itself readily. As a result, the base course becomes completely saturated resulting in the reduction of bearing capacity of the base. Attention has been drawn to the detrimental effect of high traffic density and load during the thawing period. Such conditions may cause excessive pore pressures and greatly reduce the load carrying capacity of pavement. Ice segregation during freezing may not always be a necessary precursor for the thaw weakening of soils. The supporting capacity in clayey soil subgrade may be reduced even though significant ice-lenses or heave have not occurred, because freeze-thaw shrinkage and remolding processes may reduce the subgrade strength. The reduction of soil strength during the frost melting periods and the length of time during which the strength of soils is reduced depend on the type of soil; the temperature conditions during the freezing and thawing periods; the amount and type of traffic during frost melting periods; the moisture supply during fall, winter and spring; and the drainage conditions.

2.5 General Soil and Frost Susceptibility Classification

Both the Unified Soil Classification System (USCS) and the AASHO classification system for unfrozen soils are given in Tables 2-1 and 2-2. For design and construction purposes, these soil classification systems are necessary. Generally, the USCS is extended for frozen soils, and the frozen soil classification is useful for the design of roadways in permafrost regions. The frozen soil is described and classified according to three steps (8): (1) the soil phase is identified independent of the frozen state using the USCS given in Table 2-1, (2) soil characteristics resulting from freezing of the material (Table 2-3) are added to the description, and (3) ice found in the frozen materials is described (Table 2-3).

The U.S. Army Corps of Engineers (9) has developed criteria to delineate the degree of frost susceptibility of various soils and this criteria is most commonly used in North America as a basis for the frost design (Table 2-4). In this system frost susceptible soils (with 3 percent or more, by weight, finer than 0.02 mm) are classified into one of the four groups F1, F2, F3, and F4. Soil types are listed in Table 2-4 in approximate order of increasing susceptibility to frost heaving and/or weakening as a result of frost melting, although the order of subgroups under F3 and F4 do not necessarily indicate the order of susceptibility to frost heaving of these subgroups. The basis for distinction between the F1 and F2 groups is that F1 material may be expected to show higher bearing capacity then F2 material during thaw, even though both may have experienced equal ice segregation.

2.6 Characterization Materials

Various standard tests are performed to determine the properties and characteristics of roadway materials. These tests are categorized into the main following groups for unfrozen and frozen soils.

Unfrozen Soils

1. Routine tests for strength and deformation:	a) California Bearing Ratio (CBR) b) Stabilometer (R-Value) c) Triaxial d) Pile Loading e) Indirect tensile test
2. Sophisticated tests for theoretical solutions:	a) Resilient Modulus b) Dynamic Modulus
3. Long term tests to determine material distress condition:	a) Fatigue b) Heave Rate

Frozen Soils

1. Routine tests for strength and deformation:	a) Unconfined Compression b) Triaxial c) Indirect Tensile d) Frost-Heave e) Freeze-Thaw Cycles
2. Sophisticated tests for theoretical and long term conditions:	a) Creep Test b) Resilient Modulus c) Dynamic Modulus d) Thaw-Consolidation

Common standard tests which are used to determine properties of soils are:

(i) Grain size distribution by sieve and hydrometer tests
(ii) Specific gravity
(iii) Water content
(iv) Optimum moisture content
(v) Shear strength
(vi) Permeability
(v) Consolidation (fine-grained subgrade soils only)

Standard procedures (AASHTO) are available to perform these tests and these should be followed to obtain different data which will be useful in the design and construction of roadways.

Characteristics of soils pertinent to roadway foundation are presented in Table 2-5.

2.7 Stress Distribution

The determination of stress distribution due to various traffic loading conditions is essential for the design of roadway sections. A widely accepted standard vehicle type is the 18 kip single axle (generally equivalent to 32 kip dual tandem). Thus, the effects of other vehicles are normally accounted for in the design by the equivalent 18,000 pound single axle load (EAL).

Theoretical solutions (one layer, two layer and three layer system) are available to determine the stress distribution at depths. Boussinesq's formula is commonly used to obtain the vehicle stress (σ_z) under a point load as follows:

$$\sigma_Z = \frac{P}{Z^2}\ \frac{3}{2\pi}\left[\frac{1}{\left\{1 + (\gamma/z)^2\right\}^{5/2}}\right] \qquad (2.1)$$

where Z = depth

γ = radial distance from point load (P)

The Boussinesq's Formula may also be used to calculate vertical stress under the distributed load as shown below:

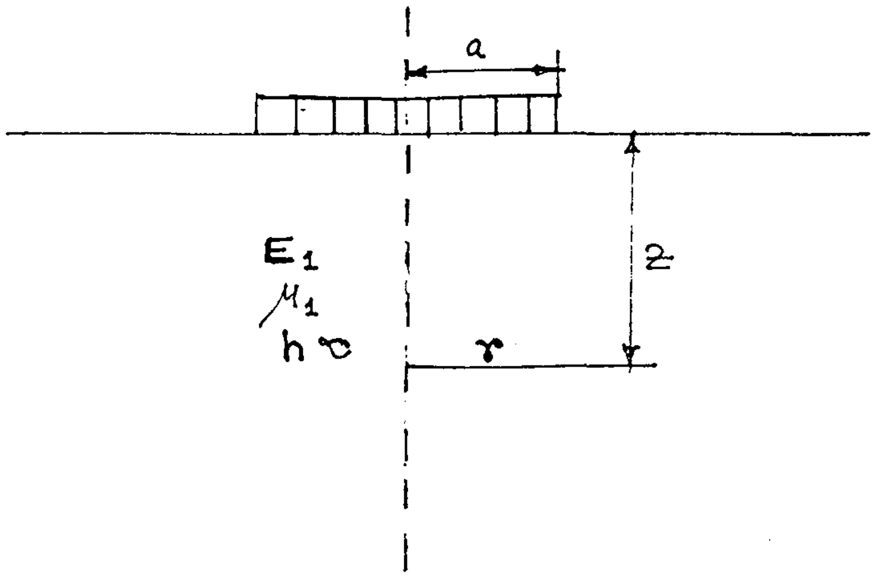


Fig. 2-7 One-Layer System

Other situations such as two layer problem and three layer system given by Burmister (10, 11) and Acum et al. (12) respectively, are often used to calculate stress and displacements in layered soil profile. Burmister's two layered solution under the center of a circular plate is given in Fig. 2-8.

Fig. 2-9 illustrates the influence of changing the pavement layer thicknesses upon the vertical compressive stress factor for a three layer payement system. It can be seen from the figure that the subgrade stress is greatly decreased by a decrease in the ratio of a/h_2 parameter. In the three layered system, the base course modulus (E_2) has the pronounced effect upon stress reduction while the pavement layer modulus (E_1) controls the subgrade stress for two layered systems.

2.8 Site and Route Exploration

Site and route exploration are required for the final design and construction of roadways. Reconnaissance survey, detailed site and route studies, environmental considerations and the availability of construction materials must be taken into account. Lack of information on permafrost conditions and the terrain factors lead to the unsatisfactory performance of roadways on permafrost and maintenance costs become excessive. The scope of site and route exploration depends on the amount of available information, its geographical location and the type of roadway.

Site and route information required are presented in Table 2-6.

Table 2-6 Site and Route Information

A. Terrain Factors

1) Relief Terrain	i) Topography ii) Slope including degree and orientation iii) Surficial information (vegetation & featur
2) Geology and Soils	i) Surface and subsurface geology ii) Temperature profile iii) Ground ice conditions iv) Engineering properties of soils and rocks v) Borrow areas
3) Hydrology	i) Surface and subsurface drainage ii) Ground water iii) Flooding iv) Icing v) Ice jams vi) Snow melt

B. Climate

a) Temperature
b) Precipitation (rain & snow)
c) Wind

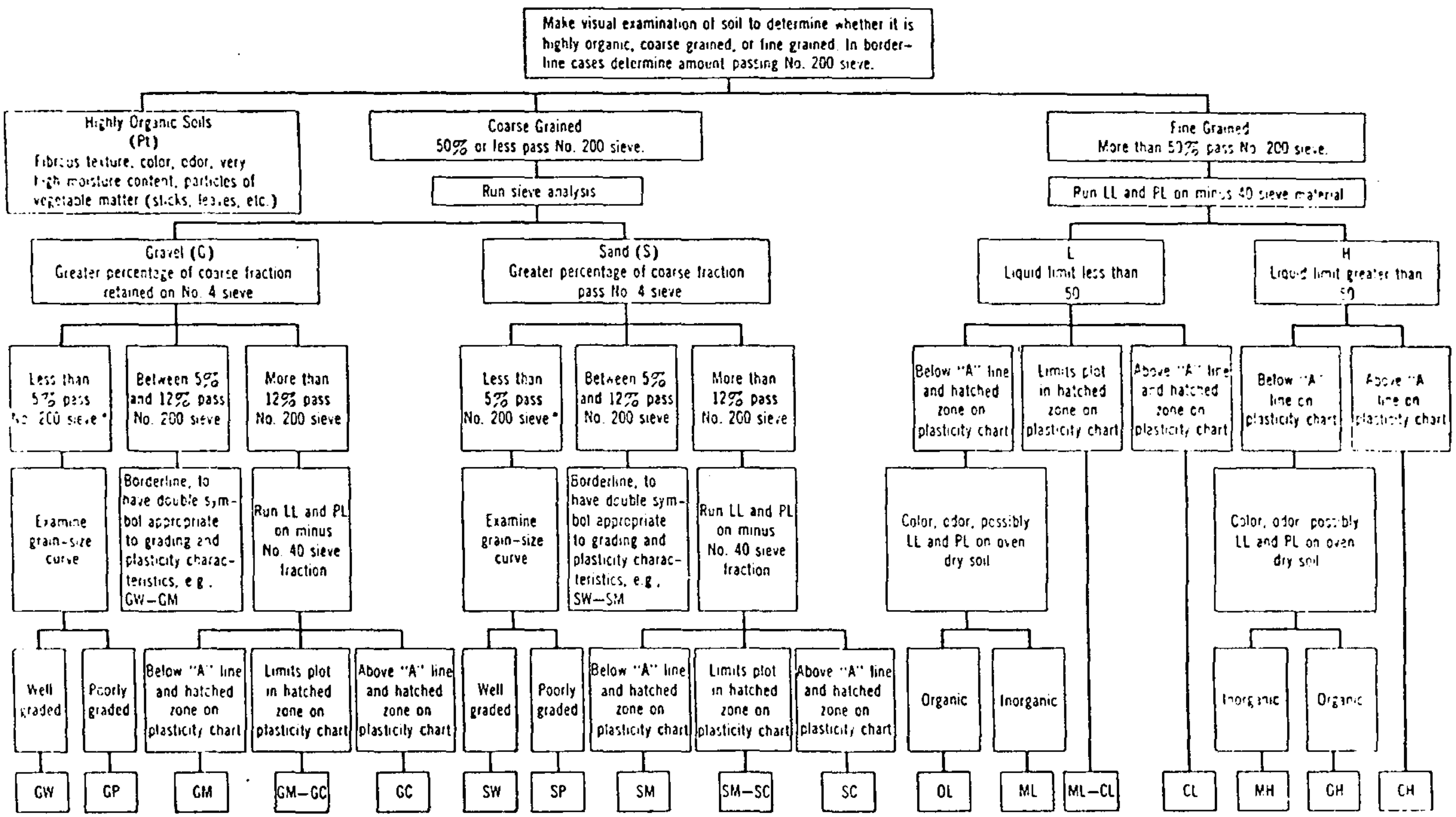
C. Environmental

i) Water pollution
ii) Land abuse (specifically wet land)
iii) Wild life (animal, fish, birds)

D. Logistics

i) Winter and summer construction
ii) Access to and from
iii) Local materials and equipment
iv) Transportation facilities

TABLE 2-1



Note: Sieve sizes are U.S. Standard.

* If fines interfere with free-draining properties use double symbol such as GW-GM, etc.

Chart for auxiliary laboratory identification procedure. (From Corps of Engineers.)

TABLE 2-2

TABLE AASHO Soil Classification

(Classification of Highway Subgrade Materials)

General classification	Granular materials (35% or less passing No. 200)			Silt-clay materials (More than 35% passing No. 200)			
Group classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, per cent passing							
No. 10							
No. 40	50 max	51 min					
No. 200	26 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40:							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	NP		10 max	10 max	11 min	11 min
Group index			4 max	8 max	12 max	16 max	20 max
General rating as subgrade	Excellent to good			Fair to poor			

(Subgroups)

General classification	Granular materials (35% or less passing No. 200)							Silt-clay materials (more than 35% passing No. 200)			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
Group classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, per cent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40:											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Group index	0		0	0		4 max		8 max	12 max	16 max	20 max
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

TABLE 2-3

Table Description and classification of frozen soils (adapted from Linnell and Kaplar, 1966)

I: Description of soil phase (independent of frozen state)	Classify soil phase by the unified soil classification system					
	Major group		Subgroup			
	Description	Designation	Description		Designation	
II: Description of frozen soil	Segregated ice not visible by eye	N	Poorly bonded or friable		Nf	
			Well bonded	No excess ice	Nb	n
				Excess ice		e
	Segregated ice visible by eye (ice 25 mm or less thick)	V	Individual ice crystals or inclusions		Vx	
			Ice coatings on particles		Vc	
			Random or irregularly oriented ice formations		Vr	
			Stratified or distinctly oriented ice formations		Vs	
III: Description of substantial ice strata	Ice greater than 25 mm thick	ICE	Ice with soil inclusions		ICE + soil type	
			Ice without soil inclusions		ICE	

TABLE 2-4 FROST DESIGN SOIL CLASSIFICATION

Frost group	Kind of soil	Typical soil types: Percentage finer than 0.02 mm by weight	Typical soil types: Under Unified Soil Classification System
NFS	Gravelly soils, sands	0-3	GW,GP,SW,SP
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	Gravelly soils	Over 20	GM,GC
	Sands, except very fine silty sands	Over 15	SM, SC
	Clays, PI>12	-	CL, CH
F4	All silts	-	ML, MH
	Very fine silty sands	Over 15	SM
	Clays, PI<12	-	CL, CL-ML
	Varved clays and other fine-grained, banded sediments	-	CL and ML; CL, ML and SM; CL, CH and ML; CL, CH, ML and SM

TABLE 2-5

Characteristics Pertinent

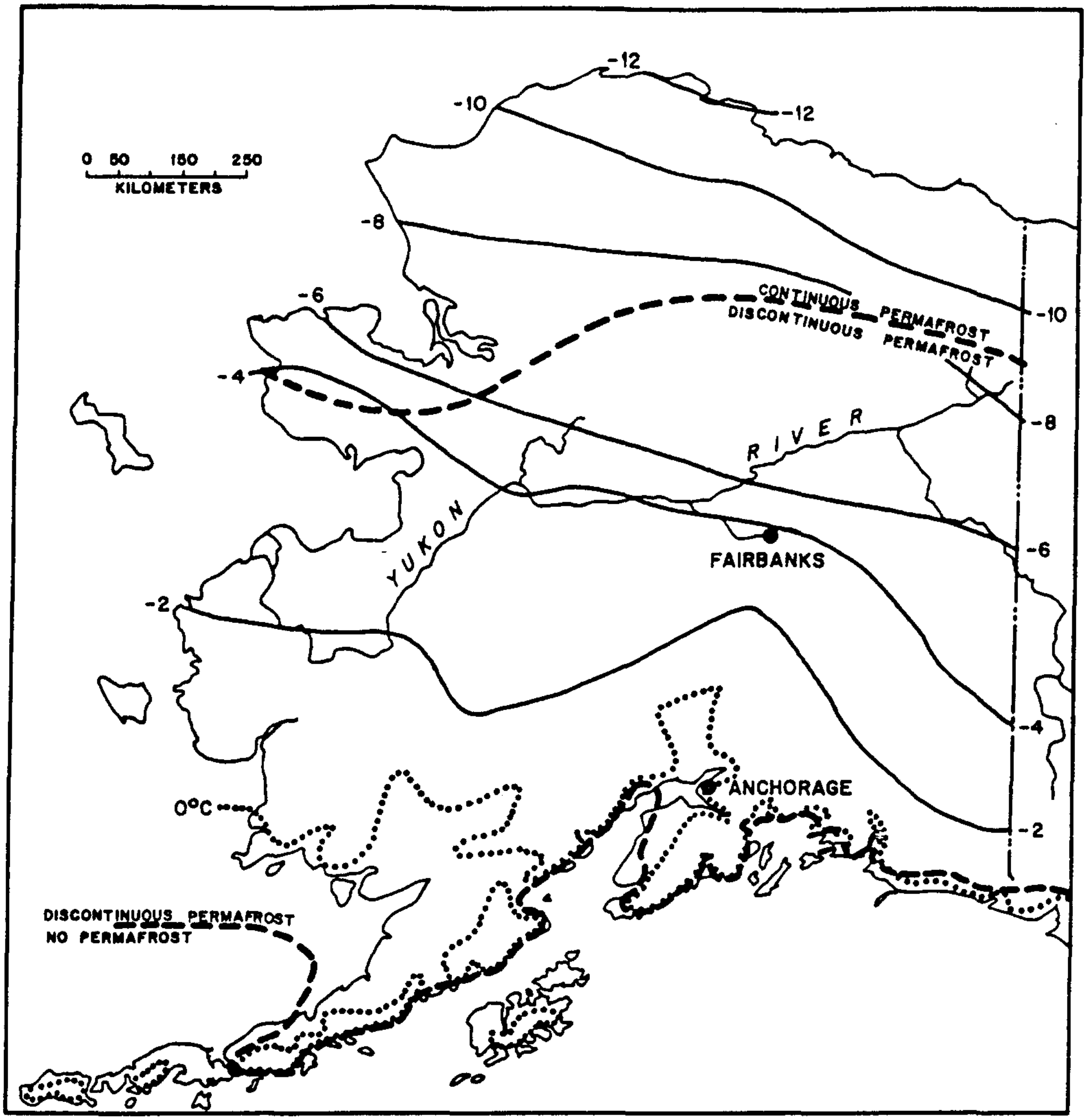
Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Frost Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Frost Action (7)
Coarse-grained soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight
		GP	Gravel or sandy gravel, poorly graded	Good to excellent	Poor to fair	None to very slight
		GU	Gravel or sandy gravel, uniformly graded	Good	Poor	None to very slight
		GM	Silty gravel or silty sandy gravel	Good to excellent	Fair to good	Slight to medium
		GC	Clayey gravel or clayey sandy gravel	Good	Poor	Slight to medium
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor	None to very slight
		SP	Sand or gravelly sand, poorly graded	Fair to good	Poor to not suitable	None to very slight
		SU	Sand or gravelly sand, uniformly graded	Fair to good	Not suitable	None to very slight
		SM	Silty sand or silty gravelly sand	Good	Poor	Slight to high
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high
Fine-grained soils	Low compressibility LL < 50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to poor	Not suitable	Medium to very high
		CL	Lean clays, sandy clays, or gravelly clays	Fair to poor	Not suitable	Medium to high
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to high
	High compressibility LL > 50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high
		CH	Fat clays	Poor to very poor	Not suitable	Medium
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium
Peat and other fibrous organic soils		Pt	Peat, humus, and other	Not suitable	Not suitable	Slight

[1] From Corps of Engineers

TABLE 2-5 (cont'd)

to Road and Runway Foundation*

Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Dry Weight (pcf) (11)	Field CBR (12)	Subgrade Modulus *k* (pci) (13)
Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125–140	60–80	300 or more
Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	120–130	35–60	300 or more
Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	115–125	25–50	300 or more
Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	130–145	40–80	300 or more
Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120–140	20–40	200–300
Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110–130	20–40	200–300
Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	105–120	15–25	200–300
Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100–115	10–20	200–300
Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	120–135	20–40	200–300
Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105–130	10–20	200–300
Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	100–125	5–15	100–200
Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100–125	5–15	100–200
Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90–105	4–8	100–200
High	Fair to poor	Rubber-tired equipment, sheepsfoot roller	80–100	4–8	100–200
High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	90–110	3–5	50–100
High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	80–105	3–5	50–100
	Fair to poor	Compaction not practical			



LEGEND

PERMAFROST ZONE BOUNDARY

APPROXIMATE POSITION OF MEAN ANNUAL AIR TEMPERATURE ISOTHERM, 0°C

MEAN ANNUAL AIR TEMPERATURE, °C

FIG. 2-1 PERMAFROST DISTRIBUTION, ALASKA (After Brown & Pewe, 1973)

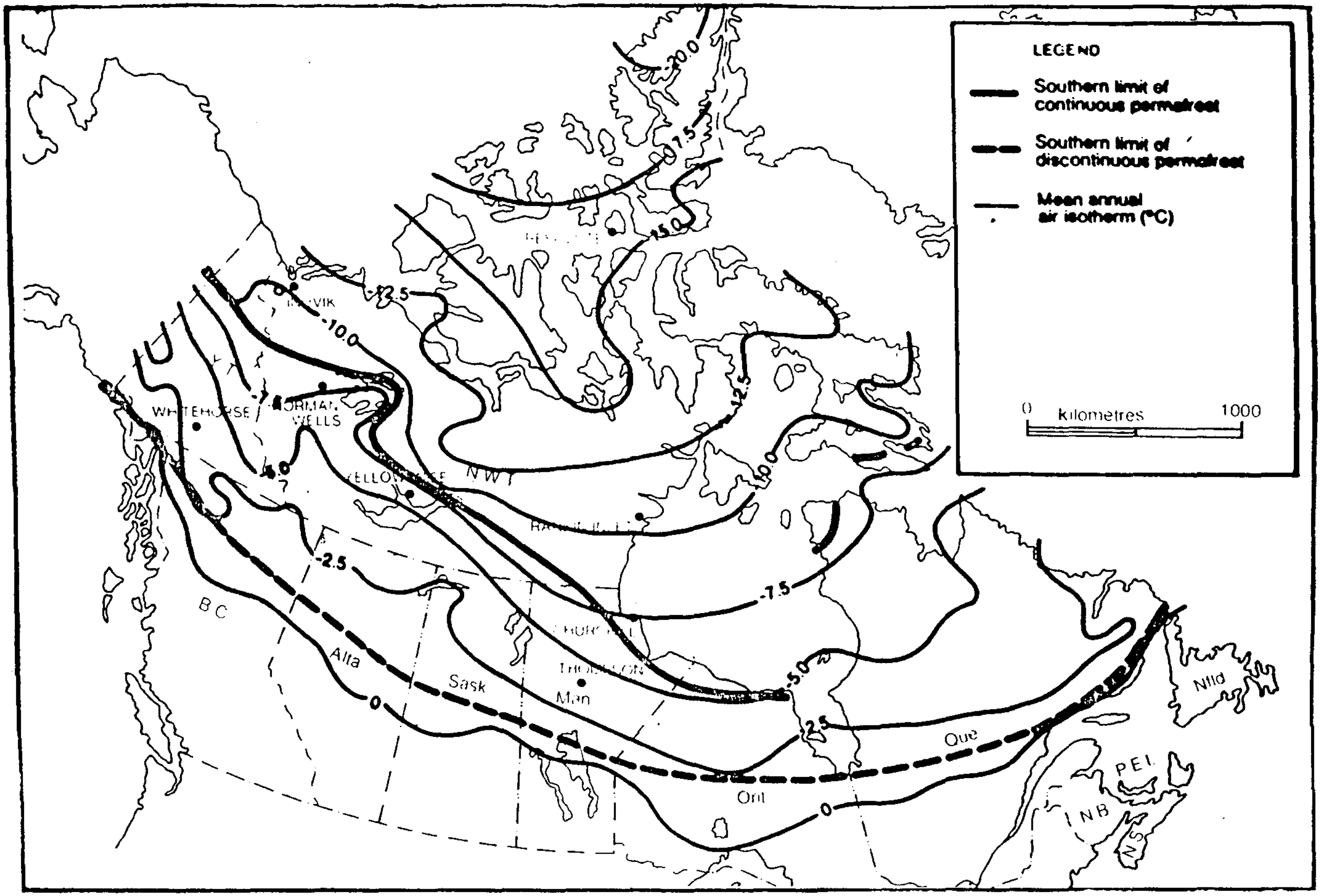


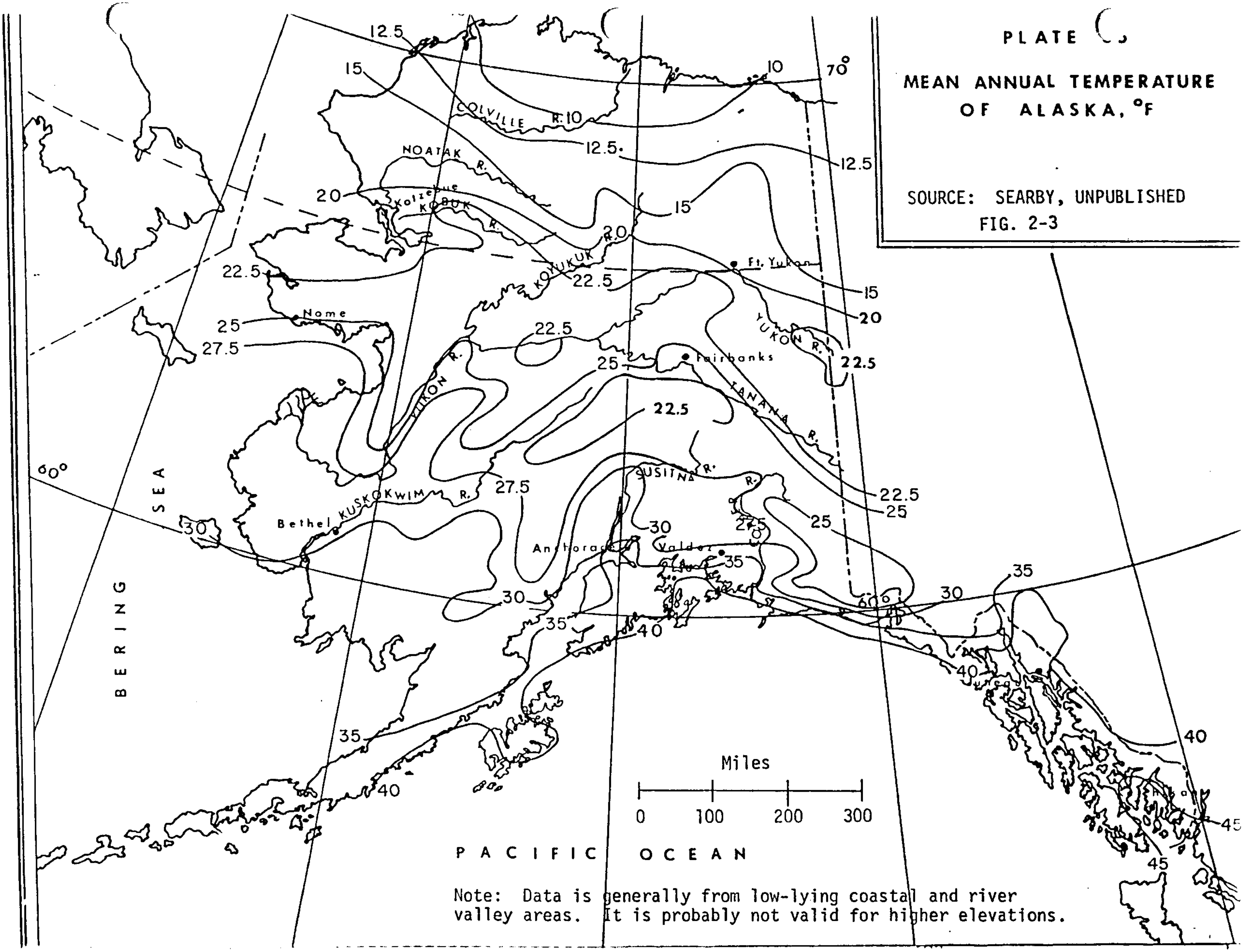
FIGURE 2-2 DISTRIBUTION OF PERMAFROST AND GROUND TEMPERATURE OBSERVATION SITES IN CANADA

PLATE

MEAN ANNUAL TEMPERATURE OF ALASKA, °F

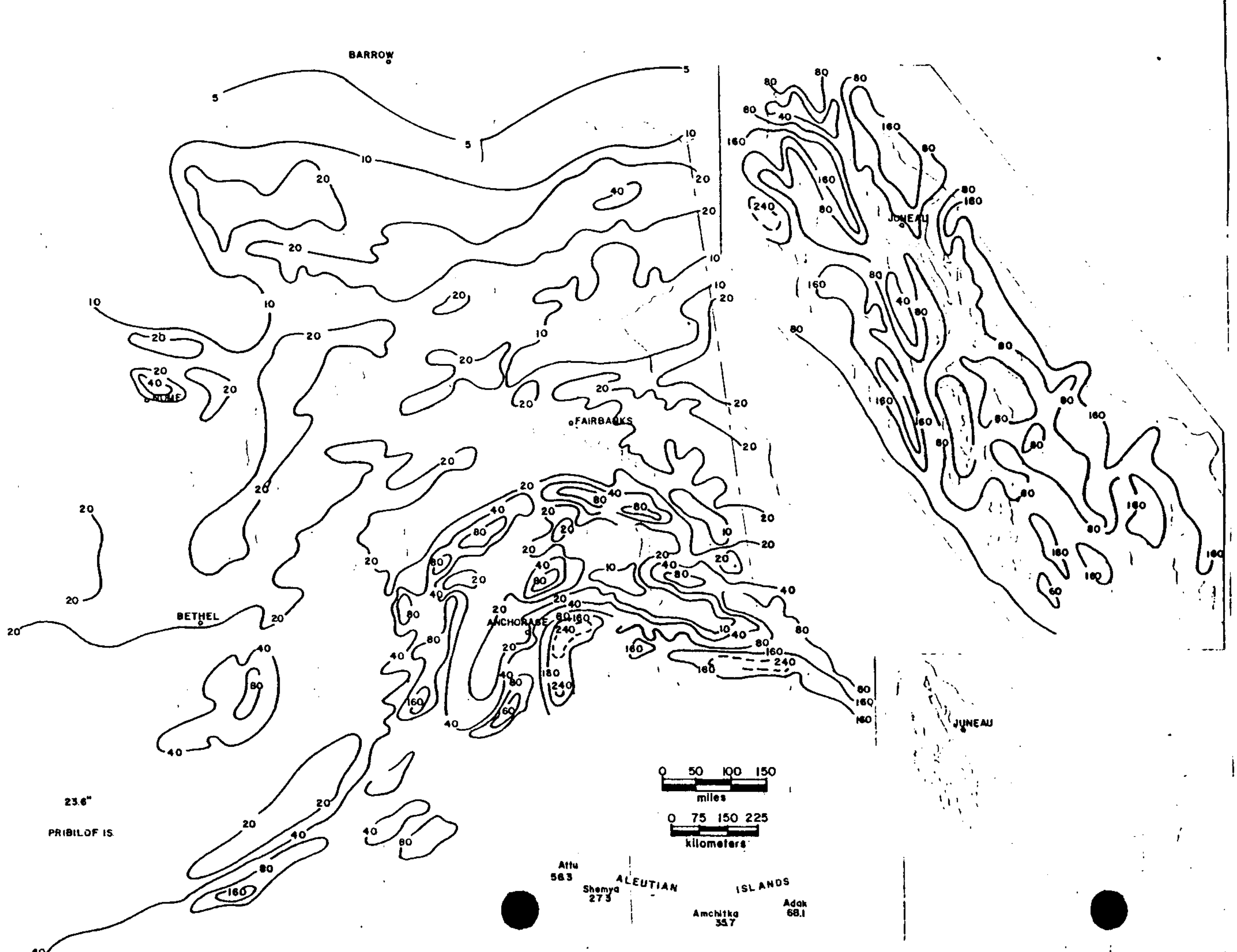
SOURCE: SEARBY, UNPUBLISHED

FIG. 2-3



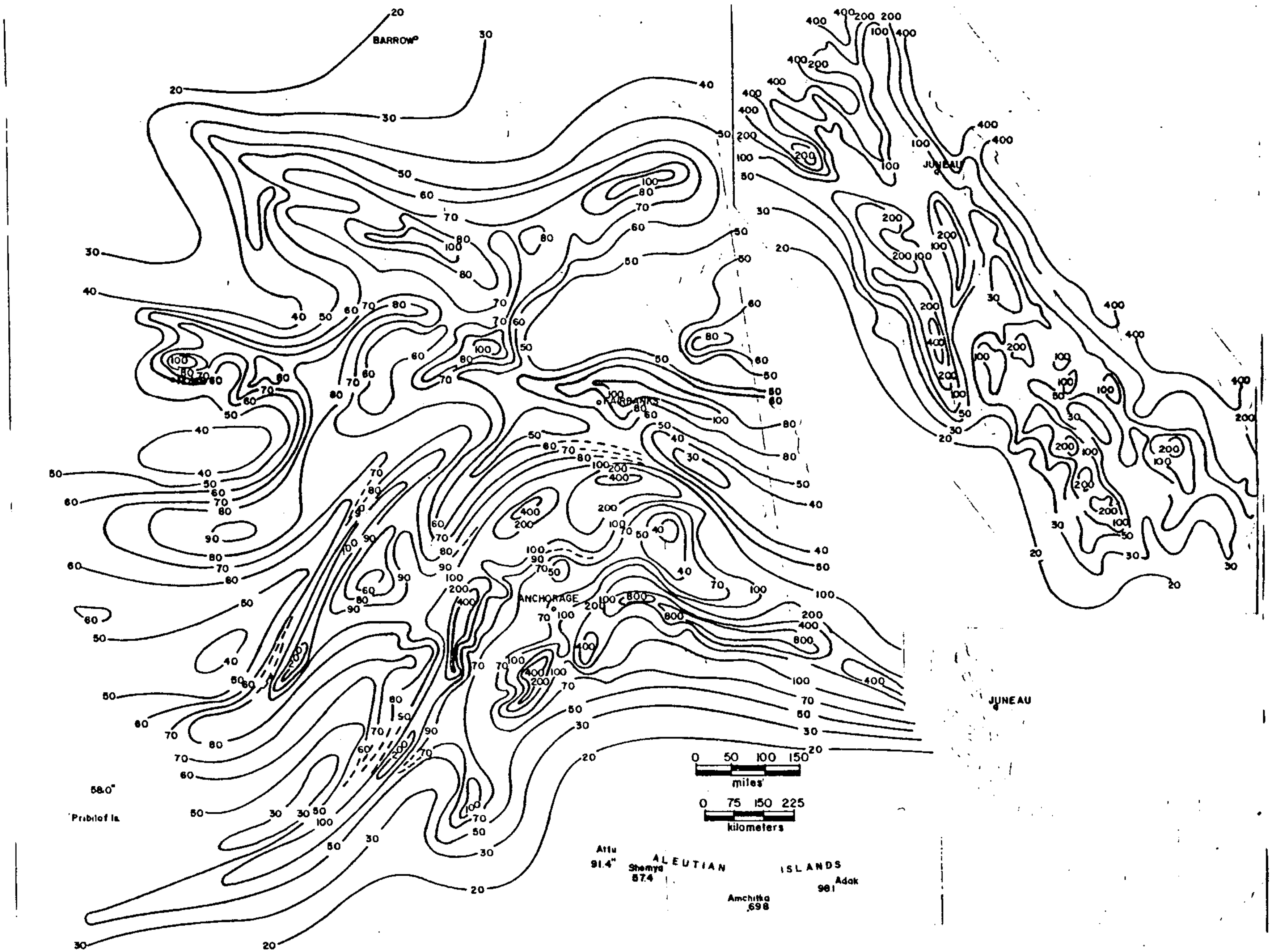
Note: Data is generally from low-lying coastal and river valley areas. It is probably not valid for higher elevations.

Precipitation values include the water equivalent of the snow.



Adapted from National Weather Service and U. S. Geological Survey

FIG. 2-4(a) MEAN PRECIPITATION (SNOW), ALASKA (After Ref. 7)



Adapted from National Weather Service and U. S. Geological Survey

FIG. 2-4(b) MEAN PRECIPITATION (RAINFALL), ALASKA (After Ref. 7)

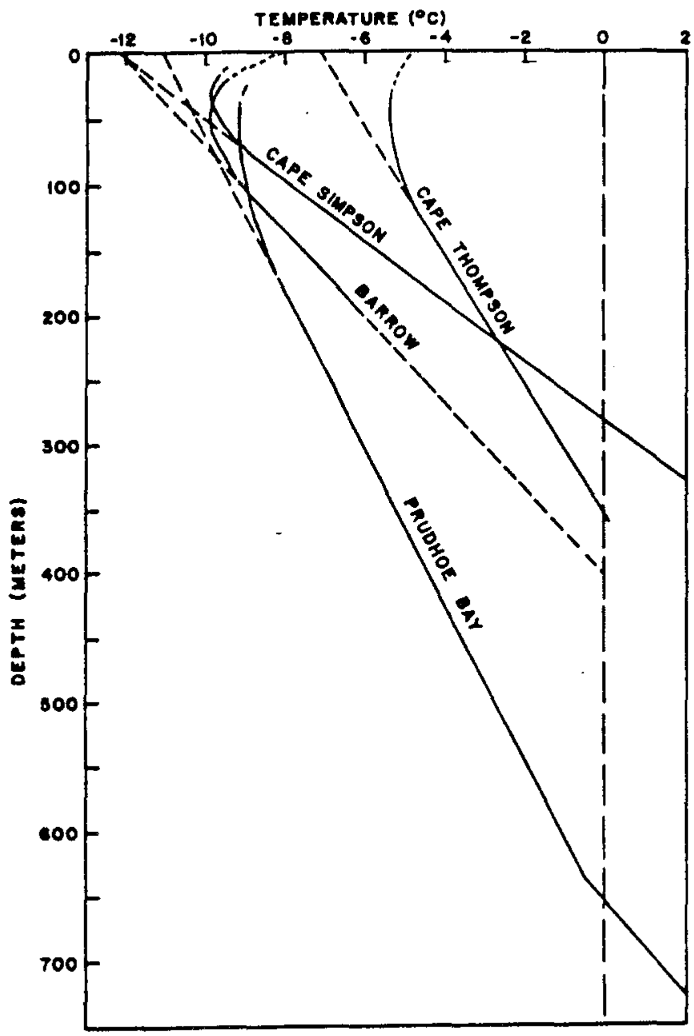


FIG. 2-5(a) GROUND TEMPERATURE PROFILE (CONTINUOUS PERMAFROST)

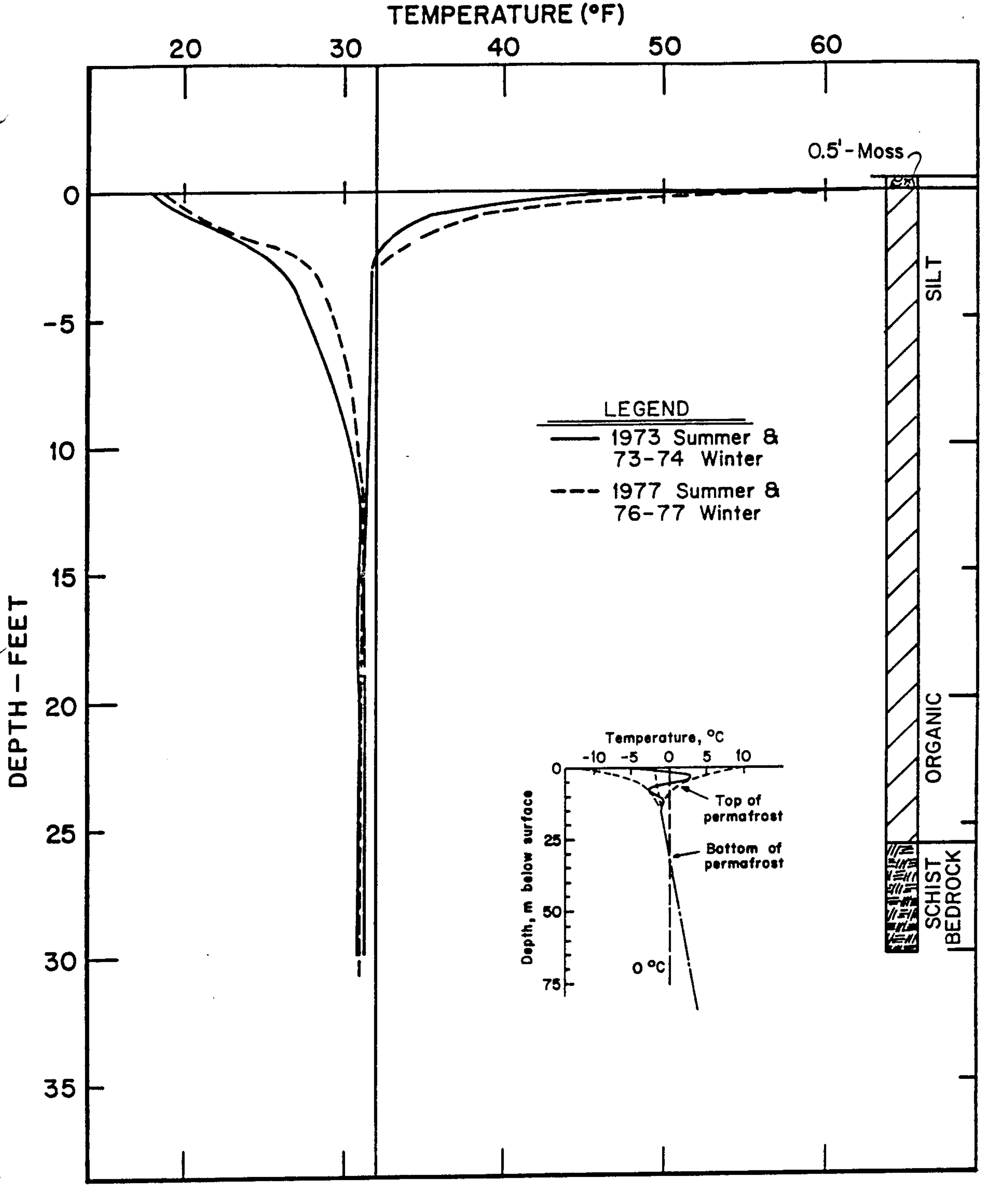


FIG. 2-5(b) GROUND TEMPERATURE PROFILE (DISCONTINUOUS PERMAFROST)

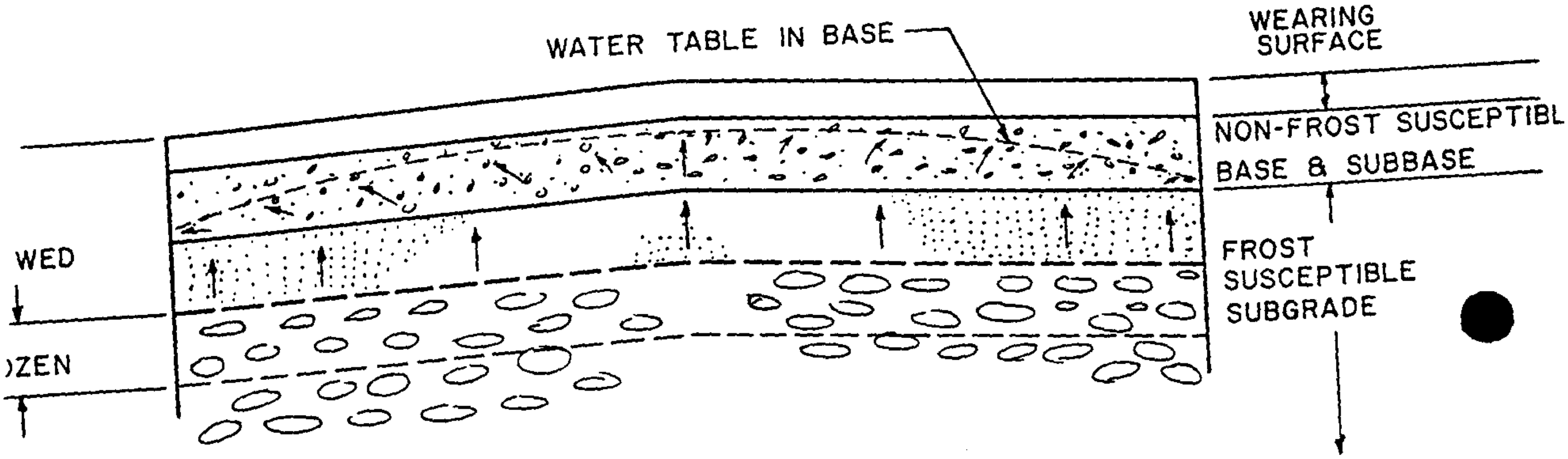


FIG. 2-6 THAW WEAKENING CONDITION

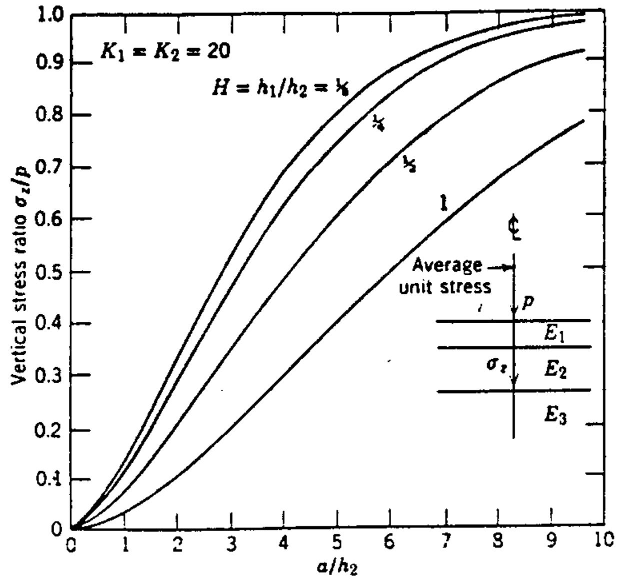


FIG. 2-9 VERTICAL STRESS DISTRIBUTION, THREE LAYERED SOLUTION (After Ref.

3. DESIGN METHODS

3.1 General

The flexural design and performance aspects of pavement design for permafrost regions has not been found to differ significantly from seasonal frost areas. In fact, the structurally acting zone of the roadway embankment which must support the imposed vehicle loadings and prevent fatigue cracking of the pavement surface, essentially always exists in the seasonal frost or active layer region of the roadway embankment. For this reason, flexural design methods for seasonal frost can be directly applied to permafrost regions. Further discussion of pavement flexural design methods will not be attempted herein. The current Alaska Department of Transportation pavement design method for roadway pavement may be referred (17).

3.2 Design Methods for Cold Regions

Roadway design in seasonal frost as well as in permafrost areas, depends on the type of subgrade soil as well as "fill" materials used for the roadway section. The main concerns are the limitation of the surface deformation resulting from seasonal thaw related settlements, the prevention of long term thawing of the roadbed soils and the adequacy of bearing capacity during the seasonal thawing period. The first factor is more severe in the seasonal frost areas, whereas the second factor is more predominant in the permafrost regions. Additional considerations must be given to the influence of construction on the existing ground thermal balance. Changes in the ground thermal regime may cause degradation of the permafrost, resulting in total or differential settlement and reduction in the bearing capacity of the pavement structure.

3.2.1 Roadway Design Under Seasonal Frost Area

Design of roadway in seasonal frost is based on two alternative concepts:

i) Roadway surface deformation resulting from frost action or seasonal thaw-freeze cycles must be controlled.
ii) Bearing capacity of the roadway must be sufficient during the adverse climate period.

The first concept required that the combined thickness of pavement and non-frost susceptible base must contain the frost or thaw penetration. This approach can be labeled "full protection" method. The second concept anticipates subgrade frost penetration and the subsequent reduced strength of the subgrade during the melting period. Using these concepts, the Corps of Engineers has developed the following design methods:

(1) Complete or full protection
(2) Limited subgrade frost protection
(3) Reduced subgrade strength

3.2.1.1 Complete Protection

The complete protection method will limit seasonal thaw or frost protection to the NFS base and subbase courses. This protection prevents the underlying frost-susceptible soils from freezing in seasonal frost areas or the frozen soil from thawing in permafrost areas. The required thickness of the base course can be computed by either of the methods outlined in Sec. 3.2.2.1. The lower 100 mm of the base must be designed as a filter. Use of high-moisture-retaining non-frost-susceptible soils, such as uniform sands, in permafrost areas decreases the required base-course thickness. The higher latent heat of these soils gives them a higher resistance to thaw penetration. Use of insulation beneath the base course to prevent subgrade freezing or thawing is described in Section 4.6.1.

3.2.1.2 Limited Subgrade Frost Protection

The limited-subgrade-frost-penetration method attempts to confine deformations to small acceptable values by use of a combined thickness of pavement and non-frost-susceptible base and subbase courses which limits the subgrade frost penetration. Added thickness may be needed in some cases to keep pavement heave and cracking within tolerable limits based on local experience and field data. Linell, et al., (18) recommended this method in seasonal frost areas for the following soils: (1) all silts, (2) very fine silty sands containing more than 15 percent fine than 0.02 mm by weight, (3) clays with plasticity indexes of less than 12, and (4) varved clays and other fine-grained banded sediments. Exceptions include extremely variable subgrade conditions for which the complete-protection method must be used or when some nonuniform heave and cracking are not considered detrimental for flexible paved areas.

This design method uses an average air-freezing index for the three coldest years in a 30-year period (or for the coldest winter in 10 years of record) to determine the combined thickness of pavement and base required to limit subgrade frost penetration. Determine the base thickness required for zero frost penetration by subtraction. The ratio r equals the water content of the subgrade divided by the water content of the base. Enter Fig. 3-12 with the base thickness for zero frost penetration into the subgrade and, using the r value, read the design base thickness on the left-hand scale and the allowable frost penetration shown on the right-hand scale. The r value is limited to a maximum of 2.0 because part of the moisture in fine-grained soils remains unfrozen. The bottom 100 mm of base should consist of non-frost-susceptible sand, gravelly sand, or similar material designed as a filter. When the combined design thickness of pavement and base exceeds 1.8 m, Linell, et al., (18) recommended consideration of the following alternatives: (1) limit the total combined thickness to 1.8 m, (2) reduce the required combined thickness by use of a base of non-frost-susceptible uniform fine sand with high moisture retention in the drained condition in lieu of more free-draining material.

3.2.1.3 Reduced Subgrade Frost Penetration

The reduced-subgrade-strength method is based on the anticipated reduced subgrade strength during the frost melting period, and the amount of heave is neglected. Linell, et al., (18) and Hennion and Lobacz (19) recommended this method for both seasonal frost and permafrost areas. This method may be used for flexible pavements on soil groups F1, F2, and F3 when subgrade conditions are sufficiently uniform to assure that objectionable differential heaving or subsidence will not occur. In certain cases nonuniform subgrade conditions are correctable by removal and replacement of pockets of more highly frost-susceptible or high-ice-content soils. For permafrost areas an estimate, based on a study of the area and local experience, should be made of the magnitude and probable unevenness that will result from future subsidence as thaw occurs in the existing frozen soil. In some areas maintenance may be more economical than providing adequate initial fill. This approach is referred to as controlled subsidence. For paved areas, base-course thickness may be increased as needed over the reduced-strength-design requirements to provide a surcharge load sufficient to reduce differential heave resulting from seasonal freezing of a frost-susceptible subgrade.

The design curves shown in Fig. 3-13 are used to determine the combined thickness of flexible pavement and non-frost-susceptible base. The design index represents all traffic expected to use the pavement during its service life. It is based on typical magnitudes and composition of traffic reduced to equivalents in terms of repetition of an 80-kN single-axel dual-tire load (27). For the design index and soil group representative of the site, Fig. 3-13 gives the combined thickness of pavement and base required by the reduced-subgrade-strength method. The combined thickness of the pavement and base should in no case be less than 230 mm where frost action is a consideration. Again the lower 100 mm of the base should be graded to provide filter action against the subgrade.

3.2.1.4 Material Characterization

Table 3-10 presents the properties of base and subgrade materials significant to the performance of flexible pavements in seasonal frost areas. The key properties include shear strength, strain-modulus, Poisson's ratio and volume change parameters.

3.3 Roadway Design Under Permafrost Areas

In permafrost areas, roadway design must consider not only the effects of seasonal thawing and freezing cycles, but also the effect of construction on the existing thermal balance.

In the continuous permafrost regions with high ice-content frozen soils at shallow depth, the appropriate approach will be to restrict the seasonal thawing to the pavement and a non-frost susceptible base course and thereby keeping the permafrost "frozen" as in the original state. This method involving preventing degradation of the permafrost is comparable to the complete protection method used in seasonal frost areas; however, the citical factor here is depth of thaw penetration rather than depth of frost penetration.

The limited subgrade frost penetration method discussed in the previous section is impractical for most permafrost areas because of high freezing index values requiring thicknesses in excess of practical and economical limitations. Therefore, design, except in areas of continuous permafrost at shallow depth, is usually based on the reduced subgrade strength method and consideration of the effect of construction on the existing thermal regime. In areas of sporadic permafrost with horizontal variable, highly frost susceptible subgrade soil and variable moisture conditions, the limited subgrade frost penetration method may be useful.

3.3.1 Depth of Thaw Penetration or Freezing

The depth of thawing or freezing is best obtained by field measurement, but can be estimated using one of the many analytical solutions in the literature (21,22,23,24). Because of the assumptions necessary in these analytical solutions, such as a step change in surface temperature and/or neglecting the soil temperature changes, they may over estimate the maximum thawing or freezing isotherm depths for the given condition and are therefore, useful in engineering computations. They are generally based on Neuman or Stephan solutions which have the form:

$$X = \alpha \sqrt{t} \tag{3.3}$$

where X = depth of thawing or freezing

α = constant

t = thawing or freezing period

The following equations may be used to calculate the depth of thawing or freezing for specific conditions:

$$X = \sqrt{\frac{2k \cdot I_g}{L}} \tag{3.4}$$

$$X = \sqrt{\frac{2K \cdot I_g}{L + C\,T_m - T_o + \frac{I_g}{2t}}} \tag{3.5}$$

$$X = \sqrt{\left(\frac{k_2}{k_1} d_1\right)^2 + \frac{2k_1 \cdot I_g - \frac{d_1^2 \cdot L_1}{2k_1}}{L_2}} - \left(\frac{k_2}{k_1} - 1 \quad d_1\right) \tag{3.6}$$

$$X = \sqrt{\frac{2k \cdot I_g}{L}} \left(1 - \frac{C \cdot I_g}{8L \cdot t}\right) \tag{3.7}$$

$$X = \Lambda \sqrt{\frac{2k \cdot I_g}{L}} \tag{3.8}$$

where: I_g = ground surface thawing index (I_t) or freezing index (I_f)

k = thermal conductivity of the material above the freezing isotherm, k_f for frost penetration and k_t for thawing calculations

L = volumetric latent heat of the material undergoing phase change

C = volumetric heat capacity of the material above the freezing isotherm, C_f or C_t

T_m = mean annual site temperature

T_o = freezing point

d = thickness of layer of material

Λ = a correction coefficient which takes into consideration the effect of temperature change in the soil and primarily accounts for the volumetric specific heat effects. It is a function of two parameters; the thermal ratio (a) and the fusion parameter (μ).

$$a = \frac{T_m - T_o}{T_s} = \frac{T_m - T_o \cdot t}{I_g}$$

$$\mu = \frac{C \cdot I_g}{L \cdot t}$$

$T_s = I_g/t$, surface freezing or thawing index divided by the length of that period.

Subscripts f and t refer to freezing and thawing, and subscript 1 and 2 refer to the surface layer and the underlying material.

Equation (3-4) is the Stephan solution for a homogenous material with a step change in surface temperature. This is modified in equation (3-5) to account for the temperature change in the freezing or thawing soil. Equation (3-6) is a two-layer solution of the Stephan equation which is useful for calculations involving snow cover, a gravel pad or a board of thermal insulation, in which case the surface layer has no latent heat and the equation is simplified. Equation (3-7) is a close approximation of the Neuman solution when the ground temperatures are near freezing. Equation (3-8), the modified Berggren equation, is perhaps the most commonly used solution for soils (93). It is understood that DOT, research section, as a computer program for the solution of this modified Berggren equation.

It should be noted that with high moisture content soils the coefficient approaches unity, the simple Stephan solution. In northern climates where the mean annual temperature is near or below freezing the thermal ratio approaches zero and the coefficient is greater than 0.9.

In very dry soils, the soil warming or cooling can be significant and should be included. Multilayered soil systems can be solved by determining that portion of the surface freezing or thawing index required to penetrate each layer. The sum of the thicknesses of the frozen or thawed layers whose indices equal the total index is equal to the depth of freeze or thaw. The partial freezing or thawing index to penetrate the n^{th} layers is:

$$I_n = \frac{L_n \cdot d_n}{2} \sum_{i}^{n-1} R + \frac{R_n}{2} \tag{3.9}$$

where I_n = the partial freezing or thawing index required to penetrate the n^{th} layer

L_n = volumetric latent heat in the n^{th} layer

d_n = thickness of the n^{th} layer

χ = the coefficient based on the weighted average values for ∂ down to and including the n^{th} layer

$\sum_{i}^{n-1} R$ = the sum of the thermal resistance of the layers above the n^{th} layer

and

$R_n = \frac{d_n}{k_n}$; the thermal resistance of the n^{th} layer

The solution of multilayered systems is facilitated by tabular arrangement of the intermediate values. The penetration into the last layer must be solved by trial and error to match the total freezing or thawing index at the site (see Appendix A).

It is necessary to determine the temperature condition at the ground surface to determine subsurface thermal effects, including the depth of freezing and thawing. Since air temperatures are readily available but surface temperatures are not, a correlation factor which combines the effects of radiation, convective and conductive heat exchange at the air-ground surface is used; it is termed the "n-factor".

$$I_g = n \cdot I_a \tag{15-14}$$

where I_g = ground surface freezing or thawing index,

I_a = air freezing or thawing index

and

n = n-factor, ratio of the surface and air temperature indices.

The n-factor is very significant in analytical ground thermal considerations. It is highly variable and is usually estimated from published observations such as those values suggested in Table 3-11.

Air or ground temperatures can often be reasonably estimated as a sinusoidal temperature fluctuation which repeats itself daily and annually. This temperature pattern is attenuated with depth and in a homogeneous material (soil) with no change of state the temperature at any depth and time can be calculated from the equations in Figure 3-3. This simple solution indicates the trends found in actual ground temperatures but, in practice, they can be significantly modified by the effects of latent heat, differences in frozen and thawed soil thermal properties (conductivity and diffusivity), non-homogenous materials, and non-symmetrical surface temperatures because of seasonal snow cover, vegetation, and other local climatic influences. No analytical closed-form solution which considers these effects exists, but numerical computer solutions which can take some of these factors into account are available.

3.3.2 Non-Frost Susceptible (NFS) Subgrade

In areas where the soils are NFS or where precipitation and groundwater conditions preclude significant ice segregation, design principles are the same as in temperate zones. Pockets of frost susceptible soils should be excavated to a depth of 5 or 6 ft. below the final grade and backfilled with NFS materials.

3.4 Complete Protection Method

In continuous permafrost regions, a design that limits seasonal thaw to a non-frost susceptible base course keeps the subgrade frozen and prevents frost heaving or damaging subsidence. The required base course thickness may best be computed using the modified Berggren equation. Other equations have been presented in the previous Section 3.2.2, or maybe approximated from Fig. 3-4. Some typical conditions are taken as examples and the corresponding thaw penetration depths are determined (Ref. Appendix A).

In order to use the minimum base course thickness required to restrict seasonal thaw to the roadway section, the use of relatively high moisture retaining NFS soils, such as uniform sands, in the lower base should be considered. After initial freezing, such soils provide a "heat sink" that resists the thaw penetration because of high latent heat.

In areas where some heaving can be tolerated the use of frost susceptible soils of groups F-1 and F-2 in a subbase may be permitted and subbase course is treated as the subgrade in determining the upper base course thickness by the reduced subgrade strength method. An insulating layer may also be used to limit seasonal thaw penetration (Ref. Section 4.6.1).

3.5 Reduced Subgrade Strength Method

In discontinuous permafrost regions the roadway sections required to prevent seasonal thawing and freezing effects in the subgrade is generally greater than 6 ft. (except for extreme cold areas) and design must usually be based on the assumption that thawing and freezing will occur in the subgrade.

This method may be used for the design of flexible pavements on F-1, F-2, F-3 and F-4 subgrade soils and when subgrade conditions are significantly uniform to preclude objectionable differential heaving or subsidence. This method may also be used for design of flexible pavements used for minor or slow-speed traffic over all subgrades when appreciable non-uniform heave on subsistence can be tolerated.

Thickness requirements for the reduced subgrade strength method provide adequate carrying capacity during the period of thaw weakening but may result in objectionable surface roughness and cracking due to heaving or subsidence. In such cases, design studies should include the frost heaving and settlement experience records from existing roadway pavements in the vicinity.

Design curves presented for seasonal frost areas may also be used for the discontinuous permafrost areas to determine the thickness of roadways.

3.6 New Design Concepts

The latest design principles are described in this section. Based on the field and laboratory studies of roadway materials, and the measurement of various data on the performance of newly built roadway sections, these new concepts may be improved. Necessary construction details may also be finalized to use these design principles.

3.6.1 "Excess Fines" Concept

Recently, the Research Section, Department of Transportation and Public Facilities (29) developed a concept named "excess fines" for the design of roadway sections on cold regions. The design procedure is based on:

a) The Critical Maximum Fines Contents throughout the pavement structure, as determined from analysis of 120 Alaskan pavement structures. This analysis has demonstrated that under Alaskan conditions, pavement structures which have -#200 (fines) content less than maximums shown by Figure 3-5 at depths from the bottom of the pavement to 48 inches will have maximum Benkleman Beam deflection levels of less than .034 inches. Calculations show for this condition D avg. = .025" & Std. Dev. = .0056". The upper 95th percentile is therefore .025 + 1.65 (.0056) = .034.

b) The reduction in stress with depth of a typical vehicle axle loading, as determined from Boussinesq analysis of the stress reduction factor (SRF) beneath an equivalent circular loaded area on a homogenous elastic solid, as shown by Figure 3-6.

c) Fines content in excess of those shown by Figure 3-6 in any layer of the pavement structure are termed "Excess Fines," and have been found to contribute to increased frost susceptibility and much higher maximum Benkleman Beam Deflections than for structures without excess fines.

d) The effect of Excess Fines on Deflections for all layers of a given pavement structure can be represented by a single number called the "Excess Fines Factor" (EFF). The average deflection for pavement structures with excess fines has been found to be D avg. = .031 + .0035 (EFF). The standard error of estimate was .015 inches. The upper 95th percentile level of deflection is therefore, 1.65 (.015) + .031 = .056 + .0035 (EFF).

e) The contribution of excess fines in a given layer is determined by the following steps:

1) Separate pavement structure to be analyzed into a series of layers of a thickenss no greater than six inches between the bottom of the pavement and a depth of one foot, and thicknesses no greater than one foot beneath that depth.

2) For each layer having an average fines (P_{200}) content in excess of the maximum value (P_{Cr}) for the center of that layer from Fig. 3-5, calculate the excess fines: (P_{200} - P_{Cr})

3) Calculate the Excess Fines Factor (EFF) for all layers by the following equation:

$$EFF = \sum_{j=1}^{n} \Delta SRF_j (P_{200} - P_{Cr})_j^{0.8}$$

where: ΔSRF_j = Change in Stress Reduction Factor for each layer, from Figure 3-6

j = Layer number

n = Total number of layers to 48

P_{200} = Average percent of -#200 size for each layer

P_{Cr} = Critical percent of -#200 size for middle of each layer, from Figure 3-5

4) Calculate the probable maximum deflection level (D_{max}) from the following equation, which has been found to represent the upper 95th percentile:

$$D_{max} = .056 + .0035(EFF)$$

5) Calculate the Design Traffic Number over the required pavement design life (usually 20 years) from the total thawing season equivalent 18^k axle loadings (EAL) by the following equation:

$$TI = 8.87(EAL_{18K} \times 10^{-6})^{0.119}$$

or:

$$TI = 4.93DTN)^{0.119} \text{ for 12 mo. season (Design Traffic No. = DTN)}$$

or:

$$DTN = (TI/4.93)^{8.4}$$

6) Determine the required pavement thickness from Figure 3-7 by entering the D_{MAX} calculated in Step 4 on the horizontal axis, moving vertically to to intersect the appropriate DTN to determine the required Asphalt Concrete Pavement Thickness (T_p). The example is presented in Table (3-12).

The entire traffic analysis and excess fines design method is presented in the latest Pavement Design Guide by DOT (29).

3.6.2 Design Based On Resilient Modulus

Numerous investigations (31,32) of the resilient modulus (total stress divided by the recoverable strain) of soils subjected to freezing and thawing have been carried out. It has been reported that the resilient modulus can be determined in the laboratory by repeated load triaxial tests on samples obtained undisturbed while frozen. Field plate-bearing tests may also be carried out to obtain a range of values for different soils constituting the pavement profile.

Once the resilient modulus properties of the materials in the roadway profile are characterized, the pavement can be analyzed as an elastic layered system to calculate the anticipated deflection. From the deflection, the potential damage or performance may be ascertained. The resilient modulus is a function of deviator stress, confining pressure, moisture content, dry density and temperature. Due to these numerous parameters, it is very difficult to characterize a true representative value for a specific site. A statistical analysis approach may be used to obtain a range of modulus values for the theoretical computation of deflections under different loading configurations.

3.6.3 Frost Protection Layers

Phukan (33) reported that the fill materials below the base course may be treated with portland cement admixtured with various agents like calcium lignosulfonate and hydroxylated carboxylic acid to reduce frost susceptibility and improve physical properties to resist thaw weakening. Such treated soils may be tested (33) to check the potential frost heave and thaw weakening as well as reduction of shear strength due to freeze-thaw cycles followed by CBR test. This CBR value may be used to design roadway sections under the design principle of the limited subgrade frost protection.

This approach is usually useful in areas where the availability of NFS materials is limited and the hauling distance may be too far away from the construction area to be cost effective. The marginal aggregates treated with cement and asphalt emulsions may be used as base or subbase materials.

Typical design "k" values for cement treated soils are presented in Table 3-13. Relationships between the subgrade modulus (k) and base shown in Fig. 3-8 may be used to design the required fill thickness.

TABLE 3-1 REGIONAL FACTORS*

Condition	R - Value
a) Roadbed materials frozen to depth of 5 in. or more	0.2 - 1.0
b) Roadbed materials dry, summer and fall	0.3 - 1.5
c) Roadbed materials wet, spring thaw	4.0 - 5.0

* Source AASHO INTERIM GUIDE

TABLE 3-2

Structural Layer Coefficients Proposed by AASHO Committee on Design,[a] October 12, 1961

Pavement Component	Coefficient[b]
Surface course	
Roadmix (low stability)	0.20
Plantmix (high stability)	0.44*
Sand asphalt	0.40
Base course	
Sandy gravel	0.07[c]
Crushed stone	0.14*
Cement-treated (no soil-cement)	
Compressive strength @ 7 days[d]	
650 psi or more	0.23[c]
400 psi to 650 psi	0.20
400 psi or less	0.15
Bituminous-treated	
Course-grated	0.34[c]
Sand asphalt	0.30
Lime-treated	0.15-0.30
Subbase course	
Sandy gravel	0.11*
Sand or sandy clay	0.05-0.10

*Established from AASHO Road Test data.
[a]From AASHO Interim Guide.
[b]It is expected that each state will study these coefficients and make such changes as experience indicates necessary.
[c]This value has been estimated fom AASHO Road Test Data, but not to the accuracy of those factors marked with an asterisk.
[d]Compressive strength at 7 days.

TABLE 3-6

Design Index Categories for Traffic

Design Index	General Character	Daily EAL[a]
DI-1	Light traffic (few vehicles heavier than passenger cars, no regular use by Group 2 or 3 vehicles)	5 or less
DI-2	Medium-light traffic (similar to DI-1, maximum 1000 VPD,[b] including not over 10% Group 2, no regular use by Group 3 vehicles)	6-20
DI-3	Medium traffic (maximum 3000 VPD, including not over 10% Group 2 and 3, 1% Group 3 vehicles)	21-75
DI-4	Medium-heavy traffic (maximum 6000 VPD, including not over 15% Group 2 and 3, 1% Group 3 vehicles)	76-250
DI-5	Heavy traffic (maximum 6000 VPD, may include 25% Group 2 and 3, 10% Group 3 vehicles)	251-900
DI-6	Very heavy traffic (over 6000 VPD, may include over 25% Group 2 or 3 vehicles)	901-3000

[a]EAL = equivalent 18 kip axle loads in design lane, average daily use over life expentancy of 20 years with normal maintenance.
[b]VPD = vehicles per day, all types, using design lane.

(Courtesy of the National Crushed Stone Association)

TABLE 3-7

Basic Design Thickness Table

Subgrade Soil		Design Thickness (inches) for Indicated Traffic Intensity Categories					
Class	CBR	D1-1	D1-2	D1-3	D1-4	D1-5	D1-6
Excellent	15+	5	6	7	8	9	10
Good	10-14	7	8	9	10	11	12
Fair	6-9	9	11	12	14	15	17
Poor[a]	5 or less	Subgrade Improvement Recommended					

[a]Poor subgrade soil should be improved to "fair" or better by protecting them with available "select" materials or by stabilization. The depth or improvement required should be adequate to provide protection to unimproved soil beneath (determined from Figure 5).

(Courtesy of the National Crushed Stone Association)

TABLE 3-8 Thickness Design

Traffic Category	Component	Very Good: Full Depth Asphalt	Very Good: Asphalt Surface	Very Good: Concrete	Good: Full Depth Asphalt	Good: Asphalt Surface	Good: Concrete	Poor: Full Depth Asphalt	Poor: Asphalt Surface	Poor: Concrete
11	Surface		2.75			4.0			6.0	
	Base		12.0			14.0			18.0	
	Total	8	14.75	6	11	18.0	8	16	24.0	10
9	Surface		2.5			3.5			5.0	
	Base		11.0			13.0			16.0	
	Total	7	13.5	6	9	16.5	7	13	21.0	9
7	Surface		2.0			3.0			4.0	
	Base		10.0			12.0			14.0	
	Total	6	12.0	6	8	15.0	6	11	18.0	8
5	Surface		1.75			2.0			2.75	
	Base		8.0			10.0			12.00	
	Total	5	9.75	5	6	12.0	6	8	14.75	6
3	Surface		1.5			1.75			2.5	
	Base		6.0			8.0			11.0	
	Total	4	7.5	5	5	9.75	5	7	13.5	6

Thickness, Inches, for Subgrade Class

(After Baker et al.)(3)

Notes:

Traffic Catagory	Equivalent 18,000 lbs Single Axle Load	Designation
11	30,700,000	Freeways
9	6,570,000	Local Street
7	430,000	Connector Street
5	9,500	Local St., Residentia

TABLE 3-9

TYPE OF FACILITY	T.I.
Minor residential streets and cul-de-sacs.	4
Average residential streets.	4.5
Residential collectors and minor or secondary collectors.	5
Major or primary collectors providing for traffic movement between minor collectors and major arterials.	6
Farm-to-market roads providing for the movement of traffic through agricultural areas to major arterials.	5 - 7
Commercial roads (arterials serving areas which are primarily commercial in nature).	7 - 9
Connector roads (highways and arterials connecting two areas of relatively high population density).	7 - 9
Major city streets and thoroughfares.	7 - 9
Streets and highways carrying heavy truck traffic. This would include streets in heavily industrialized areas.	9+

(After 17)

TABLE 3-10

CHARACTERIZATION OF MATERIAL PROPERTIES FOR FLEXIBLE PAVEMENTS IN SEASONAL FROST AREAS[a]

Properties That Should Be Characterized	Important for (Area of Design)	Need to Characterize Dependence on Temperature, Frost, or Moisture?[b]
Strength:	Fracture under large load	
Tensile (bound layers only)	Slippage under braking load Thermal Cracking	Temperature
Fatigue (bound layers only)	Fracture under repeated loading	Temperature
Shear	Plastic flow or shear under single or few excessive loads	Frost, moisture, temperature
Resistance to deformation:		
Modulus (stress-strain) Poisson's ratio	Analysis of stresses and strains for application to fracture and distortion mode of distress	Asphalt-bound: temperature Clean granular unbound base: moisture Clean granular cohesionless soil: none Silty and clayey soils: frost, moisture, temperature
Constancy of volume:		
Susceptibility to frost heave	Distortion by differential heave	Frost, moisture
Expansive properties	Distortion by differential heave	Moisture, frost
Underconsolidation	Distortion by differential settlement	Moisture, frost (consolidation during thaw)

[a]After Deacon (20)
[b]Dependence on other parameters, though highly significant, is beyond the scope to the present synthesis.

TABLE 3-11 SOME EXAMPLES OF n-FACTORS (28)

CASE	SURFACE TYPE	n-FACTORS FREEZING	n-FACTORS THAWING	COMMENTS	
1	Spruce trees, brush, moss over peat soil	.29	.37	Fairbanks, Alaska	U.S. Army (1950a)
2	Brush & trees cleared moss in place, peat soil	.25	.73		
3	Vegetation and 16" of soil stripped clean	.33	1.22		
4	Turf	.5	1.0	Alaska and Greenland	U.S. Army (1966)
5	Snow	1.0	-		
6	Sand & Gravel	.9	2.0		
7	Gravel	.76	1.99	Fairbanks	U.S. Army (1950a)
8	Gravel	.63	2.01		
9	Gravel	.6	1.4	Fairbanks	Carlson(195
10	Elevated Building	-	1.0	Alaska	U.S. Army (1966)
11	Pavement without snow	.9	-		
12	Pavement north of 45°N	.9	-	General	Sanger(1959
63	Gravel	1.0	- 1.47	Chitna,Ak	Esch(1973)
64	Gravel colored dark	-	1.40	Fairbanks	Berg(1973)
74	Sandy soil,with snow	.49	-	Lakselv	Heiersted(19
75		.02	-	OS	Norway
76		.53	-	Amli,1974	
77		1.39	-	Amli,1975	
78	General	.8	-	Southern Canada@<50°N"	McCormack (1971)
79	General	.9		Northern Candad@>60°N	
80	Gravel	-	1.5	Fairbanks	U.S. Army (1972)
81	Gravel colored dark	-	1.27		

N-Factor data, general surfaces

TABLE 3-12

Column	1	2	3	4	5	6	7	8
Obtained from:	Trial Dimensions	Specifications or Field Data	Fig. 3-20	3 - 2	Fig. 3-21	Fig 3-21	6 - 5	$7 \times 4^{0.8}$
Layer Number	Depth Interval (in.)	Fines Content (P200)	Maximum Fines (Per)	Excess Fines (%)	SRF@ Top of Layer	SRF@ Bottom of Layer	(SRF)	(EFF)
1	0-6"	6	6	0	0	0.45	0.45	0
2	6-12"	8	6	2	0.45	0.68	0.23	0.40
3	12-24"	12	9	3	0.68	0.87	0.19	0.46
4	24-36"	20	20	0	0.87	0.95	0.07	0
5	36-42"	92	42	50	0.95	0 97	0.02	0.46
								1.32

Calculations:

Traffic Data: TI = 7.3

EFF = Column 8 Total = 1.32

Design Traffic No. (DTN) = $\left(\frac{TI}{4.93}\right)^{8.4} = 27$

Table 3-13(a)
Design k Values for Cement-Treated Subbases

(Subgrade k value-approx. 100 pci)

Thickness, in.	k value, pci
4	300
5	450
6	550
7	600

CRREL Report No. 259

Table 3-13(b)
Effect of Untreated Subbase on k Values, pci

Subgrade k value	Subbase k value 4 in.	6 in.	9 in.	12 in.
50	65	75	85	110
100	130	140	160	190
200	220	230	270	320
300	320	330	370	430

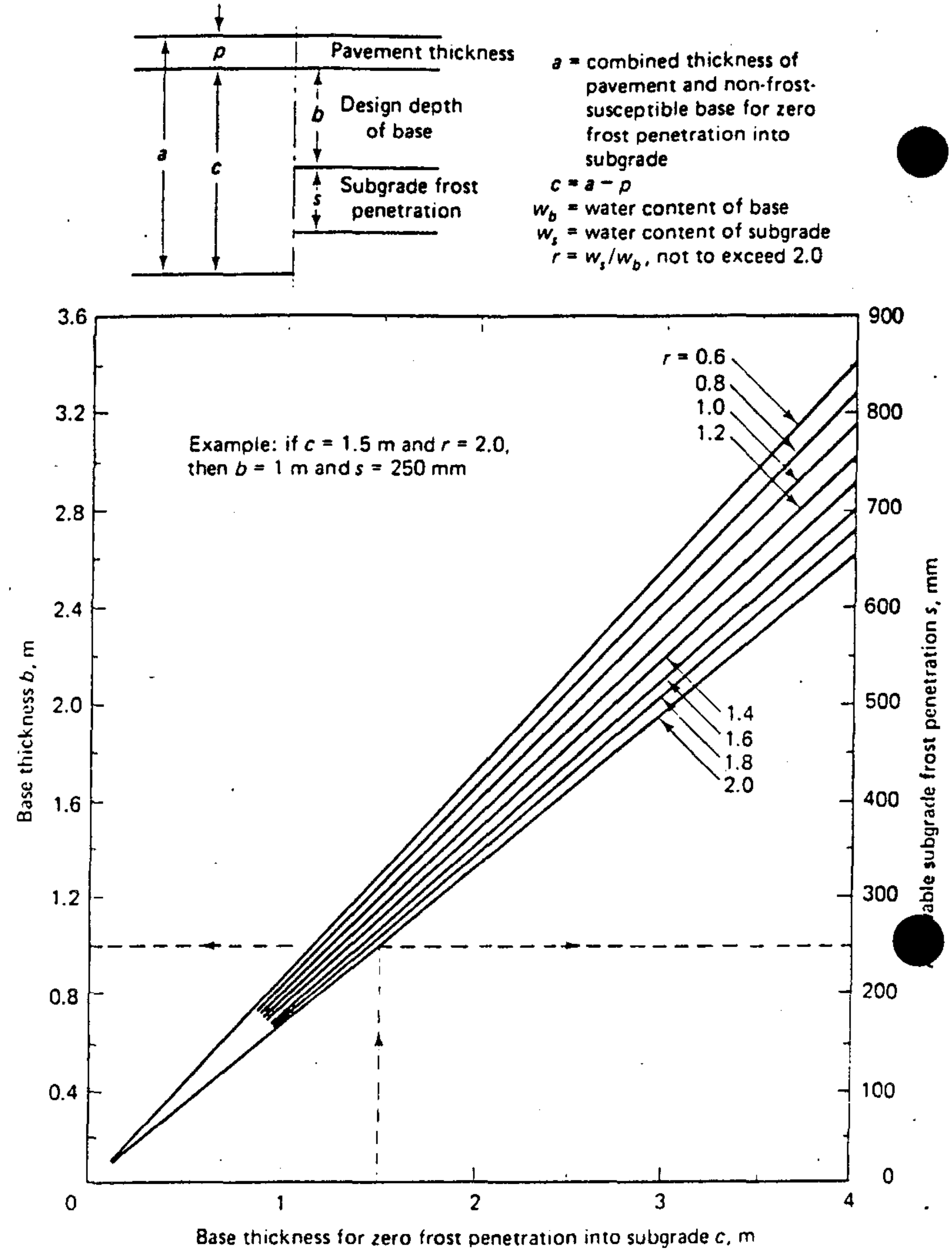


FIG. 3-1 BASE THICKNESS VS FROST PENETRATION (After Ref

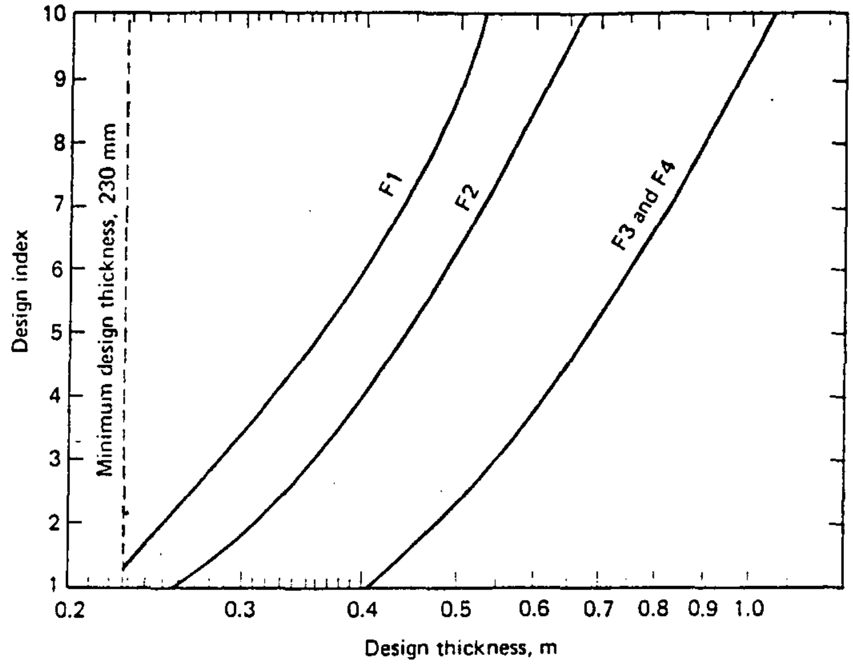


FIG. 3-2 DESIGN INDEX VS DESIGN THICKNESS (After Ref. 1

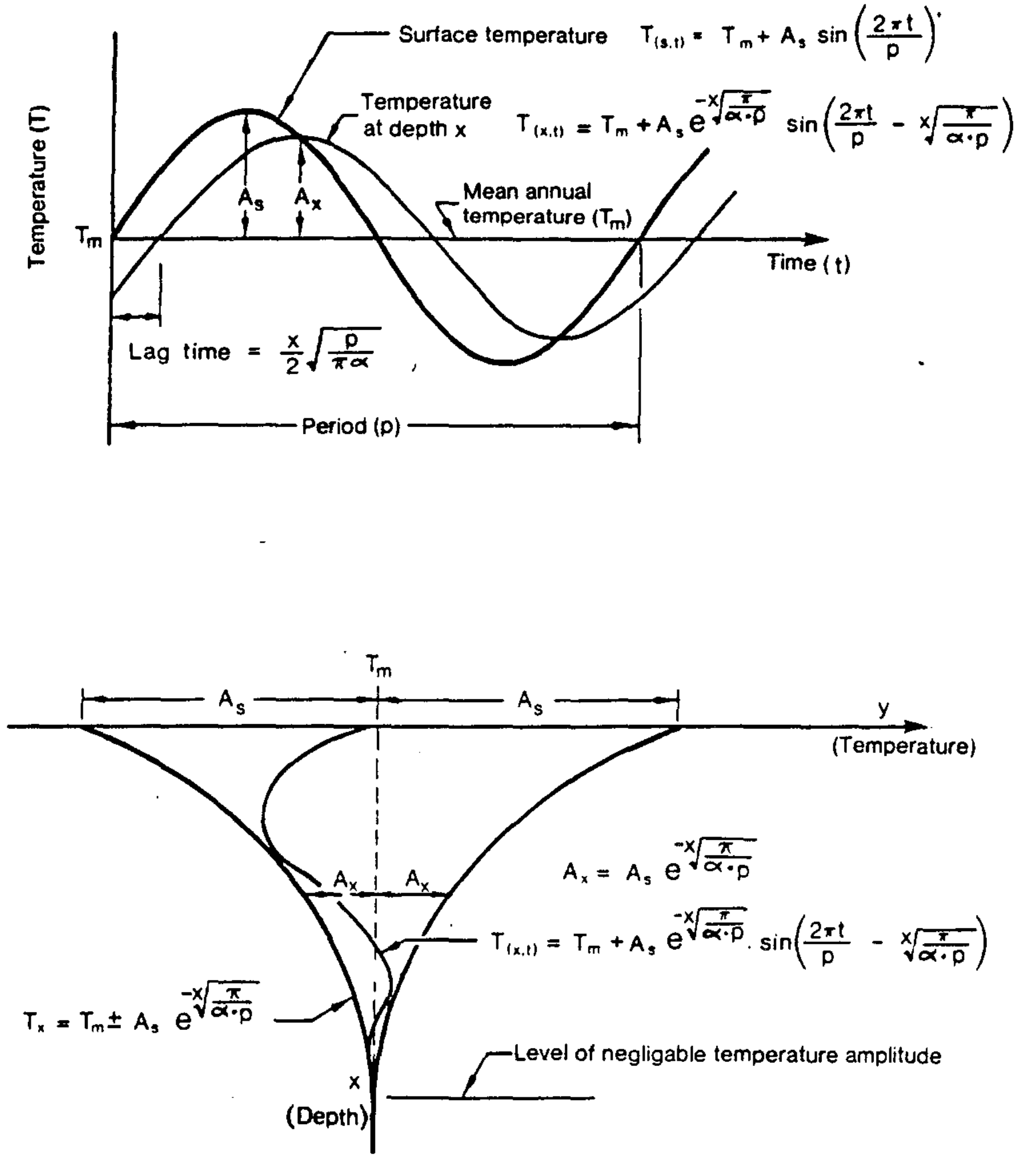


FIG. 3-3 SINUSOIDAL AIR AND GROUND TEMPERATURE VARIATION

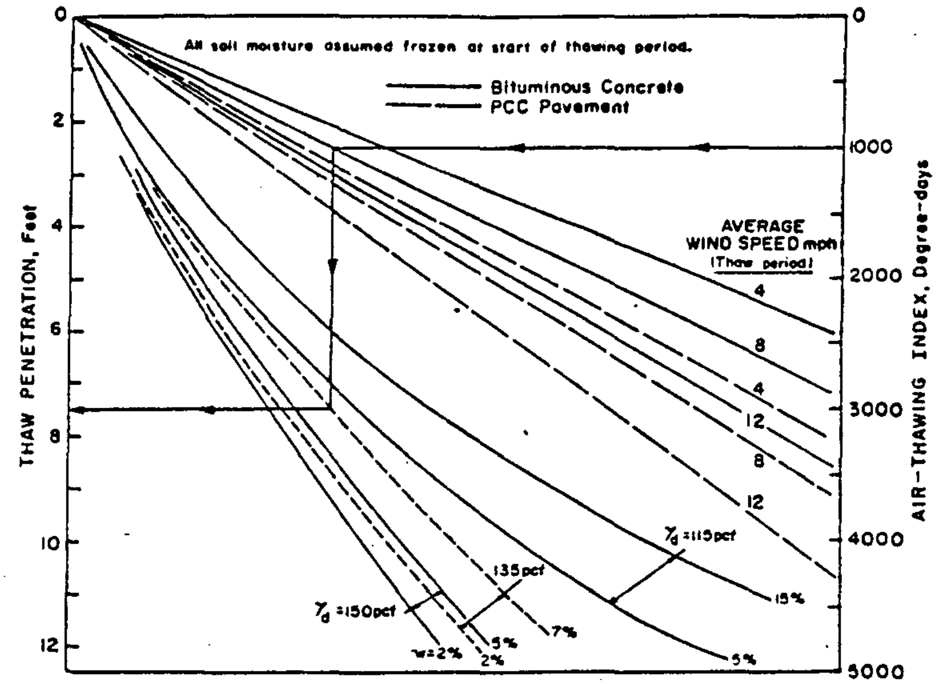
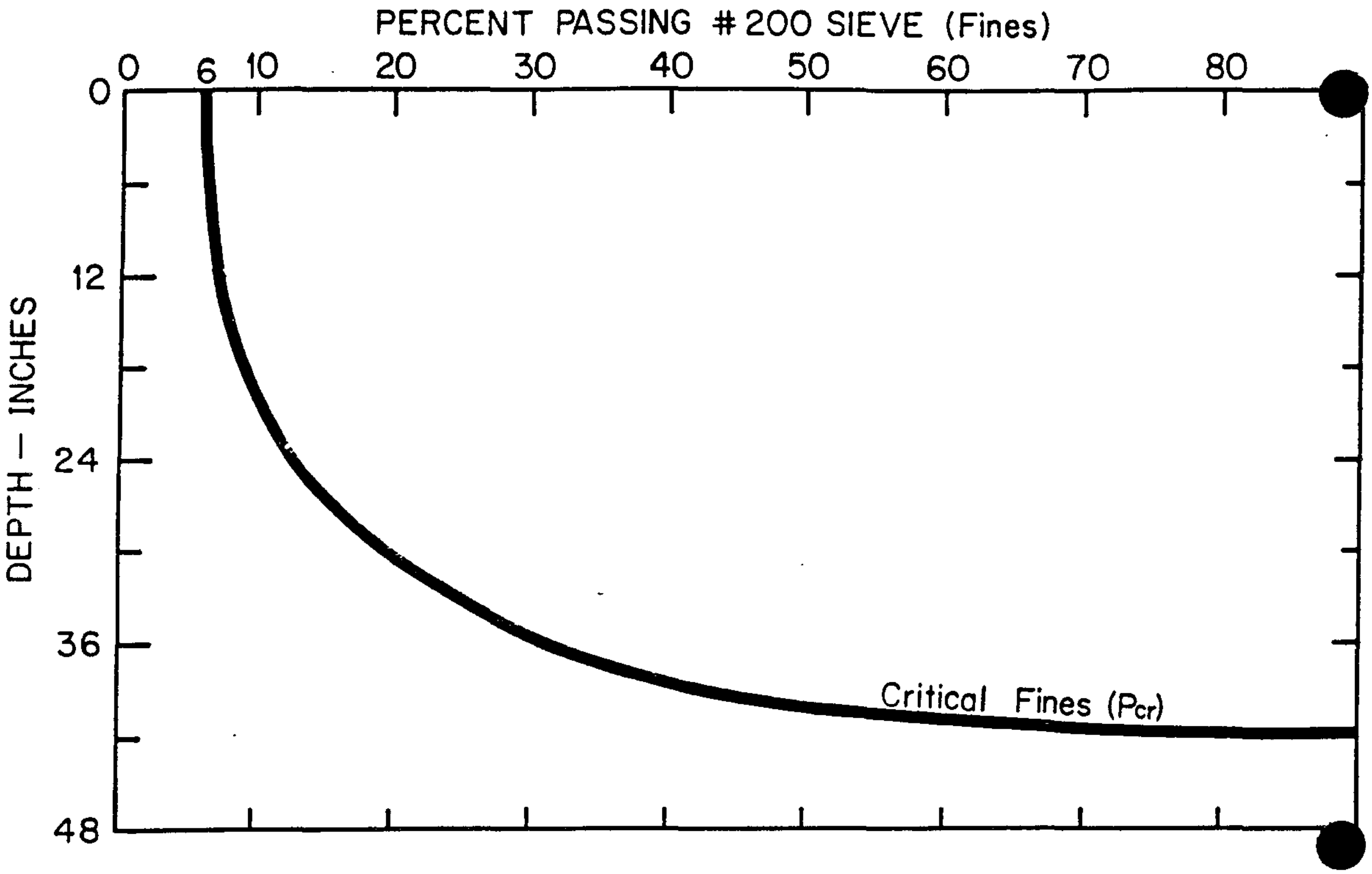


FIG. 3-4 RELATIONSHIP BETWEEN THAWING INDEX AND THAW PENETRATION (After Ref. 35)



CRITICAL FINES CONTENTS vs DEPTH FOR MINIMUM THAW WEAKENING OF PAVEMENT STRUCTURES

FIG. 3-5 CRITICAL FINES RELATIONSHIP (After Ref. 29)

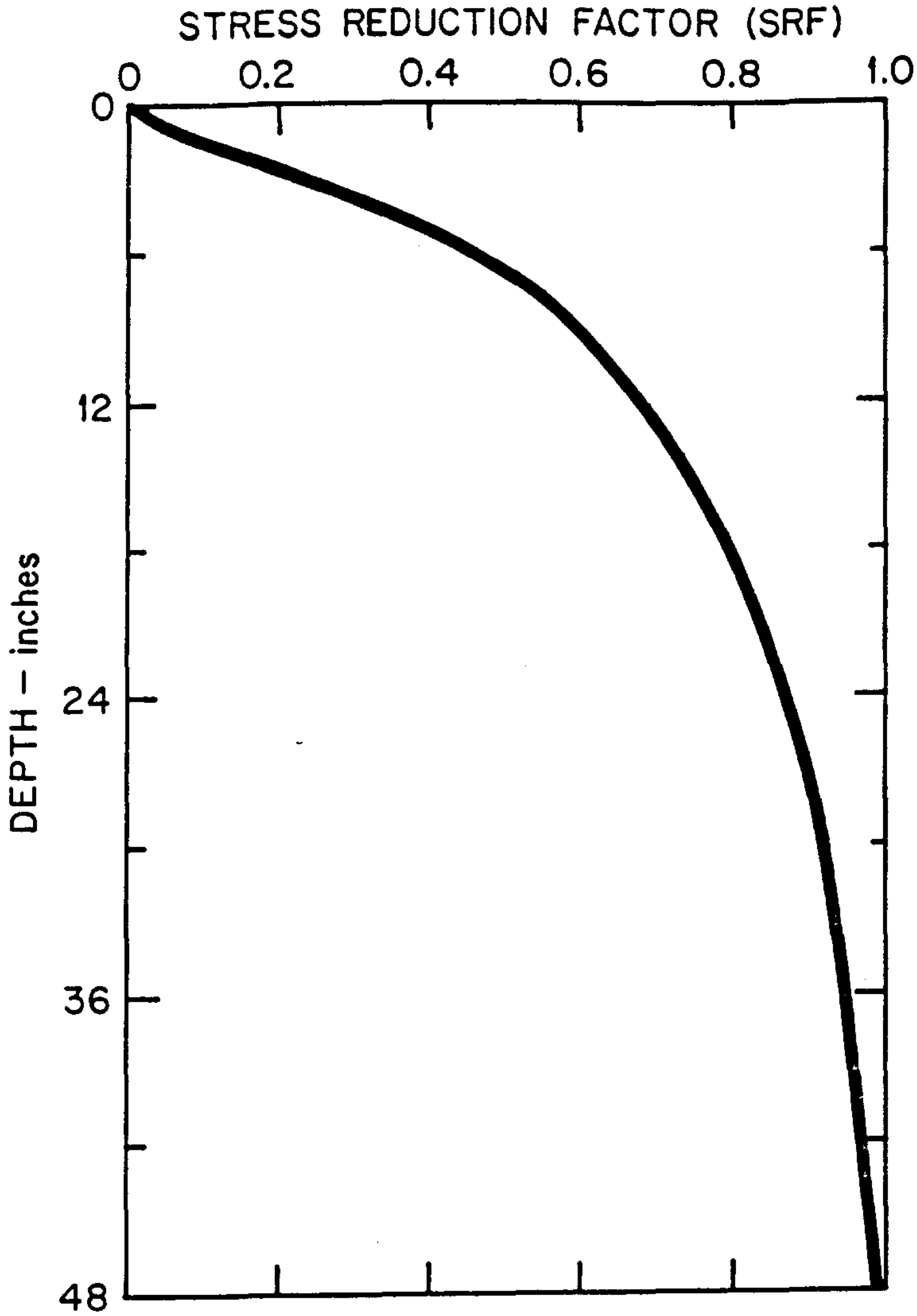


FIG. 3-6 STRESS REDUCTION FACTOR (After Ref. 29)

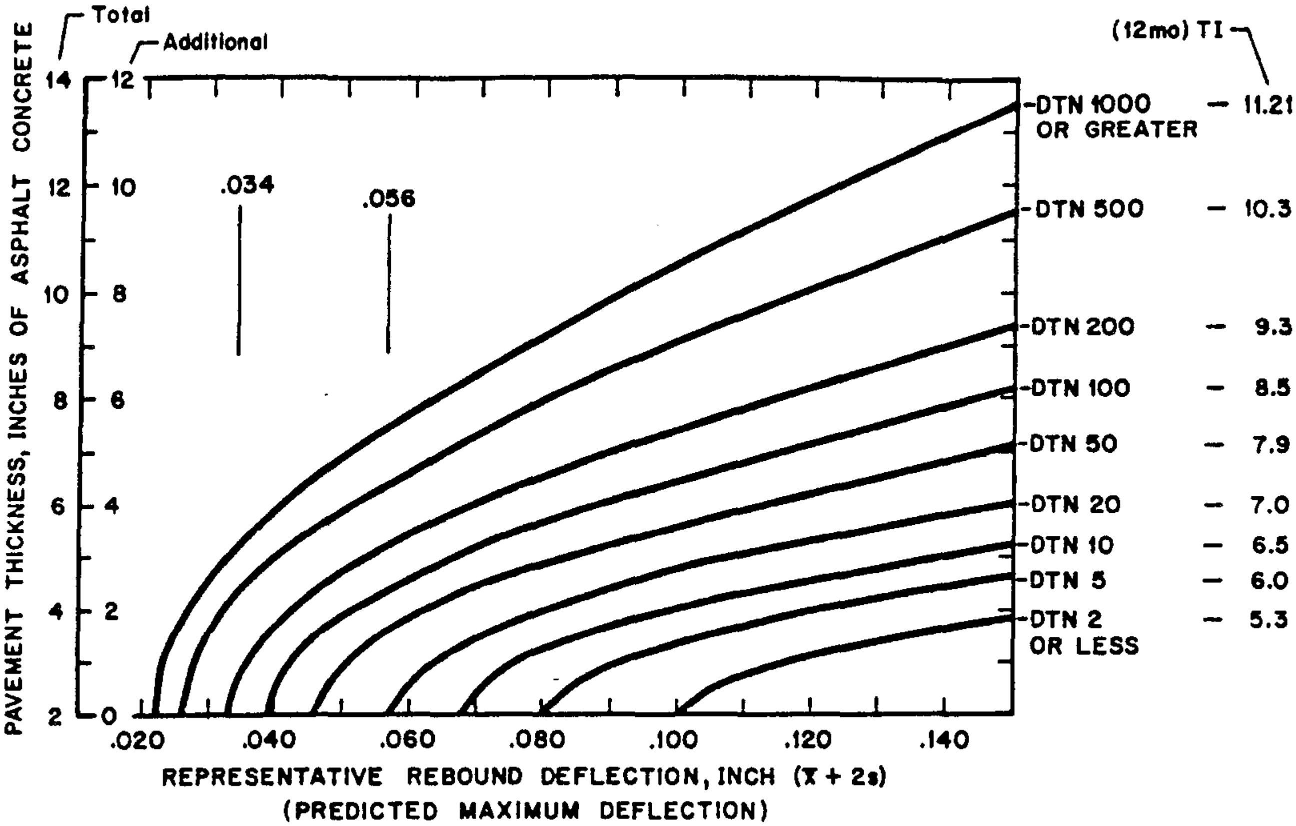


FIG. 3-7 PAVEMENT THICKNESS DETERMINATION (After Ref. 29)

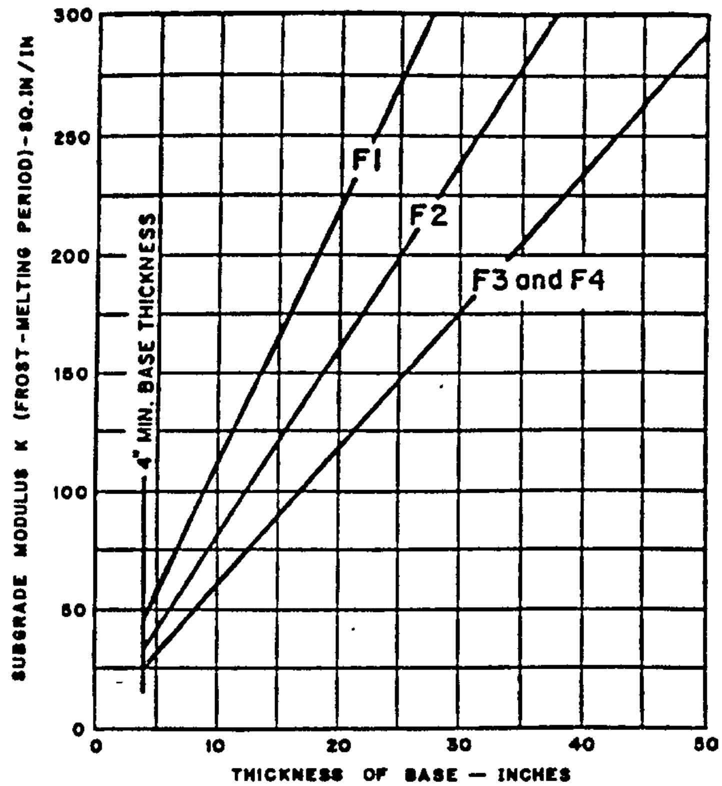


FIG. 3-8 SUBGRADE MODULUS VS BASE THICKNESS (After Ref. 2)

4. ROADWAY CONSTRUCTION IN PERMAFROST REGIONS

4.1 General

There are two principal concepts to be considered with regard to the construction of roadways on permafrost; 1) the "passive approach where the roadfill is constructed with the intention of preserving the permafrost, and 2) the "active" approach where preservation of permafrost is not possible or practical and the consequence of thaw is allowed for in the design. Another factor to be considered during the construction of roadways on permafrost is that the thermal degradation during the construction periods should be minimal unless thaw-acceleration is desired. Other factors influencing contruction include topography, soil and rock conditions, permafrost conditions, drainage, economics, scheduling and last, but not least, environmental impact. The design and construction of any road, in addition, is based to some degree on the optimum utilization of available suitable construction materials and the cost of alternate sources. The availibility of suitable construction materials may be a major factor in the design and construction of roadways on permafrost.

For any section of the route the design should compare the cost of maintenance (due to settlement and frost heave) with the cost of reducing it by using thicker, wider fills and controlled construction techniques in continuous permafrost or by allowing the permafrost to melt to the maximum possible extent during the construction period in discontinuous permafrost.

In continuous permafrost regions where it is intended that permafrost should be preserved to the maximum extent possible, winter construction operations may be advisable, particularly when no stripping or ground travel can be permitted on the right-of-way. All culverts may be placed during the winter when the water levels are low. The initial "pioneer" fill is placed directly on undisturbed ground during the winter, when the active layer is completely frozen, minimizing both consolidation of the active layer and degradation of permafrost. The initial layer is placed to the minimum height required to prevent permafrost from thawing during the following summer. This is based on the thermal calculations described in the previous sections. Winter construction decreases the volume of materials required and the subsequent settlement of fills and hence short term maintenance costs. It also minimizes environmental damage. Placement of fill above the minimum required to preserve permafrost can be completed during the following summer. Where it is intended that permafrost is to thaw and settlement is anticipated, all operations should be scheduled with thermal factor in mind. Clearing, stripping, natural conditions and a road section are shown in Fig. 4-1.

4.2 Embankment Construction on Permafrost

The "fill" or "embankment" is to be properly designed and constructed, to satisfy proper service and performance of roadways. A typical instability of an embankment section on permafrost is shown in Fig. 4-2.

The right-of-way may be cleared by hand or "hydro-axe". The embankment is constructed by the "overlay" or end dumping method in order to minimize the disturbance of vegetative cover by hauling equipment. A typical "cut" and "fill" sections for roadways constructed on "thaw stable" material are shown on Fig. 4-3. Materials brought from the borrow areas are placed on the end of the fill by the hauling equipment and pushed forward onto the undisturbed terrain by bulldozer, which compacts it as it moves back and forth. Whether placed in winter or summer, the construction must maintain the required thermal stability of the permafrost terrain. Fig. 4-4 presents a typical example of embankment construction with or without insulation.

Some cracking and sloughing of the shoulders should be anticipated because the rate and depth of thaw under the side and at the toe of the slopes will be greater than under the main body of the embankment (Ref. Fig. 4-2). In areas where thaw-unstable conditions are anticipated, the side slopes must be flattened, bermed or insulated to ensure that the integrity of the main embankment is maintained (Ref. Fig. 4-4).

Esch (30) presented a case study of the use of berm and insulation layers on an embankment section which was build on thaw-unstable soils (Fig. 4-5 and 4-6).

Conventional slopes can be constructed if the fill materials are dry and relatively well-drained and the foundation is stable when thawed. Generally, unstable conditions arise from the use of frozen soils which are difficult to compact. Problems in embankment construction are most often associated with inadequacy of either the quality of fill material or foundation conditions.

4.3 Subgrade Preparation

During construction operations local adverse subsurface conditions may be exposed which may not be revealed by the design subsoil explorations. Construction inspectors must be aware of the design and take actions when such situations occur to avoid later problems. Visual identification of frost susceptible soils encountered during construction should be checked and tested for gradation.

Pockets of frost susceptible soils or sharp variation in frost heave potential of subgrade, within the active layer, can frequently be detected only during grading operations. Such conditions may be corrected by removal and replacement by NFS materials or by providing transition zones between the areas of different heave potential so as to provide a surface which will remain acceptably smooth under the particular traffic use.

When unexpected wet areas are encountered in the subgrade, which may provide a source of moisture migration for frost action, additional drainage measures should be considered.

When isolated ice wedges are encountered in stripping and excavation in permafrost areas, the ice should be removed and replaced with NFS material, if it will be within the ultimate thaw zone under the completed pavement. If the ice is extensive the "passive" method discussed in Section 3.4.1.1 may be used to construct the roadway entirely on "fill" or considerations may be given to relocating or changing the design grade. In discontinuous permafrost areas where summer thaw penetration is deep or progressive degradation of permafrost cannot be prevented, such remedial measures may not be effective or economically feasible. It will be necessary to accept substantial summer maintenance of the surface as it is unavoidable in such a case.

Because of the ability of the bedrock to supply large quantities of water to growing ice lenses, frost heaving may often be actually more severe in rock cuts than in adjacent soil areas if concentrations of fines are present at the surface of the bedrock, in mud seams or in the base course itself. Rock cuts may, therefore, frequently require as much or more depth of base course than soil subgrades, to ensure pavement free of heaving and cracking.

4.4 Fill Materials

Typical gradations of base course and subbase materials used are presented in Tables 4.1 and 4.2.

A major problem lies in the construction of fills and compaction of soil at freezing temperatures. Frozen fill materials may not be compacted properly and if it contains ice layers or lumps of snow or ice stability will be reduced and the course will settle upon thawing.

Construction of fills using quarry-run rock, crushed rock, well-drained clean gravel at low moisture content may be compacted to about 95 percent of "modified Proctor" density. However, care must be taken to exclude any large chunks of ice or snow from the fills. Winter construction using preheated soils also is technically feasible but can be economically justified only when absolutely necessary to complete the work at an early date. Surfaces of fills may be protected overnight or over the weekend by placing calcium chloride (temperature limitation $\geq$ 25°F or -4°C) or insulating material. Salt may be used to depress the freezing point of soils moisture and keep it workable at subfreezing temperatures. It is generally found uneconomical to construct fill sections when the temperature drops to 20°F (-7°C).

4.5 Cut Sections

It is a good practice to avoid cuts in permafrost wherever possible. But it is not practical for linear structures like roadways and cuts may be required to maintain design gradients, approaches to stream crossings and so on. The behavior of cuts in frozen soils is directly related to the nature and composition of soil, the distribution of ground ice within it, and the season of construction. In ice-poor clean sand and gravels, slopes may be cut at angles comparable to those used in unfrozen soils. Slope and fill movements and erosions may develop in terrain containing ice wedges and other massive deposits if proper construction measures are not utilized. Possible methods for stabilization of cut slopes in ice rich frozen ground are presented in Figs. 4-7 and 4-8. The cut slope stabilization measures depend on the amount and type of ground ice, the soil type and the depth of the cut. Other factors, such as the long-term effect on the construction and cost of maintenance, potential environmental impact and general aesthetics, will dictate the degree of protection required. Many cut slopes may self-stabilize within the first three thaw seasons, provided reasonable care has been taken in the design and construction to ensure that unacceptable thawing and continuing retreat of the slope does not occur. The timing and procedures to follow for an excavation operation will depend on several factors such as the size of cut, soil conditions and equipment available. The use of excavated frozen cohesive soils in adjacent fills may not be possible and disposal of the material in a selected area must be considered in the design and scheduling of constuction. In some areas (close to streams), exceptional measures such as ditch checks and settling ponds may be necessary for environmental protection to avoid erosion and stream siltation caused by melt water and normal runoff from the cut.

Cuts in ice rich frozen soils generally present a serious problem and a detailed geotechnical investigation is very useful to select appropriate design and construction measures for deep cuts. The cut face may be nearly vertical to reduce the area exposed to thawing and the equilibrium condition may be achieved quickly. For shallow cuts ($\geq$ 6 ft. depth), vertical cut slopes may be made at approximately the ditch line. In deeper cuts in ice rich soils ditches should be at least 12 ft. wide so that they can be cleared periodically by earth-moving equipment. Preferrable back slopes are 1H:4V. A wide ditch will also permit placement of a revetment of rock backfill at or against the bottom of the cut slope. Surface run-off from the slope above the top of a cut should be intercepted and diverted to the side by constructing small dikes (rather than ditches) on the ground surface to prevent water from running down and eroding the face of the cut. Trees should be hand cleared from the top of the slope to a distance of from one to two times the height of the slope (Ref. Fig. 4-7).

The type of the slope protection method to be used will vary depending on the slope angle, available of granular materials, amount of excavation and severity of erosion. Several possible techniques are shown in Fig. 4-8.

4.6 Other Construction Techniques

Thermal balance in the subgrade may be maintained by thermal barriers such as insulation or materials having very low conductive values. Another construction approach may be to prevent moisture from migrating to the freezing surface by means of moisture barriers. Thaw weakening processes of subgrade soils may be counteracted by soil reinforcement to prevent any consolidation or subsidence of roadway sections. These techniques are discussed under the following headings.

4.6.1 Insulated Embankments

This method consists of conventional base and subbase layers above an insulating material of suitable properties and thickness to prevent the frost or thaw penetration into a frost susceptible subgrade or ice rich frozen ground. Layers of granular materials should be placed below the insulation to contain that part of frost or thaw penetration which occurs below the insulation. As shown in Figures 4-10 and 4-11, the thermal insulation is more effective when layers of granular materials are placed below the insulation than when the insulation placed directly on the subsurface soils. The thickness of base materials placed above the insulation should be adequate to meet the structural requirements and may be determined by elastic layered system analysis or other methods discussed in Section 3.0. The vertical stress on the insulation material caused by dead loads and wheel loads must be less than the compressive strength of the insulation. Generally a factor of safety of 3 is used to limit the maximum vertical stress on the insulation to not more than one-third of the compressive strength of the insulating material.

Alternative combinations of thicknesses of insulation and underlying granular material required to completely contain the zone of freezing can be determined from Fig. 4-9, which shows the total depth of frost of various freezing indexes and thicknesses of insulation.

In the seasonal frost areas, the total frost given by the Fig. 4-9 may be taken as the "a" value (Fig. 3-12) and a new combined thickness of pavement, base, insulation and subbase is determined.

The thickness of granular material needed beneath the insulation is obtained by subtracting the previously established thicknesses of upper base and insulation. If the adopted design permits frost penetration below the insulation, the thickness of granular material below the insulation and subgrade should not be less than 4 in.

Insulation may also be used to resist thaw penetration into the permafrost. A case study (36) presented in Figs. 4-10 and 4-11, shows the effective use of insulation to prevent thaw penetration below the embankment. Fig. 4-12 illustrates the significant difference in thaw penetration beneath an insulated embankment. Appendix A shows the procedures which can be used to determine the thaw penetration below the insulating materials. The thermal properties of different insulating materials are also included.

4.6.2 Peat Overlay Construction Method

Esch and Livingston (37) reported on the study of an experimental roadway section which was built with four and five foot thicknesses of peat placed beneath a roadway cut section in permafrost. A typical section is shown on Fig. 4-13 and the maximum thaw depths recorded under the 4' and 5' peat sections are presented in Figs. 4-14 and 4-15 respectively. It was concluded that peat overlays can be effectively utilized beneath paved roadways to resist or prevent thawing of the permafrost. The large changes in thermal properties of peat which occur upon freezing results in wintertime conductivities roughly twice as high as during the thawing season. This change causes an altered thermal balance and was demonstrated to result in more rapid annual refreezing and lowered permafrost temperatures. For climatic conditions similar to those of Fairbanks a minimum thickness of 2.5 feet of consolidated peat is required beneath a 4 foot thickness of granular fill material to prevent thawing into the underlying permafrost. The peat must be consolidated to minimize long-term secondary consolidation.

The main drawbacks of this construction method are obtaining an adequate source of thawed peat and establishing the construction control measures for the preconsolidation of peat prior to pavement construction.

4.6.3 Membrane Encapsulated Soil Layer (MESL)

The MESL technique involves the containment of a frost suspectible soil in a membrane wrapping system to prevent moisture gains and additional frost heaving. Various investigators (38, 39, 40) reported on the encapsulation of soils with membranes to restrict heave moisture migration.

Smith (40) reported the field test results of two MESL test sections which were constructed into existing gravel surface roads at Elmendorf AFB and at Ft. Wainwright in Anchorage and Fairbanks, respectively. The Elmendorf AFB MESL contains a silty clay soil whereas the Ft. Wainwright MESL contains a nonplastic silt. Both sections were constructed at soil moisture contents of approximately 2% to 3% below optimum for the modified Proctor compacting effort.

The Elmendorf silty clay, at an average moisture content of about 16% and an average dry density of about 111 lb/ft^3, has maintained a uniform moisture profile during closed system freezing in the MESL section at a freezing rate estimated at approximately 1.0 in. per day. The prefreeze and after-thaw CBR values were also essentially the same with an average value of 18.2%. With an 8 to 10 in. sand and gravel surface layer the MESL with a CBR values in the range of 10% to 24% withstood low density, light vehicular traffic and occasional heavy truck traffic with minimal surface maintenance.

The Fairbanks silt, at an average moisture content of about 14% and an average dry density of 88 pcf, has maintained a uniform moisture profile during closed system freezing in the MESL section at a freezing rate of approximately 1.5 in. per day. Traffic use of the MESL has increased its density about 3% from an average of 88 to 90 pcf. With the 8 in. to

10 in. sand and gravel surface layer, the MESL with a CBR value in the range of 7 to 10% withstood medium density, light vehicular traffic and considerable heavy truck and construction equipment with minimal surface maintenance. The degree of saturation of MESL soils placed was found to be an important factor for the performance of the road section against the potential heave. Additional field tests are needed to verify the performance of MESL sections with higher density soils. A critical question not yet resolved for MESL designs in cold regions involve the potential effects of embankment thermal cracking on the membrane, as these cracks may totally destroy its integrity.

4.6.4 Prethawing Methods

Various prethawing techniques such as clearing and stripping, gravel pad surface darkening with asphalt, and the use of clear polyethylene film may be used to prethaw the upper layers of permafrost so that thinner roadway embankments may be constructed in permafrost regions. Esch (44) reported a latest study of various methods of surface modification which may be used to prethaw upper layers of permafrost. These methods include stripping of vegetation, polyethylene film surface covering, thin (0.3m) gravel pad, darkened gravel surface and hand cleared vegetation. The thaw depths under different methods used are presented in Fig. 4-16. Based on the field results it was concluded that thermally stable embankments may be constructed more economically by maximizing the thaw depth during a single summer prior to construction. As expected, significant increases in thaw depths resulted from machine stripping, asphalt surface darkening and polyethylene surface coverings. The benefits of gravel pad installation were primarily for drainage, surcharge, and accessibility, to dissipate excess pore pressure build-up during the thawing process and thereby to accelerate the consolidation process. Prevention of erosion and trafficability are also achieved by the use of gravel pads.

An economic analysis was also performed (Esch) and the results are presented in Table 4-3. This shows excellent benefit cost ratios for the various pre-thaw modes used. Results of this type of economic analysis must be considered relative as they will vary with construction and material costs, climatic factors and soil types. The economic aspects of distress and increased maintenance resulting from these thermally unstable designs have not been evaluated.

4.6.5 Soil Reinforcement

Goughnour and DeMaggio (41) reported on soil reinforcement methods on highway projects. Methods such as reinforced earth, stone columns, permanent soil anchors and arrays of small diameter cost-in-place piles, have provided positive and cost-effective solutions to many geotechnical problems in unfrozen soils. The use of these methods in permafrost areas related to roadway construction is limited.

As used in moderate climate the earth reinforcement method appears to be most useful for the road embankment construction in permafrost regions. Especially at bridge abutments, high cut and fill sections, the earth reinforcement method will reduce the volume of construction materials to be placed (Ref. Fig. 4-18). In thaw unstable subgrade, a layer of soil reinforcement at the bottom of the "overlay" fill section may improve the CBR value for the thawing condition and the performance of the roadway during the spring thaw. Field tests are needed to finalize the soil reinforcement technique and benefits in the construction of roadways in permafrost regions.

TABLE 4-1
AGGREGATES FOR UNTREATED BASE
PERCENT PASSING BY WEIGHT

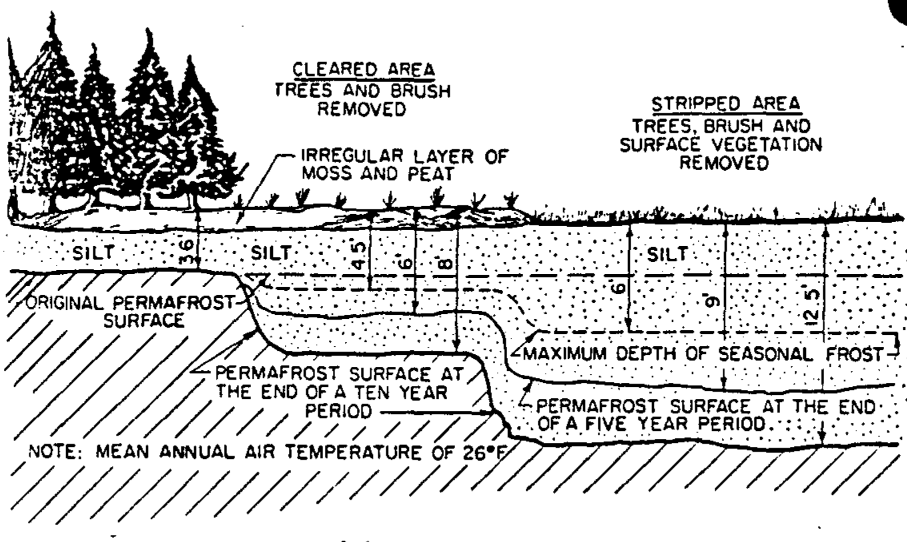
Sieve Designation	Percent Passing by Weight Grading	
	C-1	D-1
1 1/2 in.	100	--
1 inch	70-100	100
3/4	60-90	70-100
3/8	45-75	50-80
No. 4	30-60	35-65
No. 8	22-52	20-50
No. 40	8-30	8-30
No. 200	0-6	0-6

TABLE 4-2
REQUIREMENTS FOR GRADING FOR SUBBASE
PERCENT PASSING BY WEIGHT

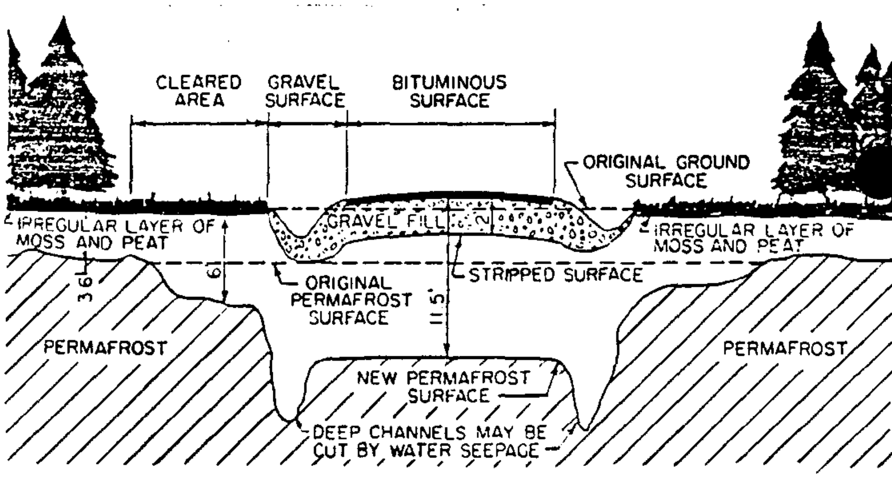
Sieve Designation	Grading				
	A	B	C	D	E
4 in.	100	-	-	-	-
2 in.	85-100	100	-	-	-
1 in.	-	-	100	-	-
3/4 in.	-	-	-	100	-
No. 4	30-70	30-70	40-75	45-80	-
No. 10	-	-	25-55	30-65	-
No. 200	10 Max.	3-10	4-10	4-12	0-6*

TABLE 4-3 BENEFITS OF PRETHAW MODES IN REDUCED COSTS FOR THERMALLY STABLE EMBANKMENTS

Prethaw Mode	Required Embankment Thickness		Material Saved Over Undisturbed	Net Savings @$6.39/m^3	Approx. Net Cost	Relative Benefit /Cost Ratio
	(m)	(ft)	(m^3/km)	($/km)	($/km)	
None - Hand Cleared	4.1	13.5	-0-	-0-	11,400	---
Stripped & Gravel	2.8	9.2	43,200	276,000	8,900	31.0
Stripped Only	2.5	8.2	51,800	331,300	8,900	37.2
Stripped + Gravel + Asphalt	2.1	7.0	60,600	387,200	26,400	14.7
Stripped+Gravel+Asphalt+Poly	2.0	6.6	64,300	410,900	34,900	11.8



(a)



(b)

FIG. 4-1 (a) CLEANING AND STRIPPING EFFECTS
(b) THERMAL DEGRADATION UNDER ROAD SECTION

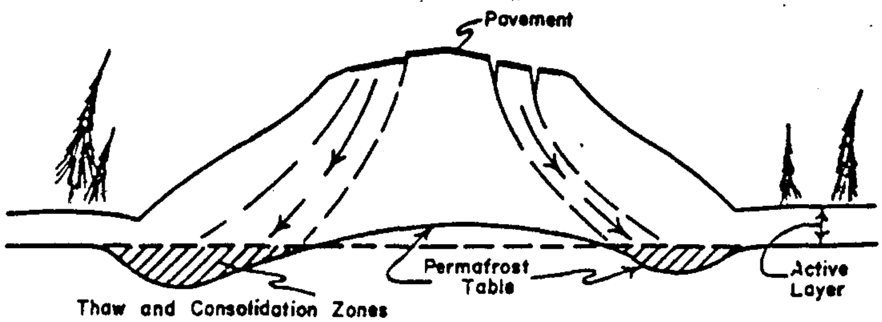


FIG. 4-2 EMBANKMENT STABILITY

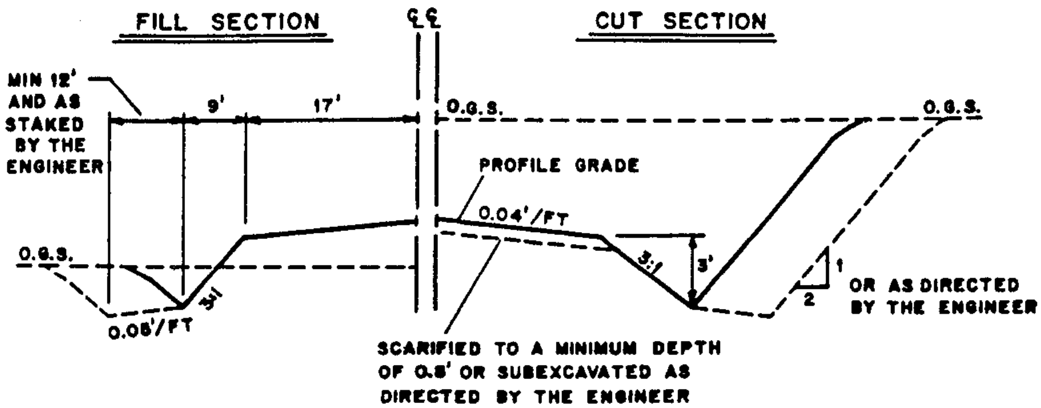


FIG. 4-3 TYPICAL SECTIONS

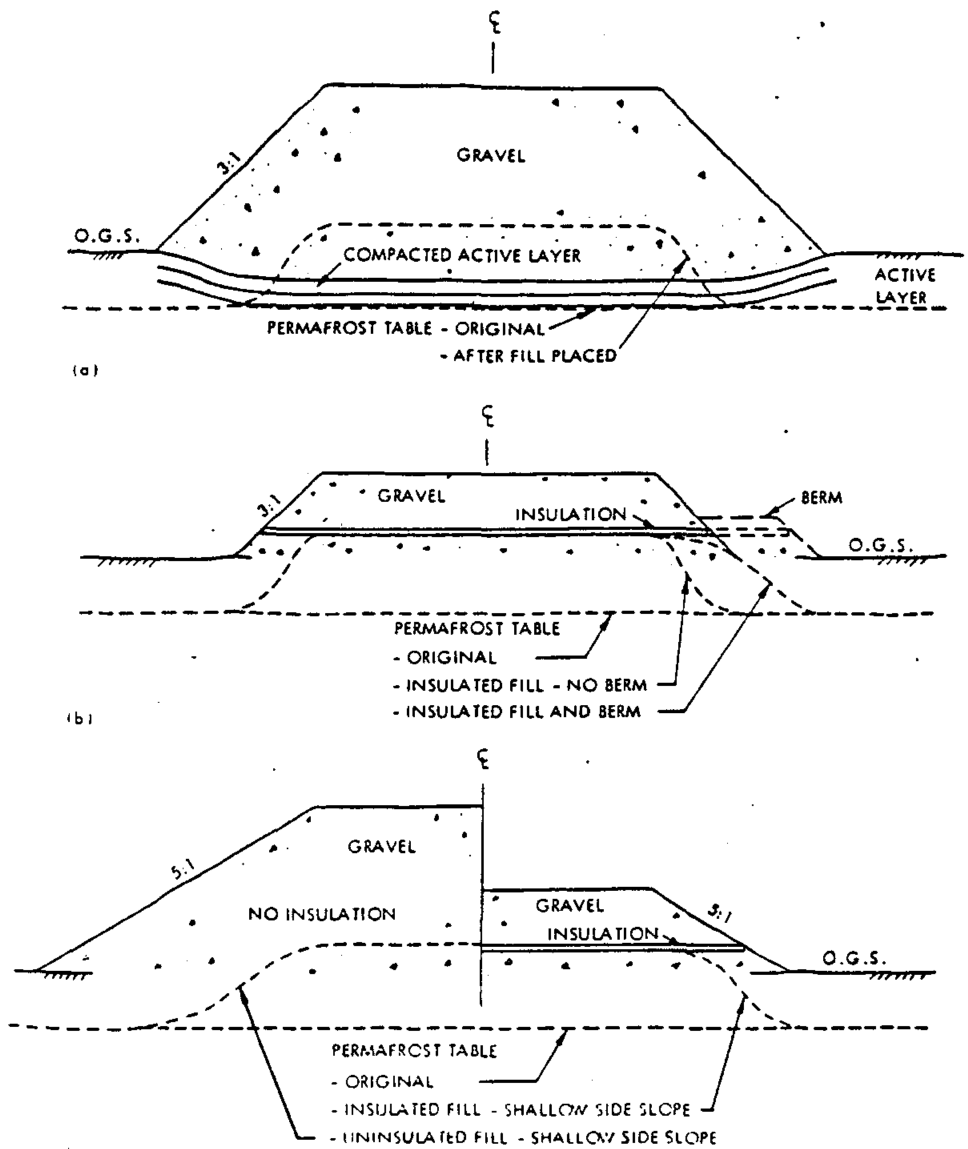


FIG. 4-4 EMBANKMENT WITH OR WITHOUT INSULATION

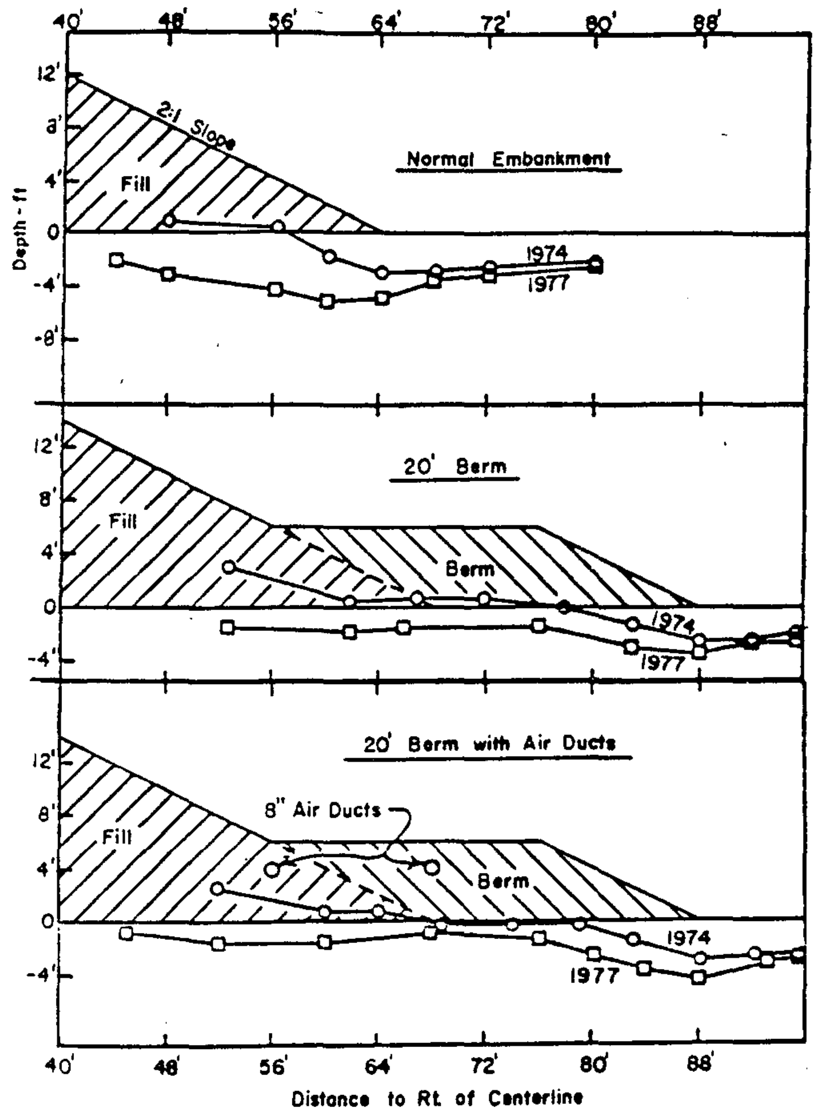


FIG. 4-5 BERM WITHOUT INSULATION (After Ref. 30)

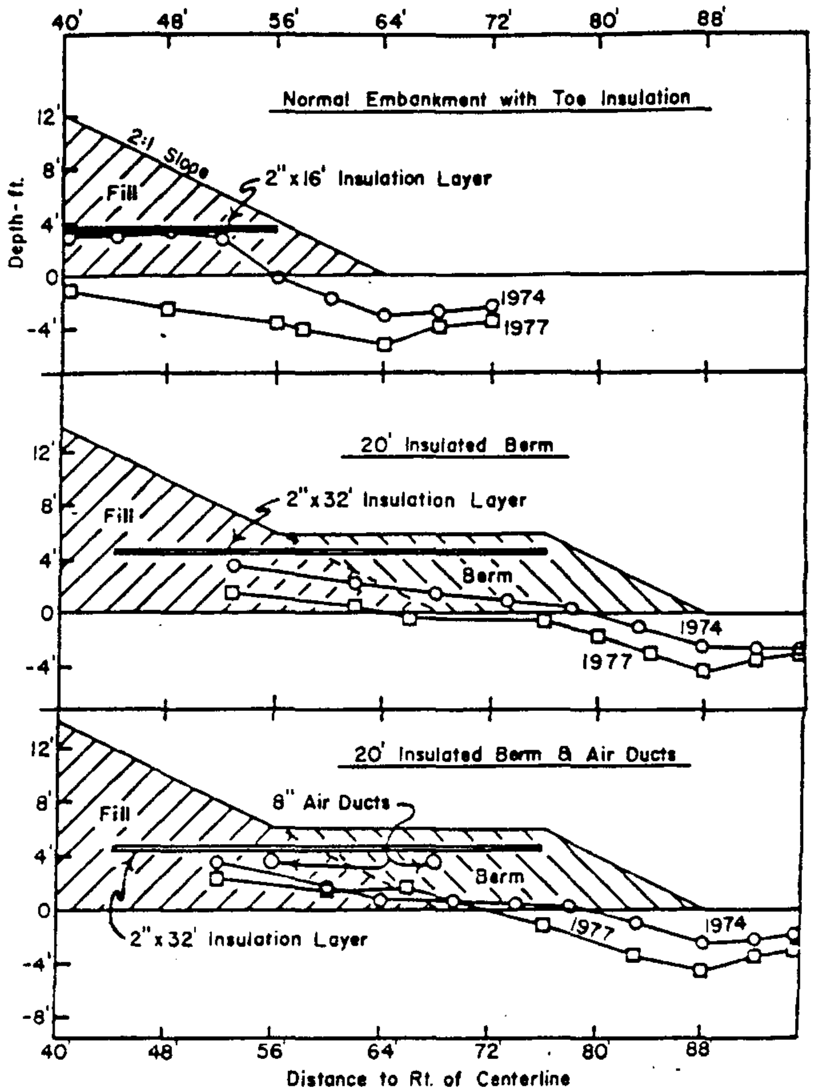


FIG. 4-6 BERM WITH INSULATION (After Ref. 30)

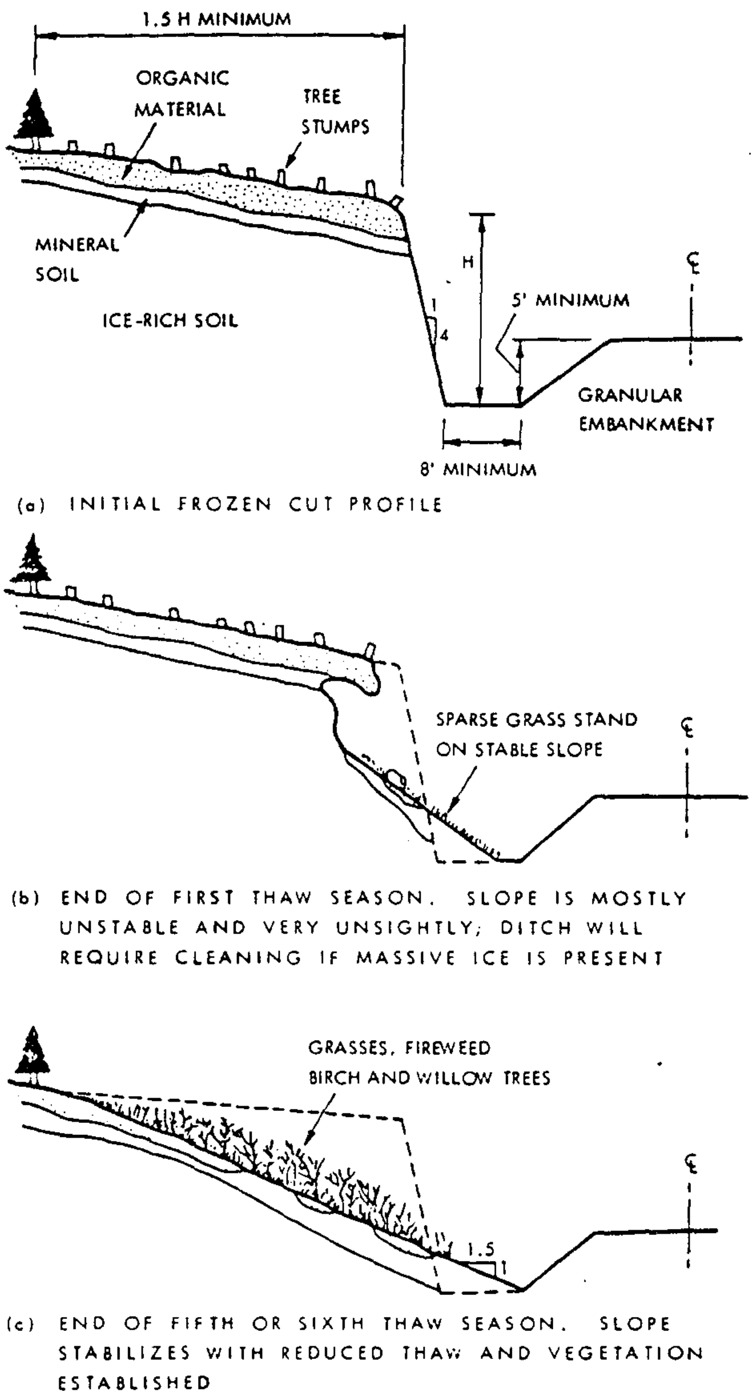


FIG. 4-7 STABILITY OF ICE-RICH CUT (After Berg and Smith, 1976)

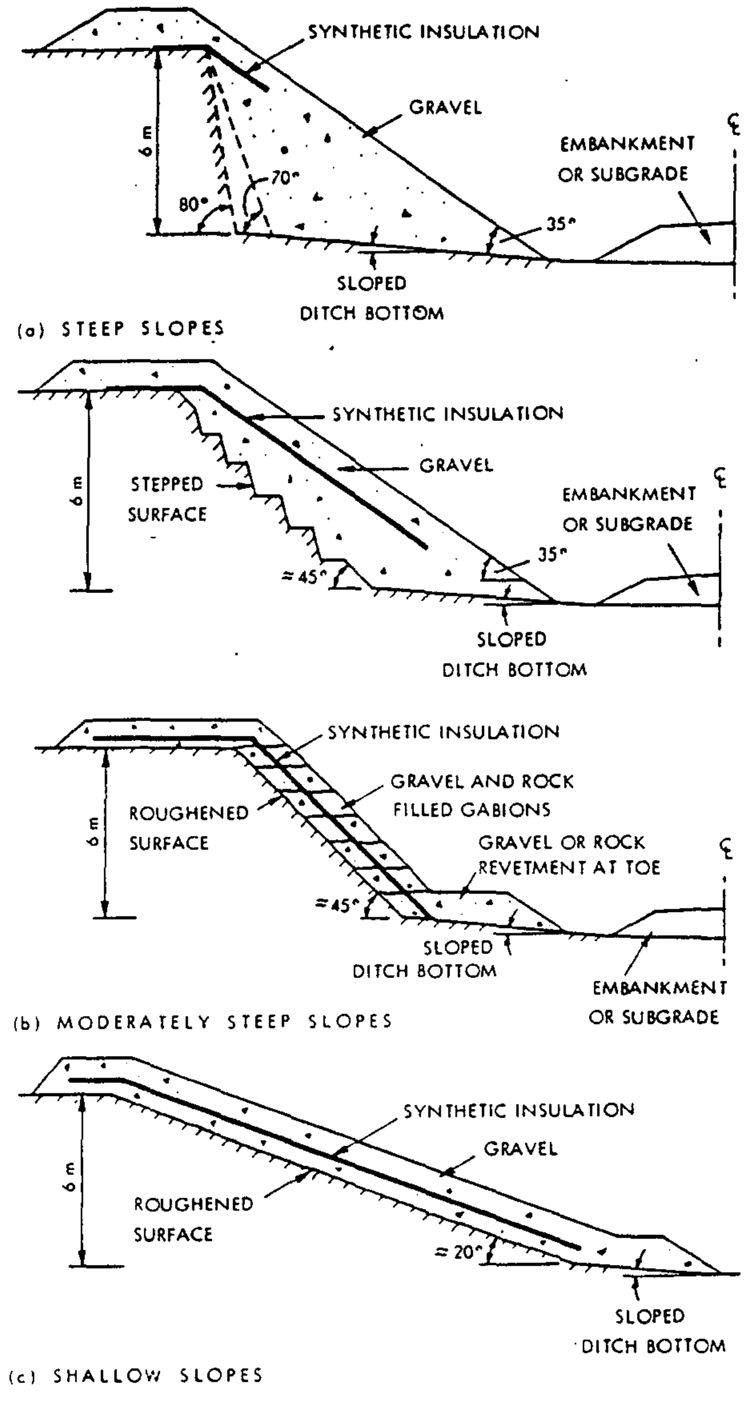
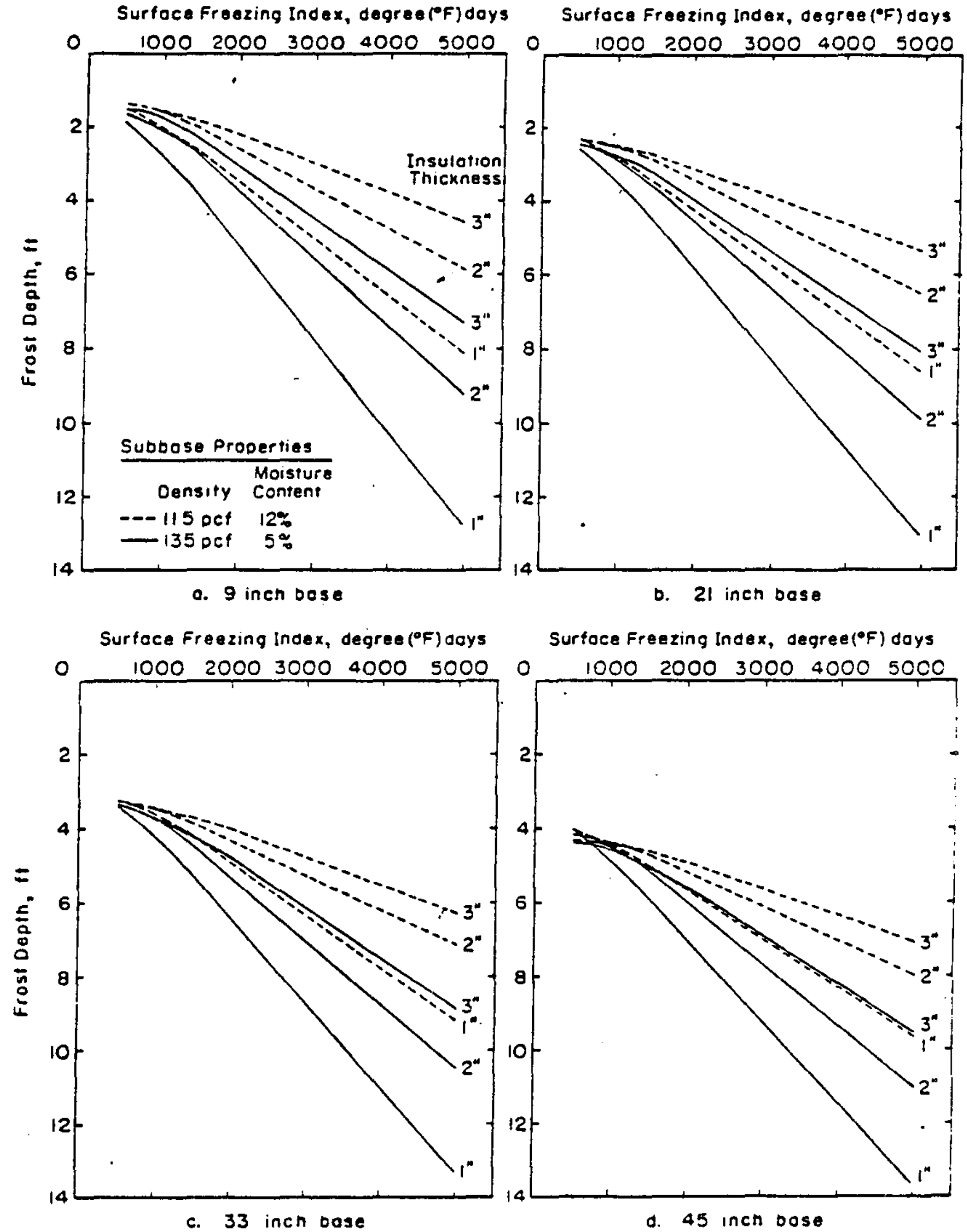


FIG. 4-8 STABILIZATION OF CUT SLOPES (After Pufahl, 1976)



Total depth of frost penetration through 3-in. asphalt pavement, base, insulation and subbase. Base density is 135 lb/cu ft and moisture content is 5%.

FIG. 4-9 FROST PENETRATION VS BASE THICKNESS (After Ref. 25)

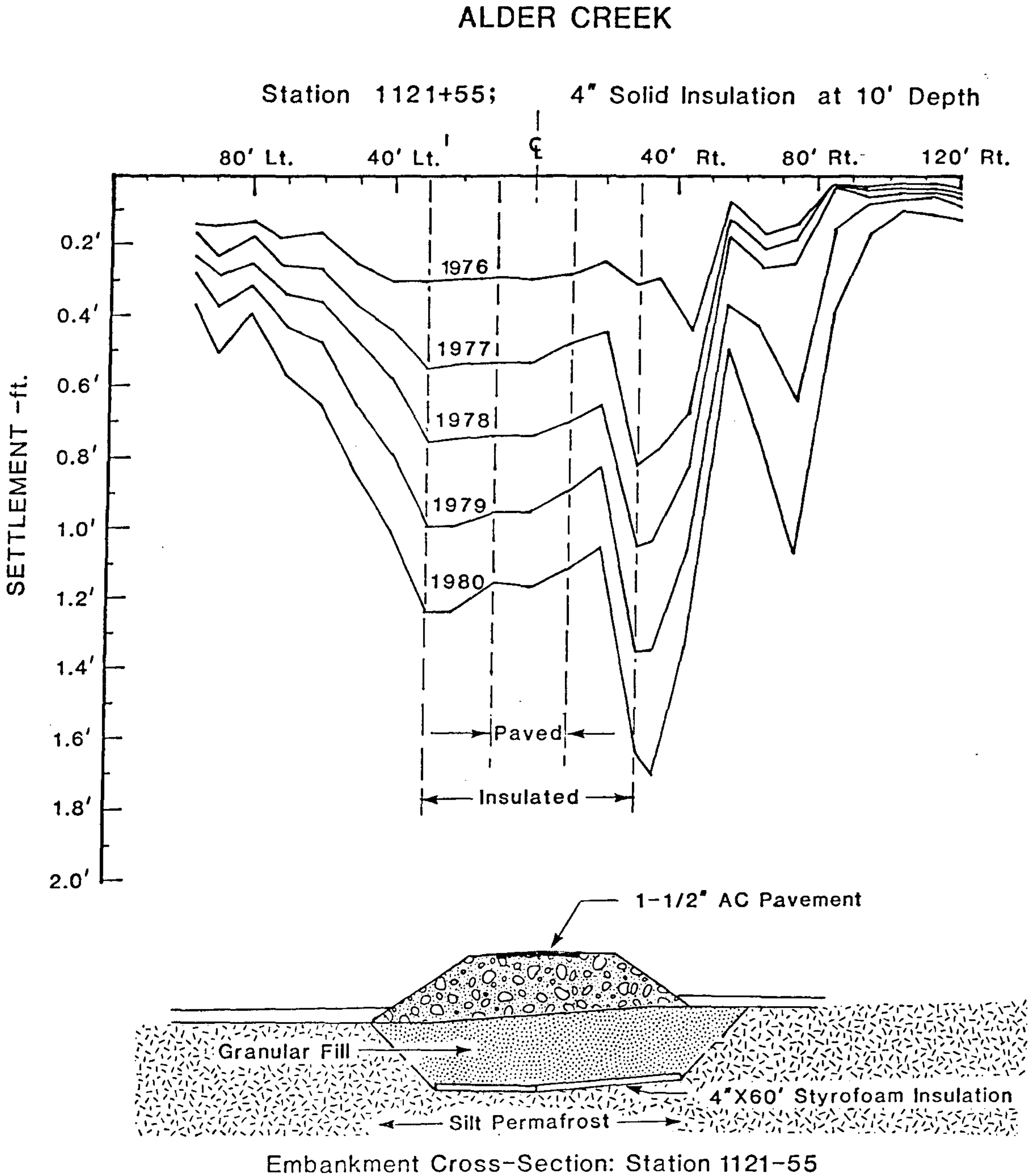


FIG. 4-10 THAW PENETRATION STUDY (After Ref. 30)

ALDER CREEK

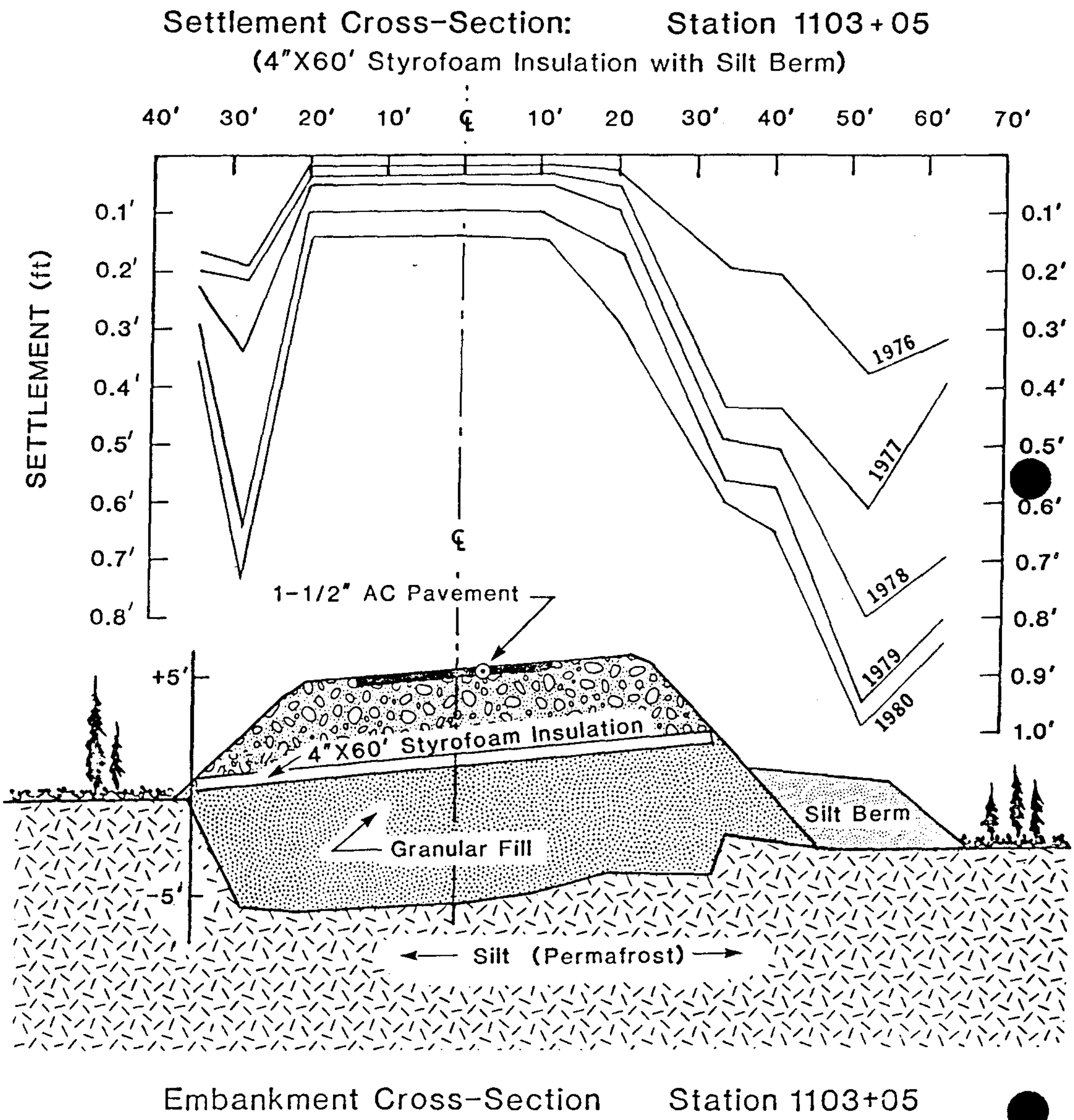
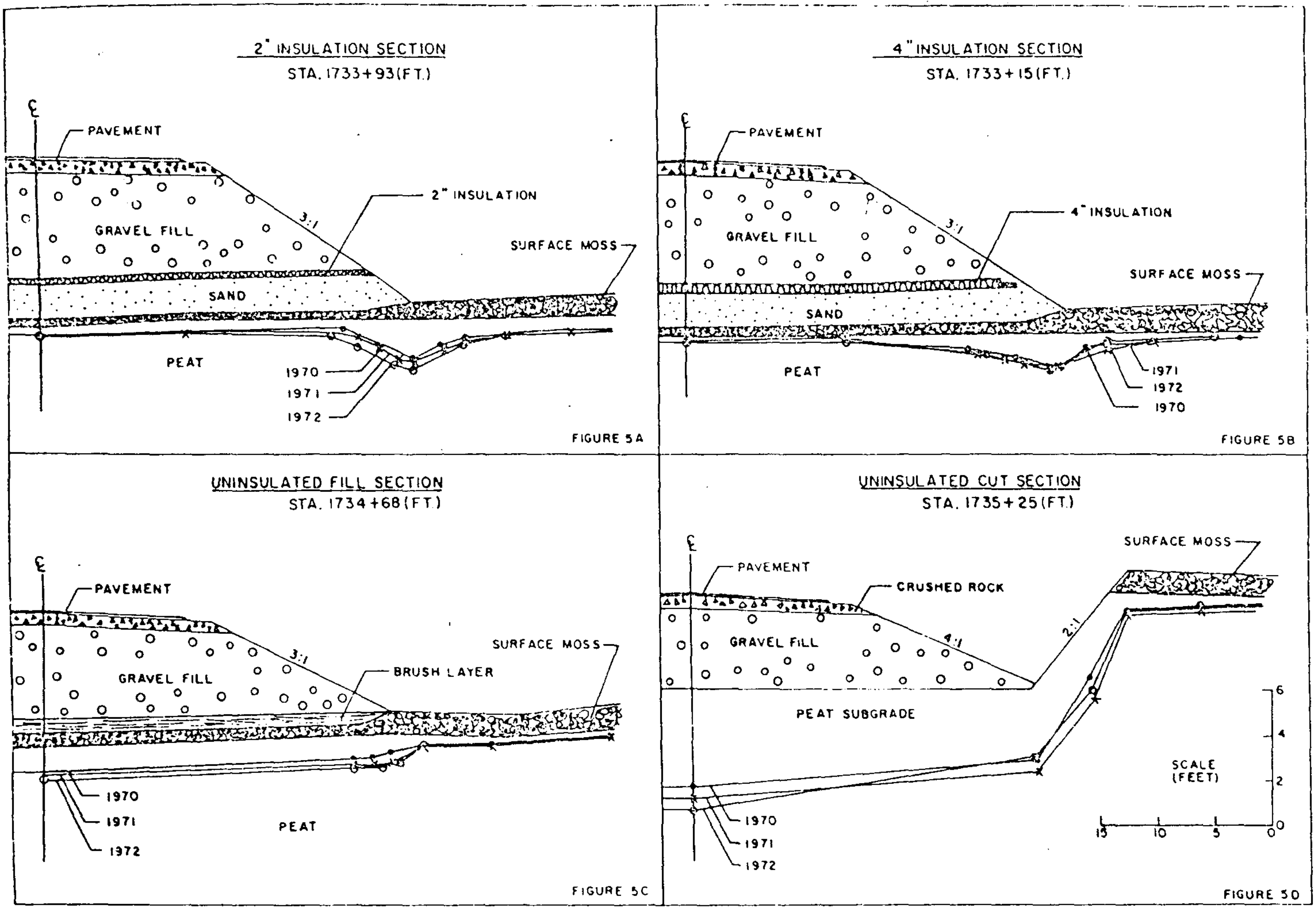


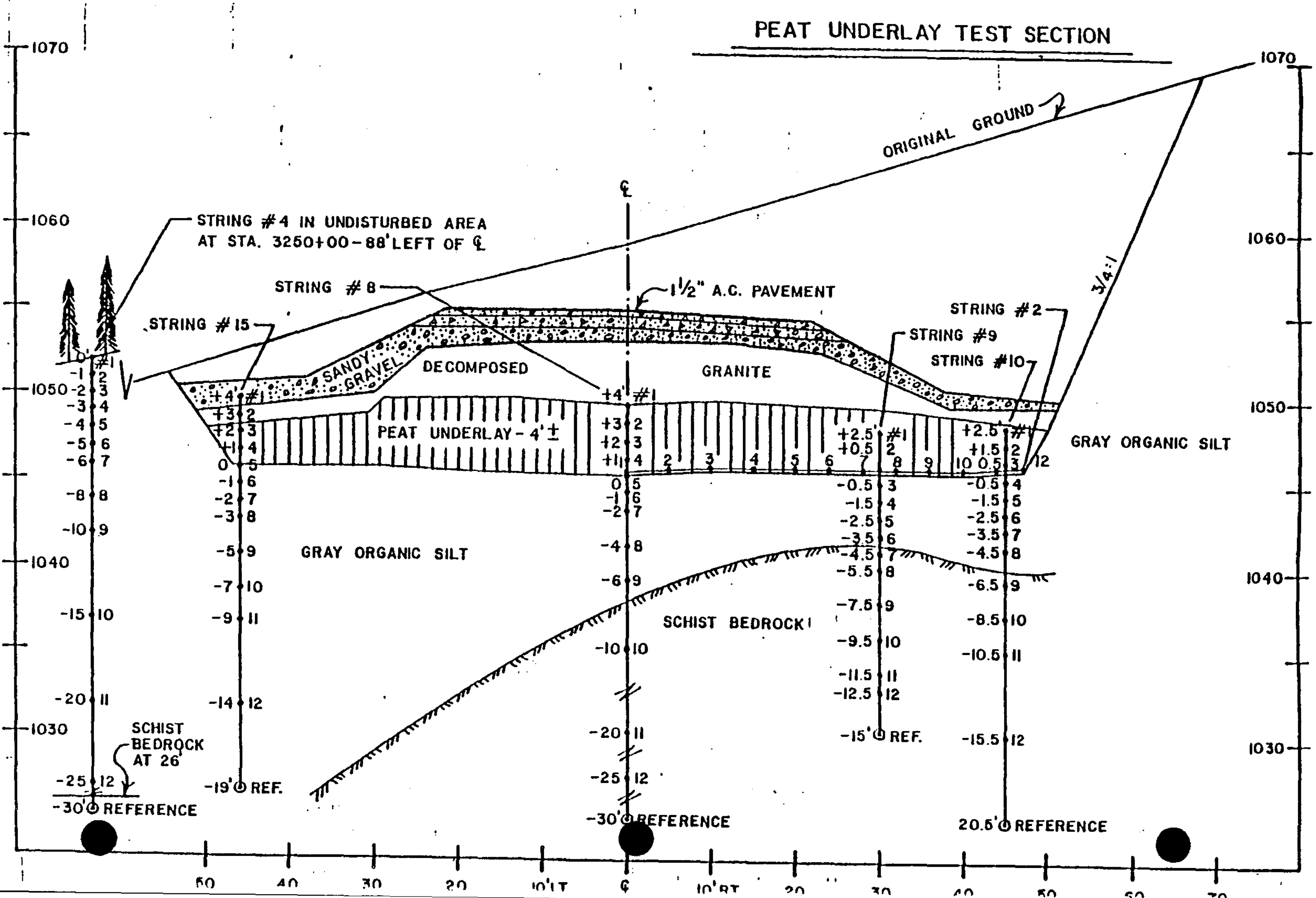
FIG. 4-11 THAW PENETRATION STUDY (After Ref. 30)



CROSS SECTIONS OF INSULATED AND UNINSULATED ROADWAY SECTIONS SHOWING MAXIMUM SEASONAL THAW DEPTHS.

FIG. 4-12 INSULATED AND UNINSULATED EMBANKMENT STUDY (After Ref. 36)

FIG. 4-13 PEAT OVERLAY TEST SECTION STUDY (After Ref. 37)



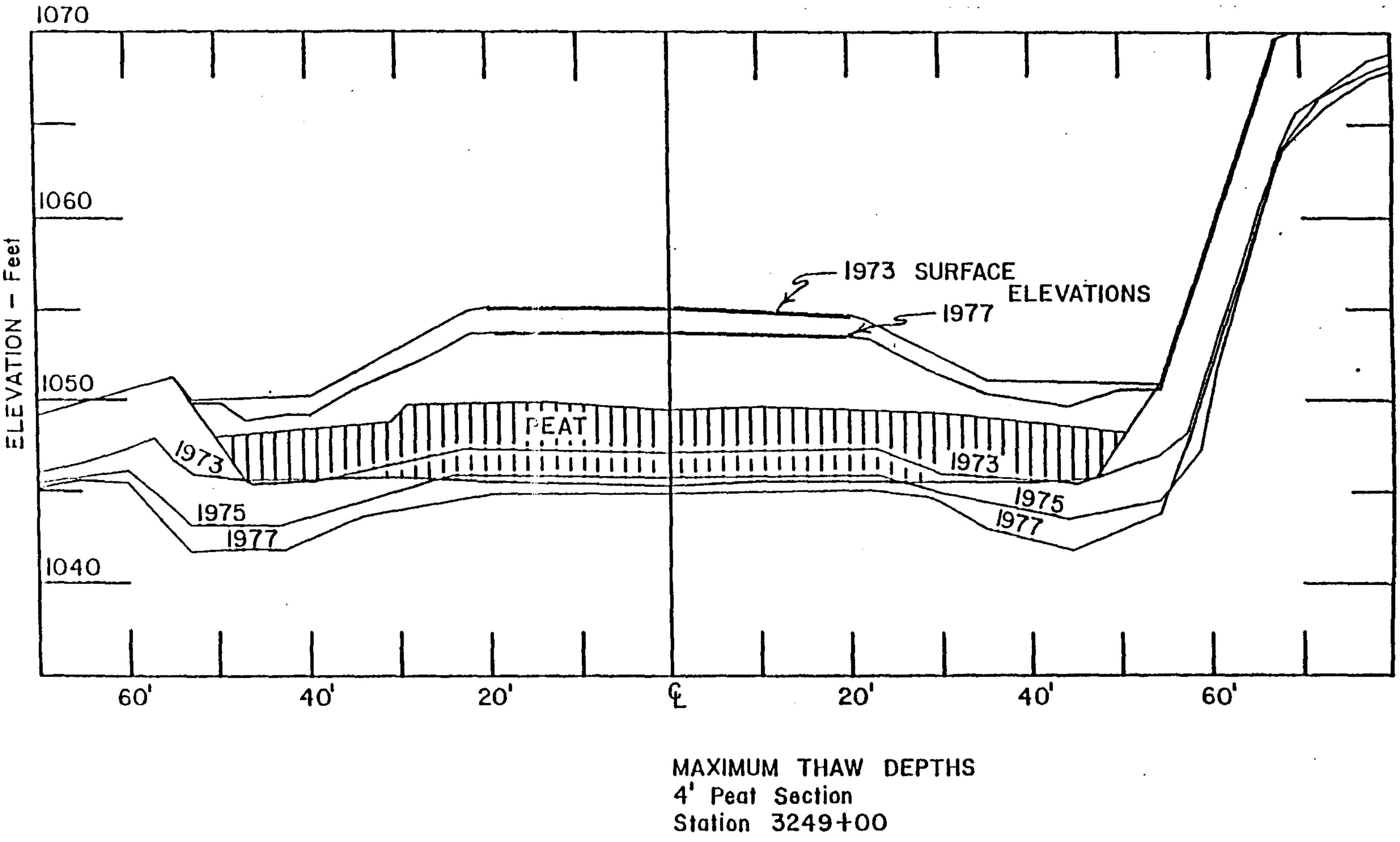
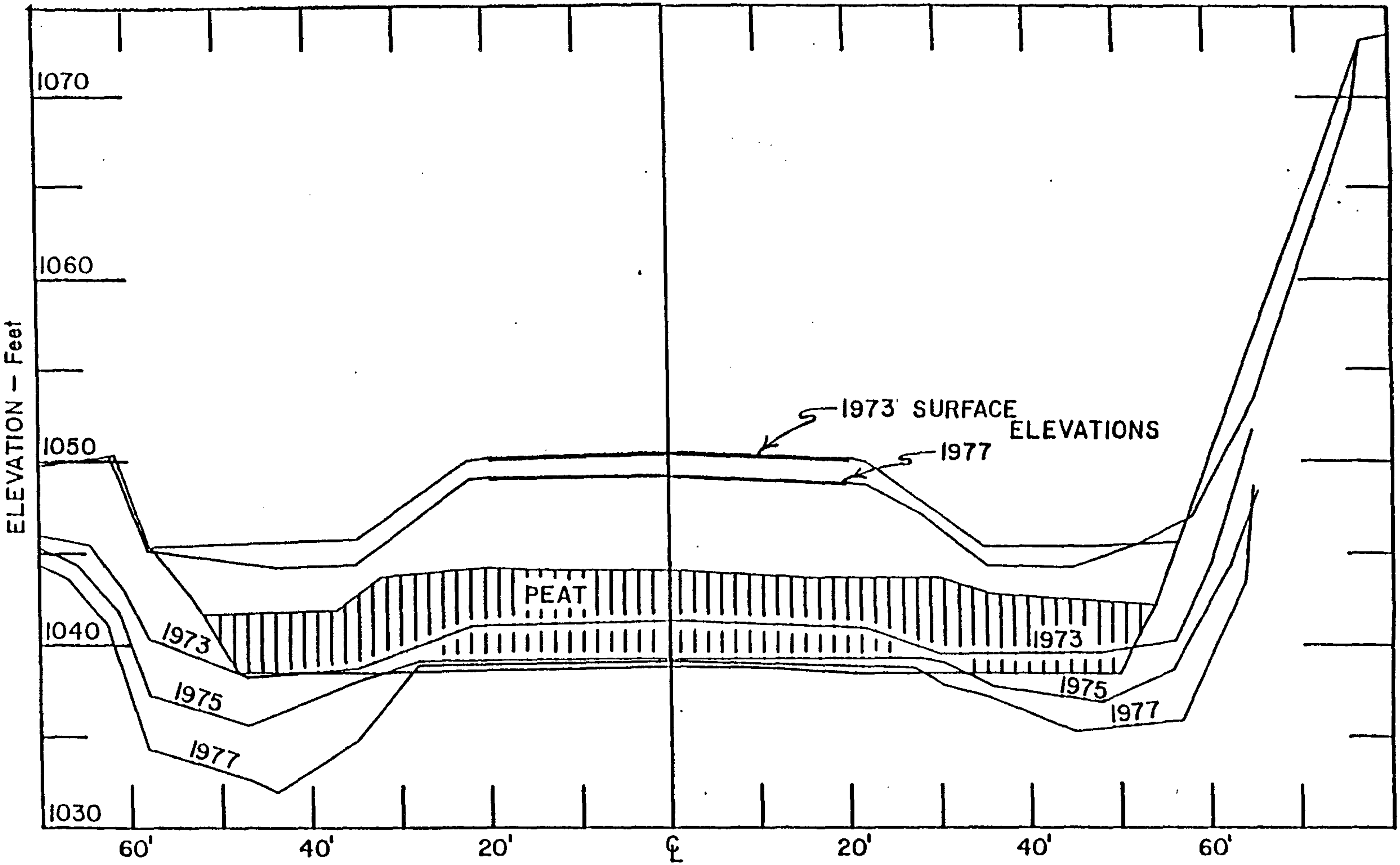


FIG. 4-14 THAW PENETRATION (4 FT PEAT SECTION) (After Ref. 37)



MAXIMUM THAW DEPTHS
5' Peat Section
Station 3250+62

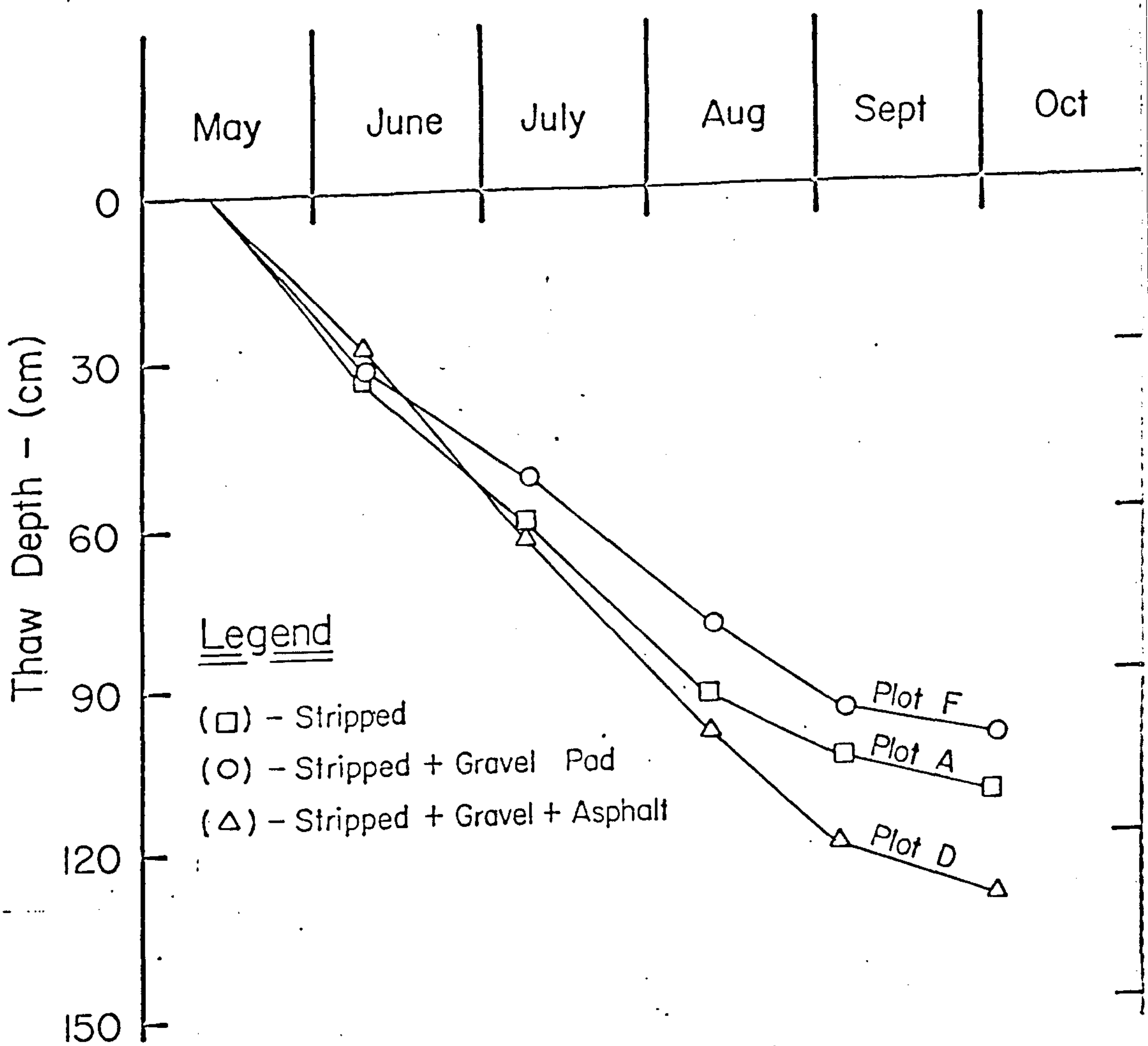


FIG. 4-16 THAW DEPTH STUDY (After Ref. 44)

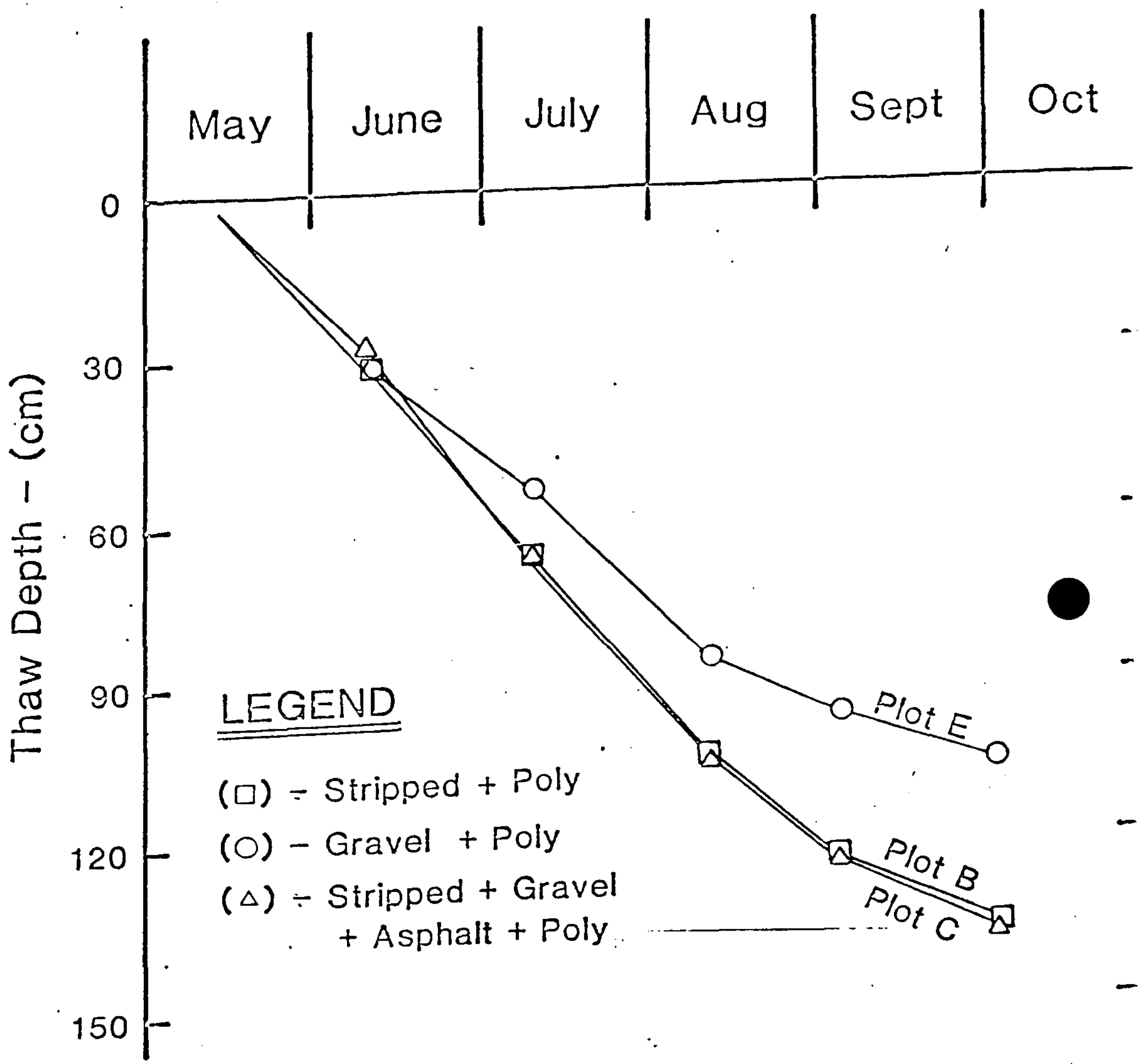


FIG. 4-17 THAW DEPTH STUDY (After Ref. 44)

5.0 CONSTRUCTION COSTS

5.1 General

The cost of a roadway section will include surveys, engineering design, right-of-way acquisition, supervision of construction, testing of materials and exploration of sites. Construction costs are described in this section. On highway projects cost per mile is generally used.

5.2 Cost Estimates

The construction elements of a typical roadway section are shown in Table 51. These items shown may be broken down further and individual costs may be calculated in terms of labor, materials and equipment. Three types of cost estimates which are commonly used in highway projects are conceptual or preliminary, final "engineers estimates", and contractor bid type. The more detailed and accurate the estimate, the more costly and time consuming the estimation process becomes. Several factors should be considered before a particular cost estimate process is selected and some of these factors are: degree of accuracy required at each stage of estimation, lowest cost estimate, amount of load involved in the project and availability of time.

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 prise index for the last seven years is shown in Fig. 5-1.

The in-place cost of excavation, hauling and placing borrow material for embankment construction have generally within the range of $10 to $15 per cubic yard in the permafrost regions of Alaska. The cost of base, subbase and fill materials varies from $10 to $30 per cubic yard depending on the location, hauling distance and availability of materials. The material transportation and placement cost for the synthetic insulation material used in insulated embankment would probably be in the range of $0.50 to $1.50 per square foot, depending on the type of insulation, thickness and method of transportation. The cost of insulation material varies from $0.35 to $0.50 per inch per square foot.

Considerably less volume of embankment material will be required to prevent thaw penetration into the subgrade permafrost if synthetic insulation material is used in comparison to utilizing gravel as the insulating material. Where NFS material costs are relatively high or where the in-place cost of insulation per square foot can be low, the synthetically insulated embankments could cost significantly less than their all-NFS material counterparts.

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

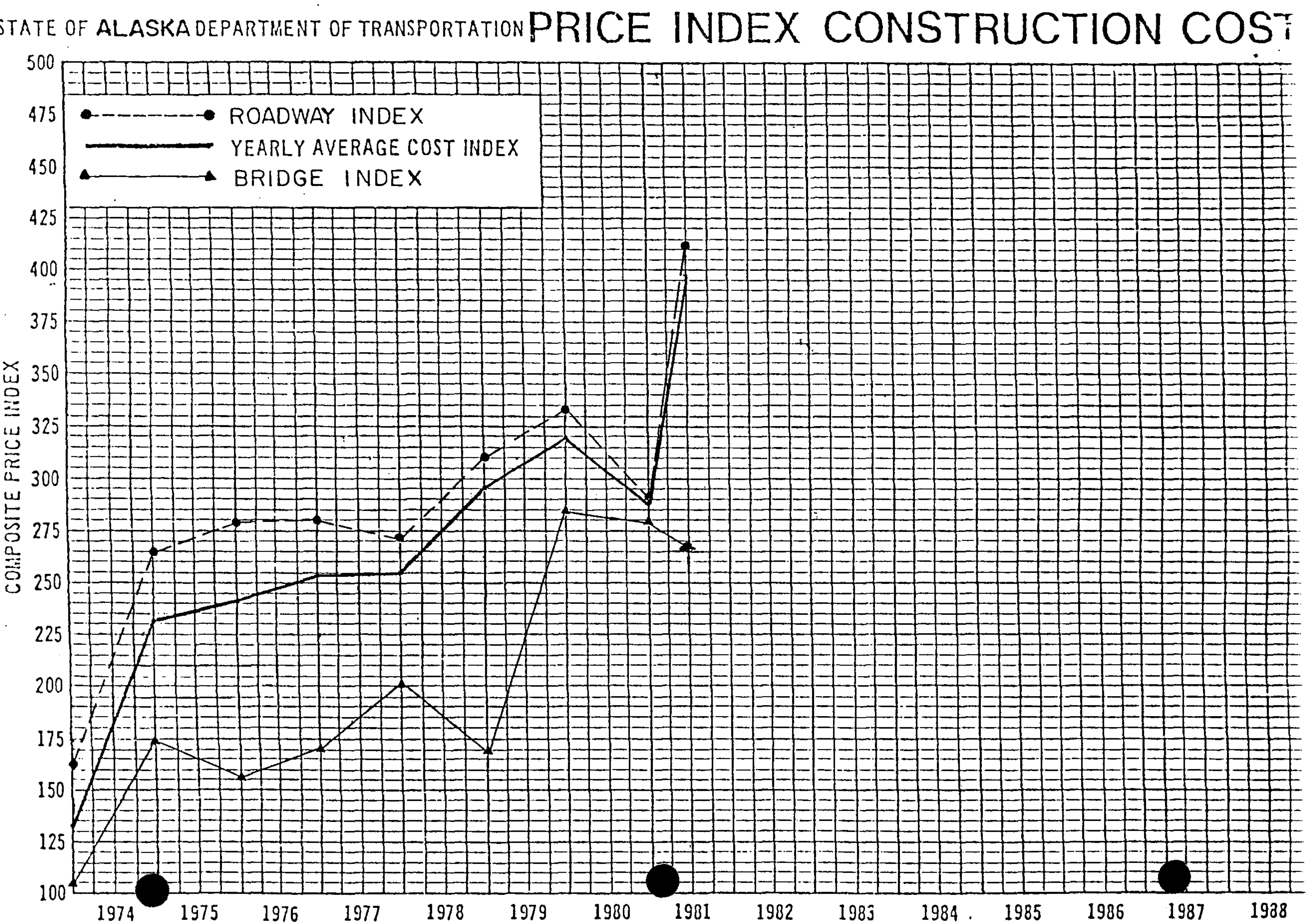
Unit of Equipment	Hrs.	Rate	Amount	Summary

Labor______________

Materials__________

Equipment__________

Total______________


STATE OF ALASKA DEPARTMENT OF TRANSPORTATION PRICE INDEX CONSTRUCTION COST
ROADWAY INDEX
YEARLY AVERAGE COST INDEX
BRIDGE INDEX
COMPOSITE PRICE INDEX
500
475
450
425
400
375
350
325
300
275
250
225
200
175
150
125
100
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
1985
1986
1987
1988

6.0 RECOMMENDATIONS

6.1 General

The fundamental differences between roadway design in permafrost areas and non-permafrost areas are that in permafrost areas considerations must be given to surface roughness, long term settlements, slope movements and thermal erosion caused by thaw weakening and thermal degradation and consolidation of cut slopes and subgrade soils. These differences have been recognized for several years, but a thorough understanding of the complex physical and thermal processes causing these effects has not yet been achieved. Mathematical models have been developed coupling surface heat exchange, heat and mass flux. Stress and strains in multilayered systems and behavior of frozen-thawed soils under different temperature gradient and dynamic loading configurations and temperature profiles may be analyzed only by complex computer programs. For different climatic conditions and surface characteristics, frost susceptibility of soils with different fines, and the thaw-strain of soils with different ice contents and cumulative damage potentials, must be determined.

It has been shown that various design and construction concepts are available for roadways in permafrost areas. However, the prediction methods of anticipated heave or differential settlement under different subsurface conditions and loading conditions are deficient. 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 permafrost areas.

6.2 Research on Factors Fundamental to Frost Susceptibility of Soils

The entire area of frost heaving versus thaw settling related roadway deterioration has not been adequately examined for permafrost regions.

Essentially all damage noted is considered to be related to thaw settlements, but seasonal heaving certainly also occurs to compound the problem. Soil factors that determine the severity of ice segregation and frost heave must be evaluated under the temperature gradients generally encountered in the permafrost regions. The moisture migration generated by the freezing plane is to be ascertained as a function of time, temperature and space. Once the thermal regimes and soil properties are known, an accurate prediction of moisture equilibrium conditions will help to predict the potential volumetric expansion of a soil.

6.3 Research on Factors Fundamental to Thaw Weakening

The thaw weakening mechanism is to be evaluated in terms of temperature gradient, soil type and ice composition, excess pore water pressure generation and dissipation, and dynamic loading configurations. Design parameters, such as CBR R-value which occur during thawing periods, are to be evaluated in terms of effective variables. The resilient modulus and shear strength under freeze-thaw cycles and stress cycles are to be studied.

6.4 Research on Material Characterization

Techniques are needed for material characterization with modeling of resistance of deformation, strength and volumetric equilibrium of various materials. Dependence of the properties on temperature, freeze-thaw cycles and access to moisture migration is to be determined.

6.5 Research on Different Construction Techniques

The use of insulation in the roadway construction is found to be of great promise. The effective construction procedures are to be determined for different topographic and subsurface conditions and climatic conditions. The combination of insulation and NFS materials to arrive at a most cost effective section for the discontinuous permafrost regions should be studied.

Various prethawing methods combined with consolidation processes appear to be very promising to reduce the required thickness of roadway embankment in discontinuous permafrost regions. Such techniques need studies for different permafrost condition, climatic regions, and drainage and topographic conditions.

The MESL technique has also promise as means of utlizing native poor soils to replace better, increasing scarce granular NFS material. Both field and laboratory experiments are required to define limitations as to soil types suitable for encapsulation and to determine the placement conditions such as optimum moisture and density that are necessary to minimize frost action.

The effectiveness of soil reinforcement in the subbase and the upper layers of permafrost needs to be assessed. Alternative systems providing soil strength during frost action and thaw settlement must be evaluated. The potential use of reinforced embankments to prevent sideslope movements and cracking should also be ascertained.

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GLOSSARY OF TERMS

Active Layer - the top layer of ground above the permafrost table that thaws each summer and refreezes each fall.

Axle Load - the total load transmitted by all wheels whose centers may be included between two parallel transverse vertical planes 40 inches apart, extending across the full width of the vehicle.

Base Course - the layer or layers of specified or selected material of designed thickness placed on a subbase or subgrade to support a surface course.

Depth of Thaw - the distance from the ground surface downward to frozen ground at any time during the thawing season.

Depth of Zero Annual Amplitude - the distance from the ground surface downward to the point beneath which there is virtually no annual fluctuation in ground temperature.

Flexible Pavement - a pavement structure which maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability.

Freezing Index - the number of degree-days (the difference between the mean temperature each day and 0°C, either positive or negative) between the highest point in the autumn and the lowest point the next spring on the cumulative degree-day time curve for one freezing season. The air freezing index is determined from temperatures measured about 1.4 m above the ground surface, while that determined from temperatures measured at or immediately below a surface is known as the surface freezing index.

Frost-Susceptible Soil - soil in which significant detrimental ice segregation occurs when the requisite moisture and freezing conditions are present.

Frozen Ground - soil or rock having a temperature below 0°C.

Ground Heave - upward movement of the ground causing a raising of the ground surface as a result of the formation of ground ice in excess of pore fillings.

Ground Settlement - downward movement of the ground causing a lowering of the ground surface resulating from the melting of ground ice in excess of pore fillings.

Ice, Excess - the ice in the ground that exceeds the total pore volume that the ground would have under natural unfrozen conditions.

Ice, Ground - ice in pores, cavities, voids or other openings in soil or rock, including massive ice.

Ice, Massive - a comprehensive term used to describe large (with dimensions measuring at least 10-100 cm) masses of underground ice, including ice wedges, pingo ice and ice lenses.

Ice, Segregated - ice formed by the migration of pore water to the freezing plane where it forms into discrete lenses, layers, or seams ranging in thickness from hairline to greater than 10 m.

Ice, Vein - a comprehensive term for ice in cracks where it occurs in bodies of various shapes, including tabular forms and wedges.

Ice Lens - 1. a dominantly horizontal lens-shaped body of ice of any dimension. 2. commonly used for layers of segregated ice that are parallel to the ground surface. The lenses may range in thickness from hairline to as much as about 10 m.

Ice Segregation - the process of formation of segregared ice by freezing of water in mineral or organic soil.

Ice Wedge - a massive generally wedge-shaped body with its apex pointing downward, composed of foliated or layered, vertically oriented, commonly white ice; from less than 10 cm to 3 m or more wide at the top, tapering to a feather edge at the apex at a depth of 1 to 10 m or more. Some ice wedges may extend downward as far as 25 m and may have shapes dissimilar from wedges. They may be "active" or "inactive," depending on whether they are or are not growing by repeated, but not necessarily annual (winter), cracking.

Modulus of Subgrade Reaction (k) - Westergaard's modulus of subgrade reaction for use in rigid pavement design (the load in pounds per square inch on a loaded area of the subgrade or subbase divided by the deflection in inches of the subgrade or subbase psi/in.).

Non-Frost Susceptible Soil - a soil that does not display significant detrimental ice segregation during freezing.

Patterned Ground - a general term for any ground surface of surficial soil materials exhibiting a descernible, more or less ordered and symmetrical, micro-physiographic pattern.

Pavement Performance - the trend of serviceability with loads applications.

Pavement Structure - a combination of subbase, base course and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.

Permafrost - the thermal condition in soil or rock of temperatures below 0°C persisting over at least two consecutive winters and the intervening summer; moisture in the form of water and ground ice may or may not be present. Earth materials in this thermal condition may be described as perennially frozen, irrespective of their water and ice content.

Permafrost, Continuous - permafrost occurring everywhere beneath the exposed land surface throughout a geographic regional zone, with the exception of widely scattered sites (such as newly deposited unconsolidated sediments) where the climate has just begun to impose its influence on the ground thermal regime and will cause the formation of continuous permafrost.

Permafrost, Degradation - a decrease in thickness and/or areal extent of permafrost because of natural or artificial causes as a result of climatic warming and/or change of terrain conditions such as disturbance or removal of an insulating vegetation layer by fire or human means.

Permafrost, Discontinuous - permafrost occurring in some areas beneath the ground surface throughout a geographic regional zone where other areas are free of permafrost.

Permafrost, Ice-Rich - perennially frozen ground that contains ice in excess of that required to fill pore spaces.

Permafrost, Sporadic - permafrost occurring in the form of scattered permafrost islands in the more discontinuous permafrost zone.

Permafrost, Table - the upper boundary of permafrost.

Permafrost, Thaw Stable - perennially frozen soils that do not, on thawing, show loss of strength below normal, long-time thawed values nor producing ground settlement.

Permafrost, Thaw Unstable - perennially frozen soils that show, on thawing, a significant loss of strength below normal, long-time thawed values and/or significant settlement, as a direct result of the melting of the excess ice in the soil.

Permafrost Thickness - the vertical distance between the permafrost table and the permafrost base.

Polygon - a type of patterned ground consisting of a closed, roughly equidimensional figure bounded by several sides, commonly more or less straight but some, or all, of which may be irregularly curved. A polygon may be either "low centre" or "high centre," depending on whether its centre is lower or higher than its margins.

Rigid Pavement - a pavement structure which distributes loads to the subgrade, having as one course a portland cement concrete slab of relatively high bending resistance.

Roadbed - the graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder.

Roadbed Material - the material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.

Seasonal Frost - seasonal temperatures causing frost (below 0°C temperatures) that affect earth materials and keep them frozen only during the winter.

Seasonally Frozen Ground - ground affected by seasonal frost.

Seasonally Thawed Ground - ground affected by seasonal thaw during the summer and seasonal frost during the winter.

Selected Material - a suitable native material obtained from a specified source such as particular roadway cut or borrow area, of a suitable material having specified characteristics to be used for a specific purpose.

Serviceability - the ability at time of observation of a pavement to serve high-speed, high-volume automobile and truck traffic.

Solifluction - the process of slow, gravitational, down slope movement of saturated, nonfrozen earth material behaving apparently as a viscous mass over a surface of frozen material. Solifluction features include lobes, stripes, sheets and terraces.

Thaw Consolidation - 1. the process of which a reduction in volume and increase in density of soil mass occurs, following thaw, in response to the escape of water under the weight of the soil itself and/or an applied load. 2. the process by which settlement due to thaw (thaw settlement) is impeded by the flow of water from the soil. Thaw consolidation is a time-dependent phenomenon that is not governed exclusively by the rate of thaw or position of the thaw front. It may proceed for many years.

Thaw Settlement - the generally differential downward movement of the ground surface resulting from escape of water on melting of excess ice in the soil and the thaw consolidation of the soil mass.

Thawing Index - the number of degree-days (the difference between the mean temperature each day and 0°C, either positive or negative) between the lowest point in the spring and the highest point the next autumn on the cumulative degree-day time curve for one thawing season. The air thawing index is determined from temperatures measured about 1.4 m above the ground surface, while that determined from temperatures measured at or immediately below a surface is known as the surface thawing index.

Thermokarst (topography) - the irregular topography resulting from the process of differential thaw settlement or caving of the ground because of the melting of ground ice in thaw unstable permafrost.

Tundra - a treeless, generally level to undulating region of lichens, mosses, sedges, grasses and some low shrubs, including dwarf willows and birches, which is characteristic of both the Arctic and high alpine regions outside of the Arctic.

Unfrozen Water Content - the ratio, expressed as a percentage, of the weight of unfrozen water to the weight of dry soil.

APPENDIX A

DEPTH OF THAW PENETRATION OR FREEZING CALCULATIONS

DEPTH OF THAW PENETRATION OR FREEZING

Formulas which can be used to calculate the depth of thaw penetration or freezing in homogeneous or multi-layered soils are presented in Section 3. The following examples are illustrated so that the reader can use the fundamental conceptions to solve various problems under different conditions.

Example A.1 Determine the depth of thaw penetration into a homogeneous frozen silt for the following conditions:

Mean annual temperature (M.A.T.) = 15°F
Surface-thawing index (nI) = 1200 degree-days
Length of thaw season = 110 days
Soil Properties: Dry unit weight (γ_d) = 100 pcg
Water content, ω = 15%

Solution:

Volumetric latent heat of fusion, L = 144(100) (0.15) = 2,160 B.t.u/cu ft.
Average volumetric heat capacity,

$$C_{avg} = 100\left(0.17 + 0.75 \times 0.15\right)$$
$$= 28.2 \text{ B.t.u./cu. ft °F}$$

Average thermal conductivity,

(unfrozen) K_u = 0.72 B.t.u./ft. hr °F (See Fig. A.1)
(frozen) K_f = 0.80 B.t.u./ft. hr °F (See Fig. A.1)
$K_{Avg} = 1/2\ (K_u + K_f)$ = 0.76 B.t.u./ft. hr. °F

Average surface temperature diffenertial

$$V_S = \frac{nI}{t} = \frac{1200}{110} = 10.91 \text{ °F}$$

Initial temperature differential

$$V_o = \text{M.A.T.} -32 = 15 -32 = 17°F \text{ (Below 32°F)}$$

$$\text{Thermal Ratio, } \alpha = \frac{V_o}{V_S} = \frac{17}{10.91} = 1.56$$

$$\begin{aligned} \text{Fusion parameter, } \mu &= V_S(C_{AV}/L) \\ &= 10.91 \left(\frac{28.2}{2160}\right) \\ &= 0.14 \end{aligned}$$

Lamba coefficient, $\Lambda = 0.79$ (See Fig. A.3)

Estimated deptth of thaw penetration,

$$S = \Lambda \sqrt{\frac{48\ K(nI)}{L}} \qquad \text{(From Eq. 3.8)}$$

$$= 0.79 \sqrt{\frac{48(0.76)(1200)}{2160}}$$

$$= \underline{3.6} \text{ ft.}$$

Example A.2 Determine the depth of thaw penetration beneath a asphalt concrete pavement for the following conditions:

Mean annual temperature (M.A.T.) = 15°F

Surface thawing index (nI) = 1560 degree-days

Length of thaw season = 105 days

Soil Boring Log:

Layer	Depth	Material	Dry unit Weight lb/cu.ft.	Water Content %
1	0.0-0.4	Asphalt Concrete	138	--
2	0.4-2.0	GW-GP	130	2.0
3	2.0-5.0	GW-GP	125	3.0
4	5.0-6.0	SM	110	7.0
5	6.0-8.0	SM-SC	105	5.0
6	8.0-9.0	SM	105	8.0

The V_S, V_0, and α values are determined in the same manner as that for the example A.1.

$$V_S = \frac{1560}{105} = 14.8°F$$

$$V_0 = 15\text{-}32 = 17°F$$

$$\alpha = \frac{17}{14.8} = 1.15$$

The thermal properties K, C, and L of the respective layers are obtained using the procedure described in example A.1.

The tabular arrangement shown in Table A.1 facilitates solution of the multi-layer problem, and in the following discussion, layer 3 is used to illustrate quantitative values. Column 9, 10, 12 and 13 are self-explanatory. Column 11, ($\overline{L}$), represents the average value of L for a layer and equal to Σ Ld/Σd = 443.8. Column 14, ($\overline{C}$) represents the average value of c and is obtained from Σ Cd/Σd = 24.1.

This $\overline{L}$ and $\overline{C}$ represents the weighted values of a depth of thaw penetration given by Σd, which is the sum of all layer thicknesses to that depth.

The fusion parameter, μ for each layer is determined from

$$V_S(\overline{C}/\overline{L}) = 14.8 \times \frac{24.1}{443.8} = 0.80$$

The Λ coefficient is equal to 0.55 (from Fig. A.3).

Column 18, R_n is the ratio d/K and for layer 3 equals 3.0/0.95 or 3.16.

Column 18, ΣR, represents the sum of the R_n values above the layer under consideration.

Column 20, $\Sigma R + (R_n/2)$, equals the sum of the R_n value of the layer being considered. For layer 3 this is $[2.14 + \frac{3.16}{2}] = 3.72$

Column 21, nI represents the number of degree-days required to thaw the layer considered and is determined from

$$nI = \frac{Ld}{24\lambda^2}\left(\Sigma R + \frac{Rn}{2}\right)$$

for layer 3

$$nI_3 = \frac{(540)(3)}{(24)(0{\cdot}3)} \times (372) = 837$$

The summation of the number of degree-days required to thaw layer 1 through 4 is 1736, Leaving (1736 - 1560) = 176. degree days to thaw layer 4. A trial and error method is used to determine the thickness of layer 4 thawed. First, it is assumed 0.5 ft. of layer 4 is thawed (designated as layer 4a). Calculations indicate 377 degree days are needed to thaw 0.5 ft. of layer 4a or 1560-1349=211 degree days less than available. A new layer 4b, is then selected by the following problem:

$$\left(\frac{211}{377}\right) 05 = 0.28 \text{ ft. say } 0{\cdot}3 \text{ ft.}$$

This new thickness results in 216 degree days required to thaw layer 5b or (1565-1560) or 5 degree days more than available. Further, trial and error is unwarranted and the estimated thaw penetration is 5.8 ft.

NOTE

A similar technique is used to estimate frost penetration in a multi-layer soil profile.

TABLE A.1 MULTILAYER SOLUTION

(1) Layer	(2) Dry Dens. (γd)	(3) Water Cont. (w)	(4) Dept. (d)	(5) Σd	(6) Vol. Heat Cap. (C)	(7) Ther. Conduc. (K)	(8) Latent Heat (L)	(9) Ld	(10) ΣLd
1	138	-	0.4	0.4	23.5	0.86	0	0	0
2	130	2	1.6	2.0	24.1	0.95	374.4	599.0	599.0
3	125	3	3.0	5.0	24.1	0.95	540.0	1620	2219
4	110	7	1.	6.0	24.5	1.02	1108.8	1108.8	3327.8
4a	110	7	0.5	5.5	24.5	1.02	1108.8	554.4	2773.4
4b	110	7	0.3	5.8	24.5	1.02	1108.8	332.6	3106.0

TABLE A.1 MULTILAYER SOLUTION (continued)

(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
$\bar{L}$	Cd	ΣCd	$\bar{C}$	μ	Λ	Λ^2	R_n	ΣR	$\Sigma R+\frac{R_n}{2}$
-	9.4	9.4	-	-	-	-	0.46	0	0.23
299.5	38.6	48.0	24.0	1.2	0.49	0.24	1.68	0.46	1.30
443.8	72.3	120.3	24.1	0.80	0.55	0.30	3.16	2.14	3.72
554.6	24.5	144.8	24.1	0.64	0.59	0.35	0.98	5.30	5.79
504.3	12.25	132.55	24.1	0.71	0.58	0.34	0.49	5.30	5.55
535.5	7.35	139.90	24.1	0.67	0.59	0.35	0.29	5.30	5.45

TABLE A.1 MULTILAYER SOLUTION (continued)

(21)	(22)
nI	ΣnI
-	-
55	135
837	972
764	$\underline{1736}$
377	1349
216	1565

$$I_2 = \frac{(3744)(1.6)}{(24)(0.24)} \times (1.30) = 135$$

$$I_3 = \frac{(540)(3)}{(24)(0.30)} \times (3.72) = 837$$

$$I_4 = \frac{(1108.8)(1)}{(24)(0.35)} \times 5.79 = 764$$

$$I_{4a} = \frac{(1108.8)(0{\cdot}5)}{(24)({\cdot}34)} \times 5.55 = 377$$

$$I_{4b} = \frac{(1108.8)(0{\cdot}3)}{(24)({\cdot}35)} \times 5.45 = 216$$

Total Thaw Penetration = 5.8 ft.

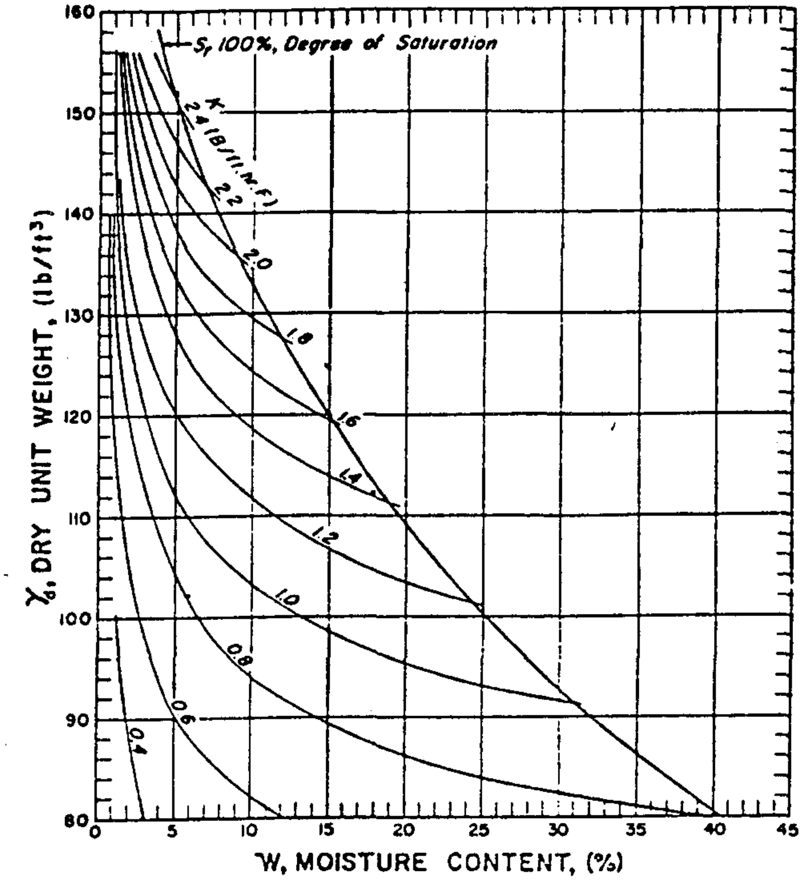


Figure A3. Dry unit weight, water content and coefficient of thermal conductivity for coarse-grained soils–unfrozen.[61]

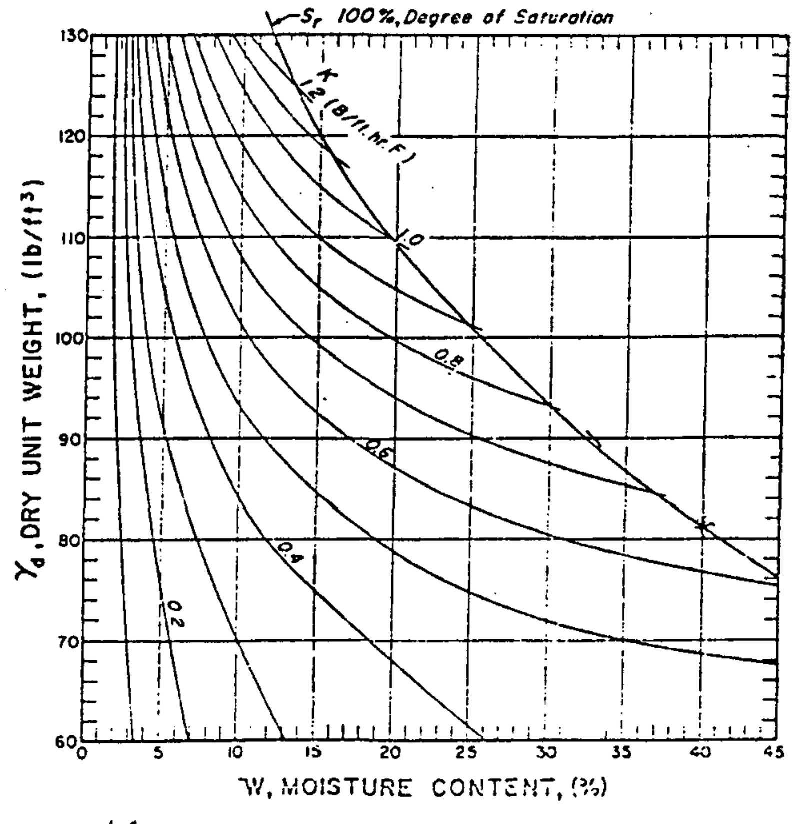


Figure A1 Dry unit weight, water content and coefficient of thermal conductivity for fine-grained soils–unfrozen.[61]

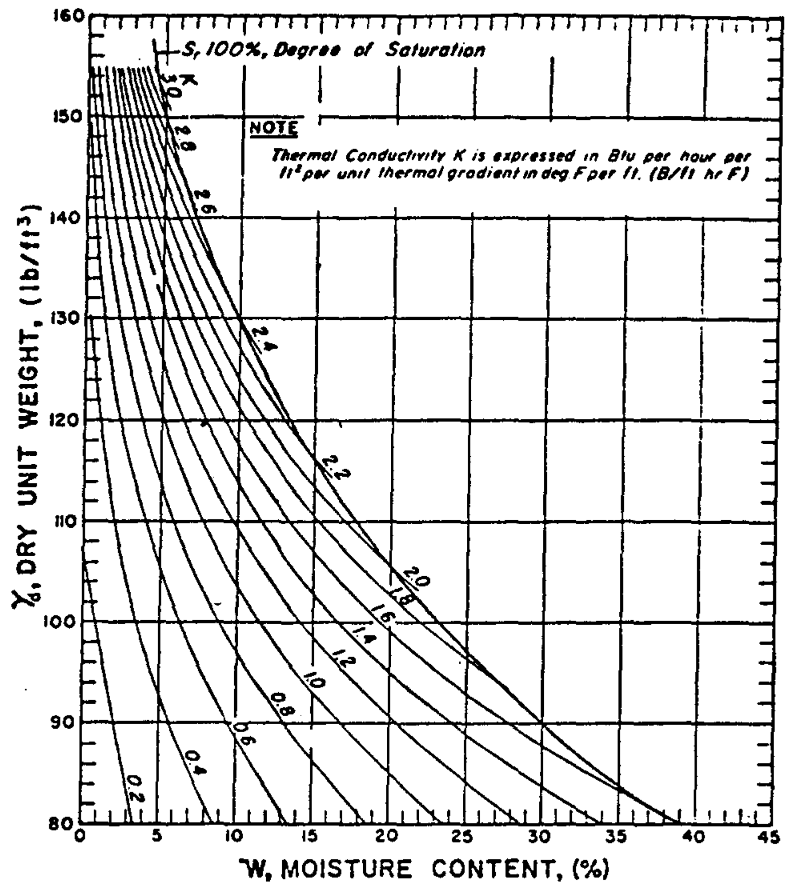


Figure A1. Dry unit weight, water content and coefficient of thermal conductivity for coarse-grained soils—frozen.[61]

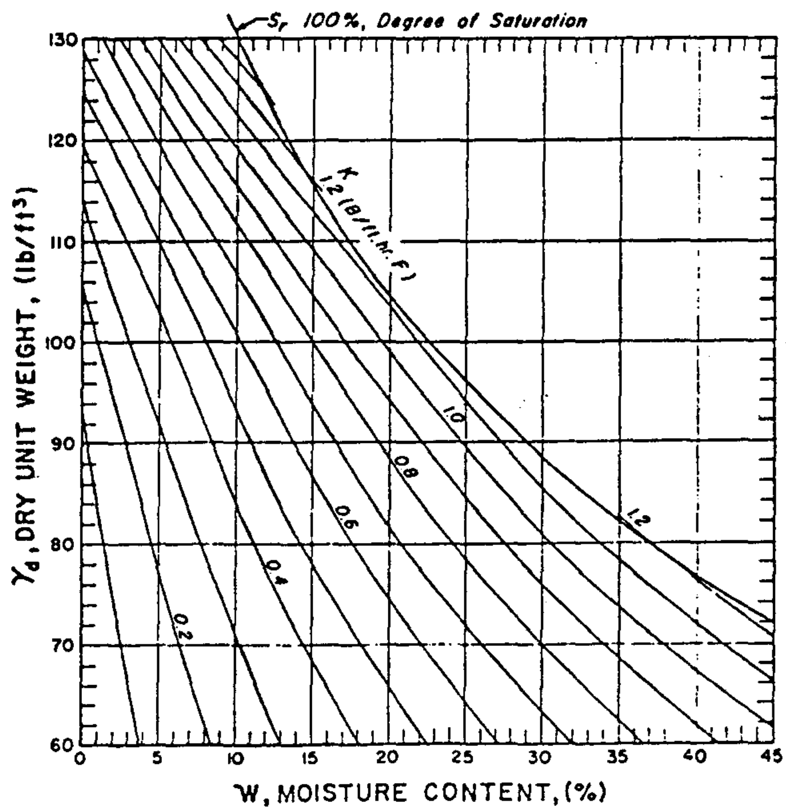


Figure A2. Dry unit weight, water content and coefficient of thermal conductivity for fine-grained soils—frozen.[61]

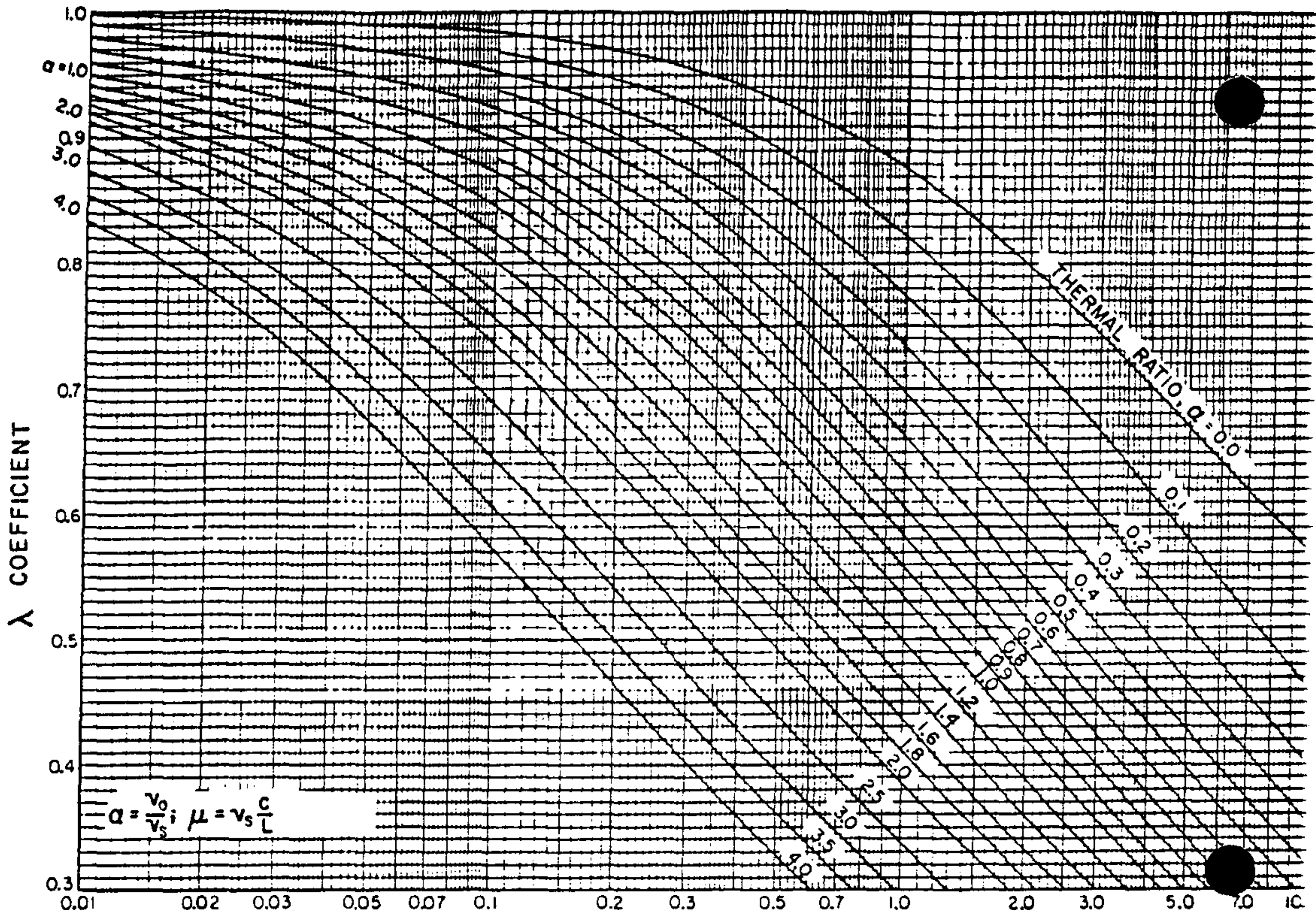
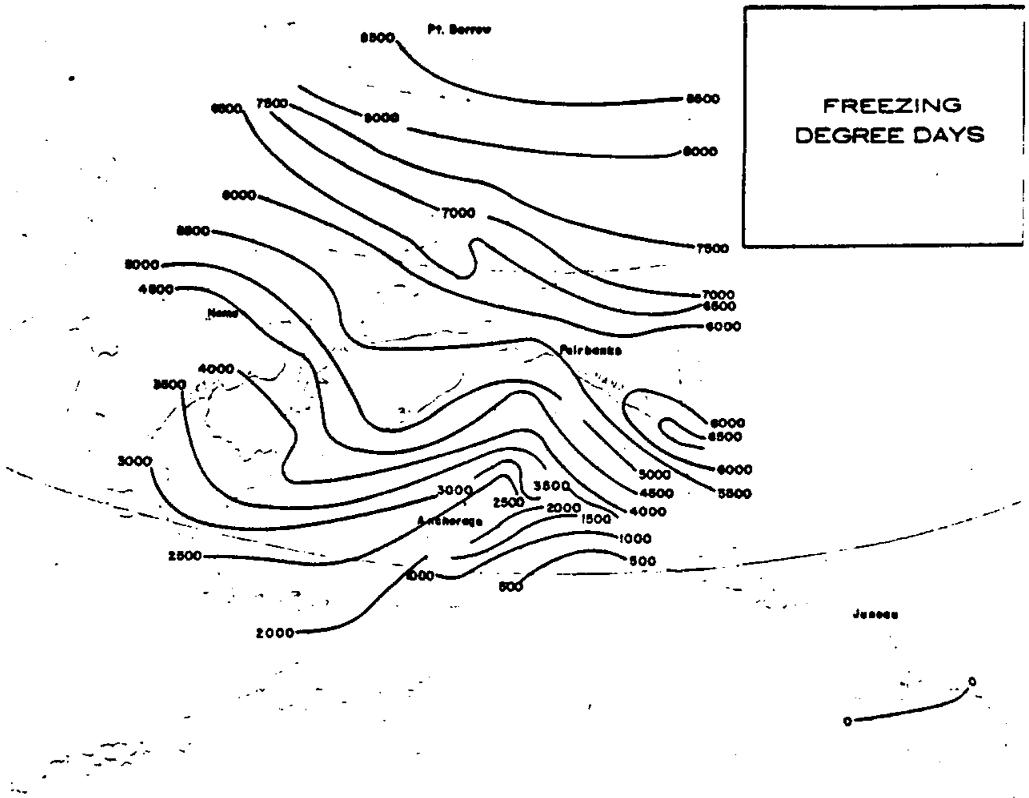


Figure A3 - μ, FUSION PARAMETER


FREEZING DEGREE DAYS
Pt. Barrow
Nome
Fairbanks
Anchorage
Juneau

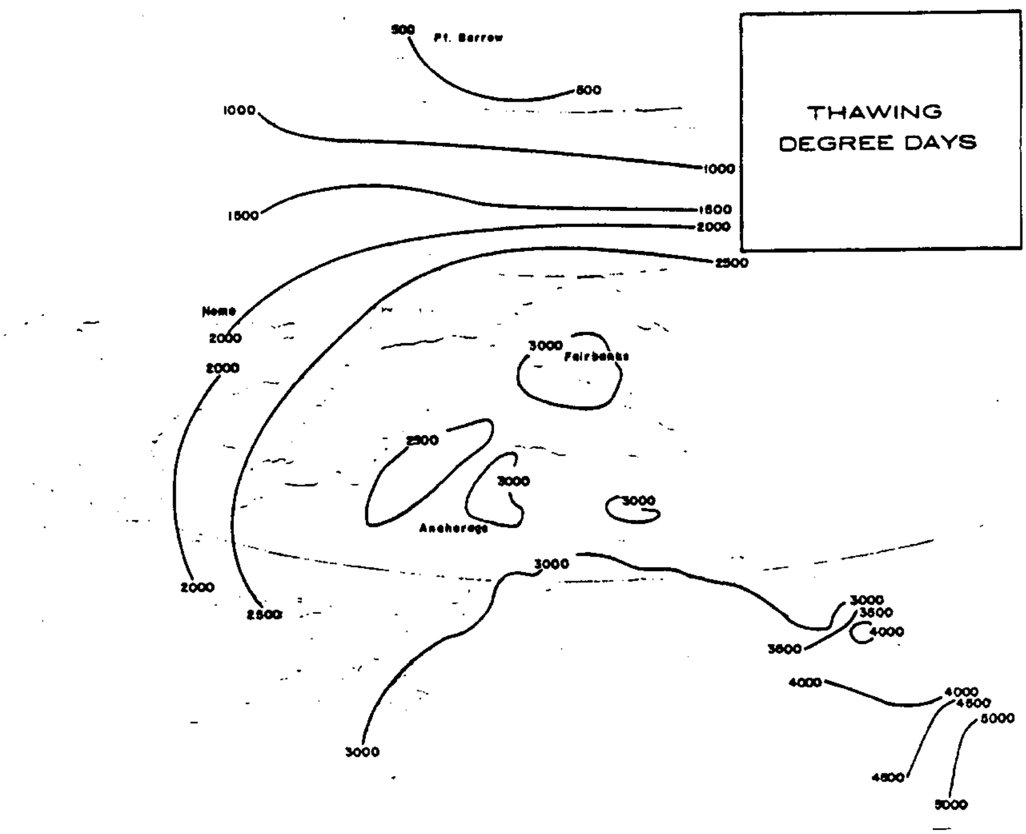

THAWING DEGREE DAYS
Pt. Barrow
Nome
Fairbanks
Anchorage

Figure A4.

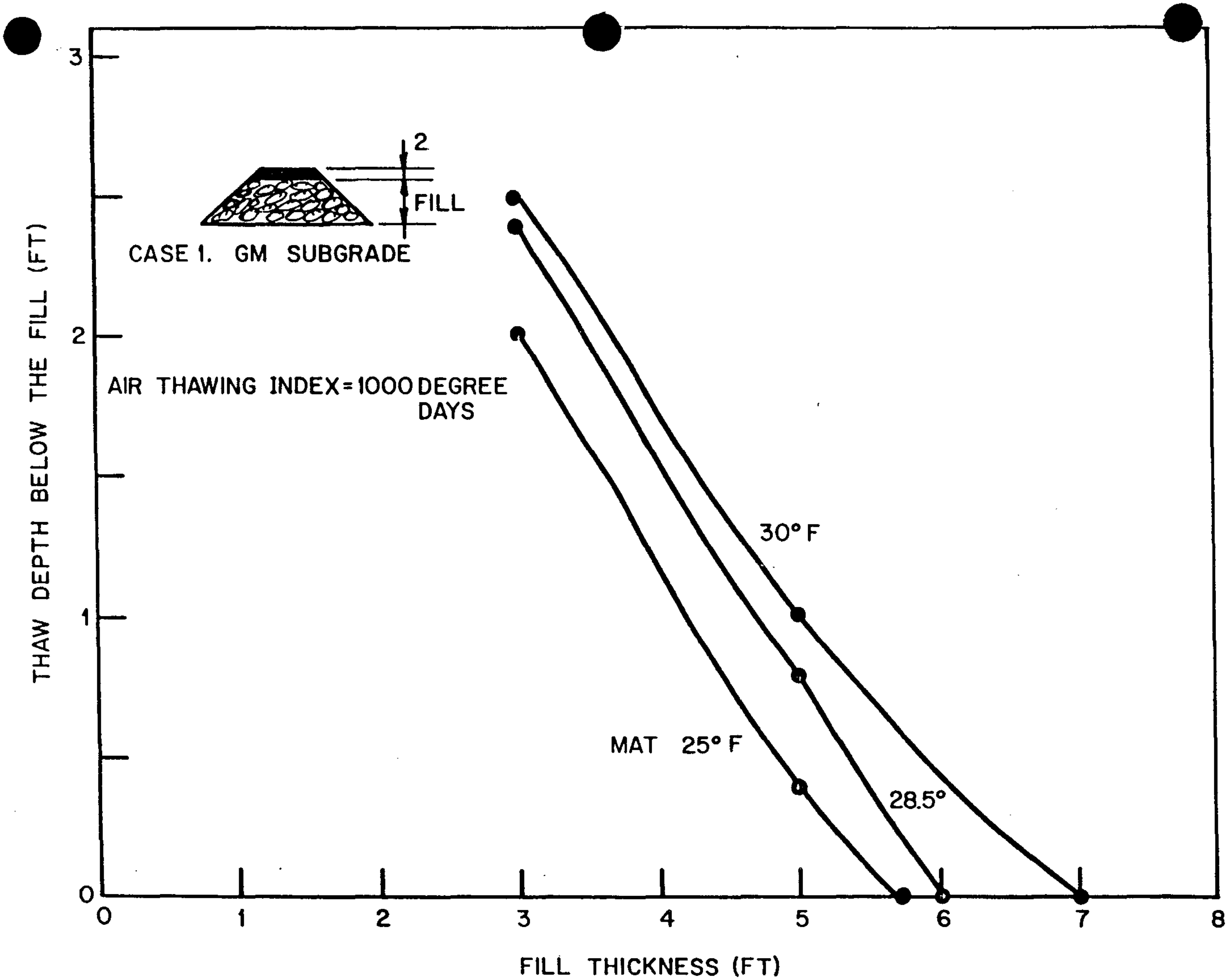

CASE 1. GM SUBGRADE
2
FILL
AIR THAWING INDEX = 1000 DEGREE DAYS
30° F
MAT 25° F
28.5°
THAW DEPTH BELOW THE FILL (FT)
FILL THICKNESS (FT)
0
1
2
3
4
5
6
7
8

Figure A5

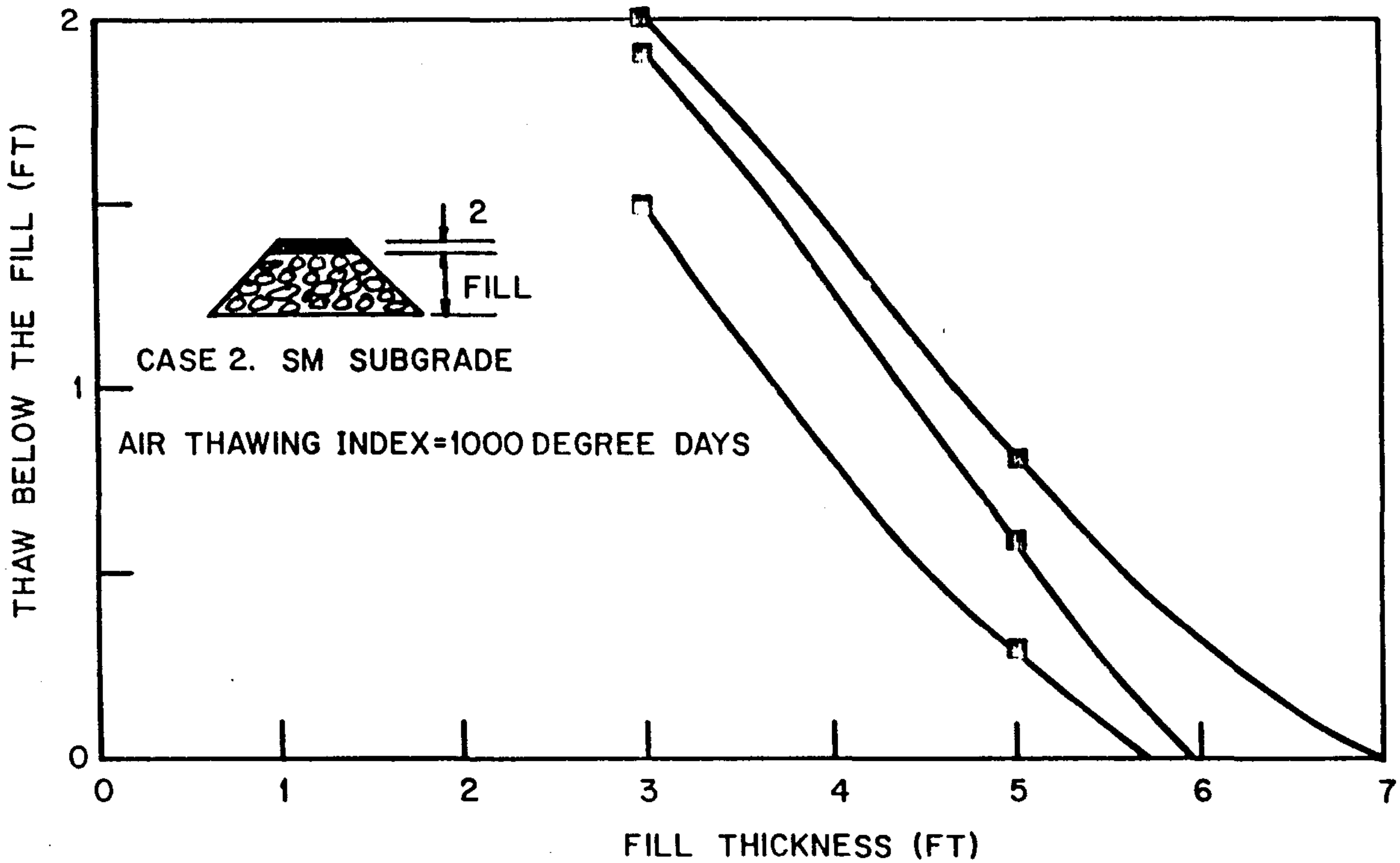
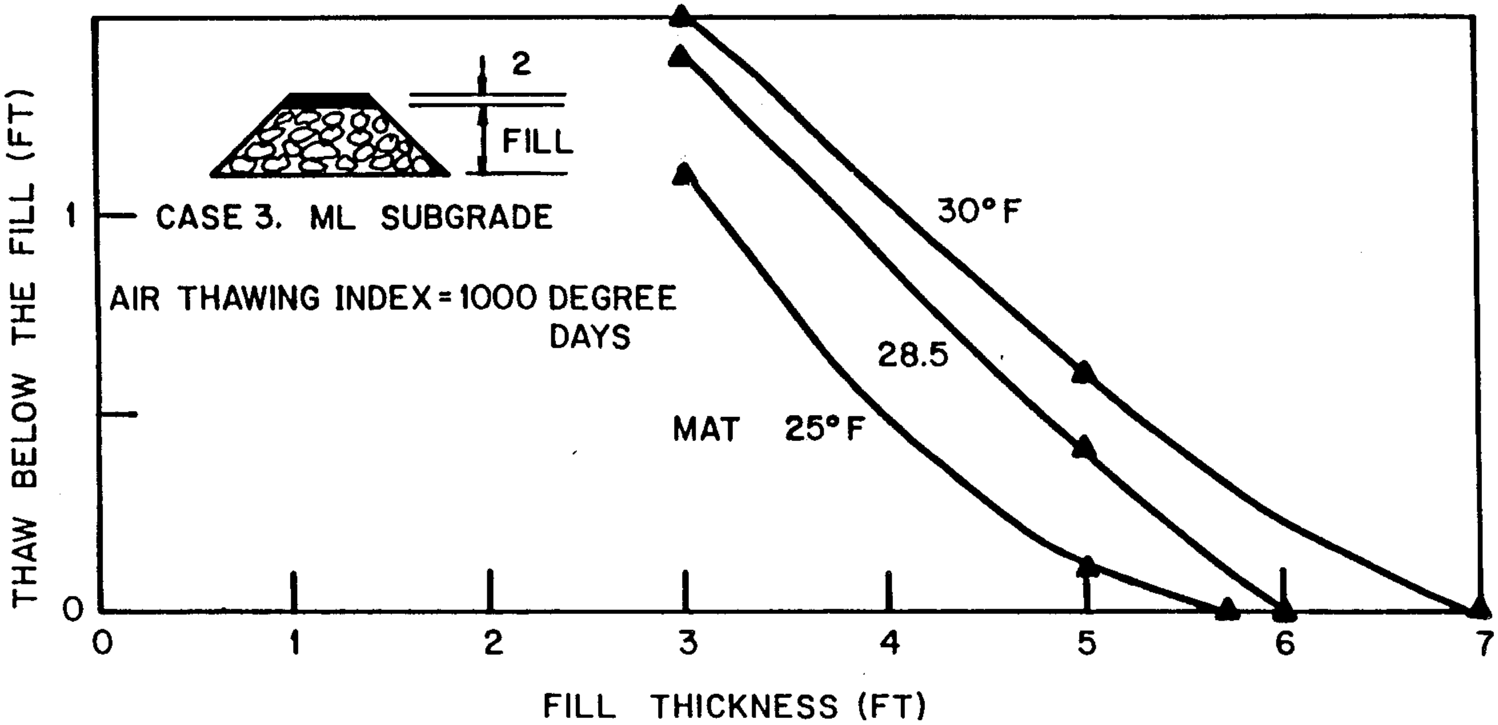

THAW BELOW THE FILL (FT)
2
1
0
0
1
2
3
4
5
6
7
FILL THICKNESS (FT)
2
FILL
CASE 2. SM SUBGRADE
AIR THAWING INDEX=1000 DEGREE DAYS

Figure A6



2
FILL
CASE 3. ML SUBGRADE
AIR THAWING INDEX = 1000 DEGREE DAYS
30° F
28.5
MAT 25° F
THAW BELOW THE FILL (FT)
FILL THICKNESS (FT)
0
1
0
1
2
3
4
5
6
7

Figure A7